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Introduction

Caltrans Division of Engineering Services, Geotechnical Services (GS), has responsibility for development and implementation of uniform standards, procedures, and guidance for the geotechnical products and services for State Highway System Projects. The Geotechnical Manual fulfills this responsibility by documenting the Department's geotechnical investigative, design, and reporting standards.

Administrative procedures prescribing GS business processes, such as project management and quality assurance, are presented in the Offices of Geotechnical Design Quality Management Plan (OGD-QMP).

Acknowledgements

The Caltrans Geotechnical Manual is the product of the combined efforts of many Geotechnical Services personnel working with the Office of Geotechnical Design Policies and Practices. Thanks for their work and appreciation for their contributions are extended here. An additional acknowledgement is given to Mr. Tony Allen of the Washington State Department of Transportation (WSDOT) for permission to use portions, whole sections, and chapters of the WSDOT Geotechnical Design Manual for this manual. The WSDOT Geotechnical Design Manual and Mr. Allen's participation in the 2006 FHWA review of the Caltrans geotechnical program provided the push for Caltrans to begin development of this manual.

Definitions

Term	Definition
Design Standard	A rule or requirement that establishes uniform engineering or technical criteria, methods, processes and practices. Includes investigations, design, and reporting.
Guidance	Advice about what you should do or how you determine a course of action, including institutional knowledge or practice.

The terminology used in this Manual for Standards and Procedures is as follows:

Term	Definition
Must, Required	The associated provisions **shall** be used.
Should	The associated provisions are preferred, or "best" practices.

Exceptions

Design standards associated with the term “must” or “required” require a written exception if not used. A *Request for Exception* form must be completed and signatures obtained prior to finalizing or issuing any related document. The completed *Request for Exception* form must be placed in the project archive and the exception(s) documented in the geotechnical report or project plans.

Design standards associated with the term “should” do not require written approval. However, they must have Branch Chief concurrence and be documented in the geotechnical report.

Scope

This manual is not a textbook, or a substitute for law, statue, regulation, code, knowledge, experience, or judgment. This manual includes Geotechnical standards and guidelines. In the absence of a documented standard or guidance, professional standards of practice should be followed. Questions regarding technical conflicts will be addressed by the Office of Geotechnical Design Policies and Practices or its delegate and its decisions are final.

Updates to the Manual

The manual will be reviewed and updated on a periodic basis to stay current with changing or accepted practice. Suggestions to update or revise the manual should have Branch Chief concurrence and be posted to the GS Discussion Board, Geotechnical Manual discussion forum.

FOUNDATION REPORTS
for
BRIDGES

January 2021



DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

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1 INTRODUCTION

The intent of this document is to define the Department's standard of practice for preparation of the Structures Preliminary Geotechnical Report (SPGR), the Preliminary Foundation Report (PFR) and the Foundation Report (FR). Standardized and consistent report presentations for projects statewide benefit the Department's staff, engineering consultants, bidders, and contractors. Geotechnical Services staff as well as any other organization preparing these reports must comply with the requirements presented herein.

1.1 Reporting for Caltrans Project Delivery

Foundation investigation and reporting generally occurs at three stages of the project development process:

- A Structures Preliminary Geotechnical Report (SPGR) to support Advanced Planning Studies, performed during the Work Breakdown Structure activity 150.15 (K Phase).
- A Preliminary Foundation Report (PFR) to support Type Selection, performed during the Work Breakdown Structure activity 160.10 (0 Phase) or 240.70 (1 Phase).
- A Foundation Report (FR) to support the design and construction of the bridge, performed during the Work Breakdown Structure activity 240.80 (1 Phase).

A separate foundation report must be prepared for each bridge structure, with the following additional requirements:

- Left, center, and/or right bridges with the same bridge number should be combined into one report.
- Earth retaining systems located within 150 feet of the bridge, or connected to the bridge, should be addressed in the bridge foundation report.

Prepare reports to succinctly communicate information pertinent to the recommendations in accordance with the report preparation requirements. The following rules must be followed:

- Use proper grammar, spelling and punctuation.
- Present only useful specific information that is relevant to the recommendations.
- Reference or cite existing standards, specifications or policy only when clarifying, modifying, or disallowing the standard, specification or policy.
- Do not include unsubstantiated disclaimers.
- Provide titles and numbers for all figures and tables.
- Tables and figures must be included within the body of the report and located as near as possible to the place where they are first referenced.
- All depth references must have a corresponding elevation in parenthesis.

1.1.1 Reports Prepared by Caltrans Staff

Foundation Reports are written to the Structure Designer, Specification Engineer, and Structure Construction, and are part of the contract.

For reports prepared by Geotechnical Services staff, Foundation Reports must be prepared using the current departmental memorandum format with the subject line of "Foundation Report for *Bridge Name*" or "Preliminary Foundation Report for *Bridge Name*" or "Structures Preliminary Geotechnical Report for *Bridge Name*". Do not include section numbers in the

report. First-level and second-level section titles presented in this document (e.g., Geotechnical Conditions, and Geology) must be included in the report. Other section titles are optional.

Do not include the Log of Test Borings (LOTB) and/or As-built LOTB as part of the FR. The Engineering Graphics Unit will send Microstation LOTB files and scanned copies of the As-built LOTB sheets to the Structure Designer for inclusion within the Contract Plans.

Sign and stamp reports in accordance with the *Communications and Reporting* section of the *Offices of Geotechnical Design – Quality Management Plan*.

1.1.2 Reports Prepared by Consultants

Foundation Reports must include the following: cover sheet, table of contents, main contents per this document, and appendices. The cover of the report and any addenda/amendments to the report must include the following information: Caltrans District, County, Route, Post Mile, Bridge Number, Bridge Name, and Expenditure Authorization (EA) number.

The LOTB and/or As-built LOTB must be submitted as part of the FR. Refer to the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* for direction on the preparation of the LOTB and As-built LOTB.

2 STRUCTURE PRELIMINARY GEOTECHNICAL REPORT (SPGR)

The SPGR is required during the early stages of a project to assist Structure Design in the preparation of an Advanced Planning Study and cost estimate for the District. Often the number, location, and type of bridge(s) are not completely known. As a result, recommendations may be general, and detailed field investigations are usually not warranted. Typical fieldwork consists of a site visit only. The SPGR provides an overview of the existing foundations, site geology, seismicity, and recommendations regarding suitable and unsuitable foundation types. If applicable, the SPGR should also discuss the anticipated field and laboratory work required to support the PFR and FR.

The following topics should be addressed in all Structure Preliminary Geotechnical Reports (SPGR).

2.1 Introduction

Summarize the purpose, scope, and types of work performed to obtain the information supporting the preliminary recommendations. Reference the request memo, preliminary plans by date so the reader knows on what plans the recommendations are based. Do not present an exhaustive list of tasks performed, a few sentences are sufficient.

Example

Per the request dated February 3, 2020, this Structure Preliminary Geotechnical Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the preliminary investigations performed and to provide preliminary foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the Advanced Planning Study dated January 15, 2020, review of As-built plans, previous geotechnical reports, BIRIS records, and a site visit.

2.2 Project Description

Describe the existing and/or proposed structure(s), and pertinent project information relating to the planned improvements. Provide project vertical datum reference.

Example: New Bridge

The bridge site is in the city of San Diego on State Route 15 at PM R3.8 which crosses over Interstate 805 (I-805) at PM 15.1. At this site, the proposed bridge replacement is necessary to accommodate the underlying highway improvements, which include the widening of the existing I-805 in order to provide additional High-Occupancy Vehicle (HOV) lanes. Based on the General Plan (dated January 15, 2020), the proposed bridge is a 2-span, cast-in-place, prestressed concrete box girder bridge supported on pile foundations.

All elevations referenced within this report are based on the National Geodetic Vertical Datum of 1929 (NGVD 29), unless otherwise noted.

Example: Bridge Widening

The Sweetwater River left and right bridges are located on I-805 in Chula Vista, and are two of several bridges along I-805 which are in the process of being widened to accommodate construction of High Occupancy Vehicle (HOV) lanes in the median. "As-built" information indicates that the existing Sweetwater River left and right bridges consist of five-span, cast-in-place, pre-stressed concrete, box-girder structures, with end-diaphragm abutments that were constructed in 1968. The existing bridges are supported on driven Class 70C concrete piles at all support locations. The proposed work includes median widening between the left and right bridges and removal and replacement/widening of a portion of the deck of the right bridge. The center widening is proposed to consist of a six-span, cast-in-place, pre-stressed concrete, box-girder structure, with end-diaphragm abutments.

The 1968 As-built plans did not include a vertical datum reference. It is assumed that the elevations are based on the National Geodetic Vertical Datum of 1929 (NGVD 29), however it is recommended that structure design verify this assumption.

2.3 Exceptions to Policies and Procedures

List exceptions to Departmental policies and procedures relating to the SPGR. Approved *Request for Exception* forms must be included in the Appendix. Omit this section if there are no exceptions.

2.4 Geotechnical Investigation

Provide an overview of the geotechnical investigation(s) that support the preliminary foundation recommendations.

Example

The As-built LOTB show that a subsurface investigation, consisting of three mud rotary borings, was performed in 1969. Additionally, a site visit was performed on February 23, 2020 to review site access and creek conditions. During the site visit the creek was flowing between Piers 2 and 3 with a water depth of approximately 2 feet.

2.5 Geotechnical Conditions

2.5.1 Geology

Identify the pertinent geologic map and the geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they influence the design and construction of the structure.

Example

The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium.

2.5.2 Surface Conditions

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

Example

The topography is relatively flat and the site appears free of geologic hazards.

2.5.3 Subsurface Conditions

Provide a generalized description of the known subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials)

Do not re-create an As-built LOTB in detail in this section. A generalized discussion or table is sufficient.

Example

Based on the 1966 As-built Log of Test Borings, the alluvial soil at the site can be separated into three general units. The upper unit consists of very loose to slightly compact silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of slightly compact to dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

2.6 Groundwater

Report groundwater elevation(s) and dates of measurements. Use of a table is recommended if there are numerous borings and/or measurements.

Table X: Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Date Measured

Example: Groundwater Present

During the 1998 subsurface investigation, groundwater was encountered in both borings. Groundwater levels varied from elevation 945 feet (depth of 20 feet) in February to elevation 938 feet (depth of 27 feet) in August.

Example: Groundwater Not Present

During the 1998 subsurface investigation, groundwater was not encountered in either boring within the explored depth of 100 feet (~ Elev. 900 feet).

Example: Groundwater Information Not Available

Groundwater information was not available based upon the literature search performed.

Example: Groundwater Information Available Nearby

Groundwater measurements available from a DWR monitoring well, located 800 feet northwest of the proposed structure, had groundwater elevations that varied between 930 feet and 920 feet from 2015 to present.

2.7 As-built Data

Include brief discussion of relevant As-built foundation data, such as:

- Existing foundation types and details (e.g., pile tip elevations)
- As-built geotechnical capacities or resistances.
- Construction reports or records such as pile driving logs, pile load test reports, construction difficulties, etc.

Use the tables in the examples below to present foundation data.

Example: Driven Piles

Construction of the original bridge was completed in 1971 with all three supports supported on driven Alternative (Alt) "X" concrete piles. The 1971 As-built LOTB provided the data in Table 1.

Table 1: Summary of the 1971 As-built Data

Support Location	*Foundation Type*	*Design Load*	*Bottom of Pile Cap Elev. (feet)*	*Min. Penetration Elev. (feet)*	*Avg. Penetration Elev. (feet)*	*Max Penetration Elev. (feet)*
Abutment 1	*12" Driven Alt "X" Pile*	*45 ton*	*958.1*	*929.3*	*927.1*	*926.5*
Bent 2	*12" Driven Alt "X" Pile*	*45 ton*	*935.5*	*920.2*	*918.1*	*916.1*
Abutment 3	*12" Driven Alt "X" Pile*	*45 ton*	*953.6*	*928.1*	*926.5*	*925.3*

Example: Shallow Foundations

The existing Cenda Ditch Bridge consists of a two-span, cast-in-place, slab bridge that was constructed in 1963. Abutments 1 and 3 are end-diaphragm abutments and are supported on spread footings placed in approximately 20 feet of embankment fill material. Pier 2 is supported by seven columns on spread footings founded on rock. The As-built bottom of footing elevations and design loads for the bridge are shown below in Table 2.

Table 2: As-built Information

Location	*As-built Bottom of Footing Elevation (feet)*	*As-built Allowable Footing Pressure (tsf)*	*As-built Design Footing Pressure (tsf)*
Abutment 1	*4499.1*	*2.0*	*2.0*
Pier 2	*4475.0*	*5.0*	*5.0*
Abutment 3	*4490.7*	*2.0*	*2.0*

2.8 Scour Data

Report pertinent scour information obtained from the geotechnical investigations (e.g., BIRIS records, Preliminary Hydraulics Report) including the potential for scour. Use the table in the example to present scour data.

Example: Scour Data Available

The bridge site is underlain by alluvial soil, which is considered potentially scourable. The Structure Hydraulics Branch provided the following scour information in a Preliminary Hydraulics Report dated May 15, 2020 (Table 1).

Table 1: Scour Data

Support Location		*Long Term Scour (Degradation and Contraction) Elevation (feet)*	*Short Term Scour (Local) Depth (feet)*
Left Bridge	*Abut 1*	*2285.6*	*3*
	Abut 2	*2285.1*	*3*
Right Bridge	*Abut 1*	*2291.9*	*3*
	Abut 2	*2291.6*	*3*

Example: No Scour

The bridge does not span a watercourse, therefore there is no scour potential.

Example: Scour Data Unavailable

The bridge spans a watercourse. BIRIS records do not identify any historic scour issues. The Structures Hydraulics Branch has not yet issued a Preliminary Hydraulics Report.

2.9 Corrosion Evaluation

Report and discuss pertinent site corrosion data.

Example: No information available

Historical corrosion data is not available. For preliminary design purposes the site should be considered non-corrosive based on the presence of predominantly cohesionless soil. Corrosion samples will be obtained during the design phase to evaluate the corrosion potential of the site.

Example: Non-Corrosive

Three soil samples and one water sample were collected for corrosion testing during the 2011 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on Caltrans' standards, the site is considered non-corrosive.

Example: Corrosive

During the 2011 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-11-001 and RC-11-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on Caltrans' standards, and corrosion mitigation may be required depending on the type/depth of foundation selected.

Table 1: Soil Corrosion Test Summary

Boring ID	*Elevation (feet)*	*Minimum Resistivity (Ohm-Cm)*	*pH*	*Chloride Content (ppm)*	*Sulfate Content (ppm)*	*Corrosive?*
RC-11-001	*15.8 to 14.3*	*1544*	*7.24*	*N/A*	*N/A*	*No*
RC-11-001	*-4.2 to -3.2*	*683*	*7.94*	*384*	*432*	*No*
RC-11-002	*-69.1 to -70.6*	*73*	*6.86*	*850*	*1500*	*Yes*
RC-11-002	*-104.1 to -105.6*	*78*	*7.71*	*1000*	*1600*	*Yes*

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

2.10 Seismic Information

Report all information required in Section 2.10.1 in the SPGR. Referencing a Seismic Report that was delivered separately is not acceptable. Information required in Section 2.10.2 should be summarized while referencing the reader to the applicable report (e.g., Fault Rupture Report).

2.10.1 Ground Motion Hazard

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)
b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., CPT or SPT correlations). See *Design Acceleration Response Spectrum (ARS)* module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
    i. The ARS per the *Design Acceleration Response Spectrum* module.
    ii. The horizontal peak ground acceleration (HPGA).
    iii. Deaggregated mean earthquake moment magnitude (M) for the HPGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. Ground Motion Data Sheet (see *Forms and Templates*)
e. Soil Profile Classification (Class S1 or Class S2) for the site or support locations per the SDC v2.0, Section 6.1 and Section 6.2.3. Omit if sufficient site data is not available.

Example

The site is susceptible to strong earthquake induced ground motions during the design life of the bridge.

Based on available subsurface information and Standard Penetration Test (SPT) correlations for determining shear wave velocity, the time-average shear wave velocity (V_{S30}) for the upper 100 feet of soil at the site is estimated to be 980 ft/sec.

The Design Spectrum for the Safety Evaluation Earthquake, as specified in Caltrans Seismic Design Criteria with October 2019 interim revisions, Version 2.0 (SDC v2.0), is the probabilistic response spectrum representing the horizontal ground motion at the site with a

5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the Design Spectrum in the form of the design Acceleration Response Spectrum (ARS).

Caltrans web-based tool ARS Online v3.0 was utilized to determine the design ground motion parameters, including the ARS, for the subject structure site. Based on the ARS Online v3.0 tool, the design PGA = 0.27g, and the deaggregated mean earthquake moment magnitude for PGA is M = 6.7 and mean site-to-source distance for 1.0 second period spectral acceleration is R = 41.2 miles.

The Ground Motion Data Sheet, presenting the design ARS data, plot, and other relevant information, is attached.

The soil at the site is "Class S1" per Section 6.1 and 6.2.3 of the SDC, v2.0.

2.10.2 Other Seismic Hazards

The section must include information on the following seismic hazards, as applicable at the site:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Seismically induced total and differential ground settlements
d. Lateral spreading potential (see *Lateral Spreading* module)
e. Seismic slope instability
f. Tsunami risk

Example

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

Groundwater was not encountered within the As-built borings drilled to depths ranging from 70 to 100 feet (~ Elev. 90 to 60 feet) from the existing ground surface. Dense and/or stiff soils were encountered in these borings below a depth of about 60 feet (~Elev. 100 feet) from the existing ground surface. Based on these groundwater and subsurface soil conditions, the project site is not susceptible to liquefaction or related seismic hazards, including seismic total or differential ground settlement, seismic downdrag and lateral spreading.

The project site and the adjacent areas are relatively flat. The existing abutment and approach embankment slopes consist of dense and stiff compacted fill soil. Based on these soil conditions and the absence of soil liquefaction potential, the existing fill slopes at the site are not considered subject to instability during the design seismic ground motion event.

The site is located more than 0.5 miles from the nearest coastline and is situated above elevation 40 feet, therefore the risk for tsunami does not exist (per MTD 20-13).

2.11 Geotechnical Recommendations

Recommendations must include discussion of the appropriateness of shallow foundations, driven pile foundations, and CIDH concrete pile foundations. Recommendations must be presented in the order of preference with the recommended foundation type(s) presented first; followed by feasible, but not preferred, alternatives; followed by foundation types not recommended. If applicable, include commentary relating to foundation types proposed by the Structure Designer (MTD 3-1, Table 3-2).

Example

The following is a discussion of the foundation system alternatives. This discussion is based upon an understanding of the regional geology and the observations of the subsurface conditions from the 1990 field investigation and construction of the existing bridge in 1992.

- *Driven Displacement Piles: Driven displacement piles such as Standard Plan precast prestressed concrete piles or closed end pipe piles are recommended for support of the new structure.*
- *Driven Non-Displacement Piles: Driven non-displacement open-ended pipe piles or H-piles are feasible for foundation support, however installed pile lengths are expected to be variable and difficult to predict in these subsurface conditions, particularly for the H-pile alternative. Driven displacement piles are preferable to driven H-piles.*
- *Cast-in-Steel-Shell (CISS) Concrete Piles: CISS piles are feasible for foundation support, however installed pile lengths are expected to be variable and difficult to predict.*
- *Large Diameter Drilled Shafts (CIDH Concrete Piles): Large diameter drilled shafts, those with diameters greater than 24 inches, are not recommended for support. Saturated granular foundation soils exist at this location. Caving and flowing soils are expected, and "wet" construction methods would be required.*
- *Small Diameter Drilled Shafts (CIDH Concrete Piles): Small diameter drilled shafts, those with diameters of 24 inches and less, are not recommended for support. Saturated granular foundation soils exist at this location. Caving and flowing soils are expected, and "wet" construction methods would be required.*
- *Spread Footings: The foundation conditions are not suitable for spread footings because of the presence of loose material in the upper 15 feet.*

2.12 Additional Field Work and Laboratory Testing

Describe the anticipated scope and types of fieldwork and testing that may be required to complete the geotechnical investigation. Discuss the potential need for entry permits, task orders, groundwater monitoring, access road construction, lane closures, etc.

Example

The available site information will not provide adequate data to complete the design recommendations for Dry Creek Bridge. Therefore, a field investigation consisting of borings, seismic CPT, and laboratory testing will be performed to characterize the site.

The District Project Manager must initiate the process of obtaining drilling clearances (e.g., environmental permits, right of entry, categorical exemptions, etc.) so that drilling, preliminary design, and the Preliminary Foundation Report can be completed prior to the end of PA&ED. For foundation investigation details, the District Project Manager may contact the Office of Geotechnical Design X.

Any questions regarding the above recommendations should be directed to the attention of NAME and PHONE.

2.13 Report Copy List

The SPGR must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* module.

2.14 Appendices

Appendix I: Ground Motion Data Sheet

3 PRELIMINARY FOUNDATION REPORT (PFR) and FOUNDATION REPORT (FR)

The PFR is prepared after completion of the SPGR and Advanced Planning Study, and prior to the Structure Type Selection. The number, location, types of foundations and associated loads will be better defined, and the site investigation may be complete. The amount of information provided in the PFR will be relative to the information provided by Structure Design and the extent of geotechnical investigation completed.

The FR expands on data provided in the PFR and updates the foundation recommendations based upon final loads provided by Structure Design. The FR becomes part of the contract documents via its inclusion in the Information Handout per Standard Special Provision 2-1.06B, "Supplemental Project Information."

The following topics, if applicable, must be addressed in the Preliminary Foundation Report and Foundation Report.

3.1 Introduction

Summarize the scope and types of work performed to obtain the information supporting the foundation recommendations.

Example: Preliminary Foundation Report with 0-Phase Drilling

Per the request dated May 3, 2020, this Preliminary Foundation Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the investigations performed and to provide preliminary foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the draft general plan dated January 15, 2020, a subsurface investigation consisting of borings at the abutments, and preliminary loads and scour information provided by Structure Design. Borings were not completed at Bent 2 because access permits were not yet available.

Example: Foundation Report

Per the request dated May 3, 2020, this Foundation Report has been prepared for the proposed widening of Dry Creek Bridge. The purpose of this report is to summarize the investigations performed and to provide foundation recommendations for Dry Creek Bridge. The recommendations presented in this report are based on the general and foundation plans dated January 15, 2020, a subsurface investigation, and loads and scour information provided by Structure Design.

3.2 Project Description

Describe the existing and/or proposed structure(s), and pertinent project information relating to the planned foundation improvements. Provide project vertical datum reference.

Example

The bridge site is located in the city of San Diego on State Route 15 at PM R3.8, which crosses over Interstate 805 (I-805) at PM 15.1. At this site, the proposed bridge replacement is necessary to accommodate the underlying highway improvements, which

include the widening of the existing I-805 in order to provide additional High-Occupancy Vehicle (HOV) lanes. All elevations referenced within this report are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted. To convert an elevation at this site from National Geodetic Vertical Datum of 1929 (NGVD 29) to NAVD 88, add 2.3 feet to the NGVD 29 elevation.

Based on the General Plan (dated January 15, 2020), the proposed bridge is a 2-span, cast-in-place, prestressed concrete box girder structure supported on pile foundations.

3.3 Exceptions to Policies and Procedures

Discuss exceptions to Departmental policies and procedures relating to the PFR/FR. Approved *Request for Exception* forms must be included in the Appendix. Omit this section if there are no exceptions.

3.4 Geotechnical Investigation

Provide an overview of the geotechnical investigation(s) performed to support the geotechnical recommendations including the number of boreholes/CPT soundings with maximum depth(s) and elevation(s), and the types of field and/or downhole testing (e.g., in-situ, geophysical).

Example

The Geotechnical Investigation included a review of the as-built borings from the 1966 investigation and drilling three borings in June 2020. The 1966 foundation investigation consisted of one 3-inch mud rotary boring and eight 1-inch driven soil tube borings. In June 2020, three mud rotary borings were drilled to a maximum depth of 80 feet (~ Elev. 230 feet) using a CS2000 drill rig. The Standard Penetration Test (SPT) was performed at regular intervals to evaluate the engineering properties of the earth materials. The type(s) and location(s) of field testing are shown on the LOTB sheets.

3.5 Laboratory Testing Program

Provide an overview of the laboratory testing program, if performed, to support the geotechnical recommendations. Briefly explain what the tests were used for (e.g. soil classification, settlement, strength parameters).

Example

During the June 2020 field investigation, soil samples were collected from borings RC-20-001 and RC-20-002 for soil classification and corrosion evaluation (Particle Size Analysis, Plasticity Index, Corrosion Testing). A summary of the test results is provided in the Appendix, and the test sample locations are shown on the Log of Test Borings.

3.6 Geotechnical Conditions

3.6.1 Geology

Identify the pertinent geologic map and the prominent geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they may influence the design and construction of the structure.

Example

The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium.

3.6.2 Surface Conditions

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

Example

The topography is relatively flat and no geologic hazards have been identified.

3.6.3 Subsurface Conditions

Provide a generalized description of the subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials)

Do not re-create the LOTB(s) in detail in this section. A generalized discussion or table is sufficient.

Example

Based on the 2020 site investigation, the alluvial soil at the site can generally be separated into three units. The upper unit consists of very loose to medium dense silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum explored depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

3.7 Groundwater

Report groundwater elevation(s) and dates of measurements. Use of the following table is recommended if there are numerous borings and/or measurements. Discuss surface water conditions that might influence the design or construction of the foundations. State the groundwater elevation(s) (e.g., liquefaction, pile design) used for analyses and design.

Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation (feet)	Depth to Groundwater (feet)	Groundwater Elevation (feet)	Date Measured

Example

As-built LOTB's from the April 1968 subsurface investigation indicate that groundwater was encountered in several borings at that time, and ranged from elevation 19.0 feet to elevation 21.2 feet (NAVD88 datum). During the 2020 subsurface investigation groundwater was measured in Boring RC-20-001 at elevation 15.3 feet, and in Boring RC-20-002 at elevation 13.9 feet. During the 2020 subsurface investigation, groundwater was measured in boring RC-20-003 at elevation 17.1 feet, which corresponded to the level of the water in the riverbed at that time. The groundwater elevation used for design was 21 feet.

3.8 As-built Data

Include brief discussion of relevant As-built data, such as:

- Existing foundation types and details.
- As-built geotechnical capacities or resistances.
- Construction reports or records such as pile driving logs, pile load test reports, construction difficulties, etc.

Use the tables in the examples below to present foundation data.

Example: Driven Piles

Construction of the original bridge was completed in 1971 with all three supports supported on driven Alternative (Alt) "X" concrete piles with design loads of 45 tons. The 1971 As-built LOTB provided pile driving information, which included the minimum, average, and maximum penetration elevations for the piles. The bottom of pile cap elevations listed were obtained from the As-built foundation plan. Table 1 presents a summary of the 1971 As-built Data.

Table 1: Summary of the 1971 As-built Data

Support Location	*Foundation Type*	*Design Load*	*Bottom of Pile Cap Elev. (feet)*	*Min. Penetration Elev. (feet)*	*Avg. Penetration Elev. (feet)*	*Max Penetration Elev. (feet)*
Abutment 1	*12" Driven Alt "X" Pile*	*45 ton*	*958.1*	*929.3*	*927.1*	*926.5*
Bent 2	*12" Driven Alt "X" Pile*	*45 ton*	*935.5*	*920.2*	*918.1*	*916.1*
Abutment 3	*12" Driven Alt "X" Pile*	*45 ton*	*953.6*	*928.1*	*926.5*	*925.3*

Example: Shallow Foundations

The existing Cenda Ditch Bridge consists of a two-span, cast-in-place, slab bridge that was constructed in 1963. Abutments 1 and 3 are end-diaphragm abutments and are supported on spread footings placed in approximately 20 feet of embankment fill material. Pier 2 is supported on seven columns, each with a spread footing founded on rock. The As-built bottom of footing elevations and design loads for the bridge are shown below in Table 2.

Table 2: As-built Information

Location	*As-built Bottom of Footing Elevation (feet)*	*As-built Allowable Footing Pressure (tsf)*	*As-built Design Footing Pressure (tsf)*
Abutment 1	*4499.1*	*2.0*	*2.0*
Pier 2	*4475.0*	*5.0*	*5.0*
Abutment 3	*4490.7*	*2.0*	*2.0*

3.9 Scour Data

Report pertinent scour information obtained from the geotechnical investigations (e.g., BIRIS records, Hydraulics Report) including the potential for scour. Use the table in the example to present scour data from the Hydraulics Report.

If the field investigation reveals geologic information that contradicts the hydraulics report, then the Geoprofessional must discuss the findings in the PFR/FR and provide that information to the author of the hydraulics report so that the scour recommendations can be re-evaluated.

Example: Scour Data Available

The bridge site is underlain by alluvial soil, which are considered potentially scourable. The Structure Hydraulics Branch provided the following scour information in a report dated January 15, 2020 (Table 1).

Table 1: Scour Data

Support Location		*Long Term Scour (Degradation and Contraction) Elevation (feet)*	*Short Term Scour (Local) Depth (feet)*
Left Bridge	*Abut 1*	*2285.6*	*3*
	Abut 2	*2285.1*	*3*
Ruight Bridge	*Abut 1*	*2291.9*	*3*
	Abut 2	*2291.6*	*3*

Example: No Scour

The bridge does not span a watercourse, therefore there is no scour potential.

Example: Scour Data Unavailable

The bridge spans a watercourse. BIRIS records do not identify historic scour issues. The Structures Hydraulics Branch has not yet provided a Hydraulics Report to this Office.

3.10 Corrosion Evaluation

Include and update the corrosion data from the SPGR based on new findings and field investigations. If corrosion testing was not completed during the foundation investigation, provide justification for the corrosion recommendations.

Example: Non-Corrosive

Three soil samples and one water sample were collected for corrosion testing during the 2020 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on Caltrans' standards, the site is considered non-corrosive.

Example: Corrosive

During the 2020 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-20-001 and RC-20-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on Caltrans' standards, and corrosion mitigation is required.

Table 1: Soil Corrosion Test Summary

Boring ID	*Elevation (feet)*	*Minimum Resistivity (Ohm-Cm)*	*pH*	*Chloride Content (ppm)*	*Sulfate Content (ppm)*	*Corrosive?*
RC-11-001	*15.8 to 14.3*	*1544*	*7.24*	*N/A*	*N/A*	*No*
RC-11-001	*-4.2 to -3.2*	*683*	*7.94*	*384*	*432*	*No*
RC-11-002	*-69.1 to -70.6*	*73*	*6.86*	*850*	*1500*	*Yes*
RC-11-002	*-104.1 to -105.6*	*78*	*7.71*	*1000*	*1600*	*Yes*

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

3.11 Seismic Information

Update the seismic information required for the SPGR based on new findings and/or investigations. Summarize analyses and evaluations performed, and recommendations relating to seismic design.

3.11.1 Ground Motion Hazard

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)
b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., geophysics, seismic CPT or SPT correlations). For sites with a low V_{S30}, a site-specific ground response analysis may be required to determine the final design ARS. See the *Design Acceleration Response Spectrum* module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
    i. The ARS per the *Design Acceleration Response Spectrum* module.
    ii. The horizontal peak ground acceleration (HPGA).
    iii. Deaggregated mean earthquake moment magnitude (M) for the HPGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. Ground Motion Data Sheet (see *Forms and Templates*)
e. Soil Profile Classification (Class S1 or Class S2) for the site or support locations per the SDC, Section 6.1 and Section 6.2.3. For the PFR, omit if sufficient site data is not available.
f. <u>If requested:</u> provide site data and support for the development of ground motion time-histories for bridge project sites (typically for sites with low V_{S30}).
g. <u>If requested:</u> include the design ARS for the Functional Evaluation Earthquake with a Return Period = 225 years (see *Design Acceleration Response Spectrum* module).

<u>Example</u>

The site is susceptible to strong earthquake induced ground motions during the design life of the bridge.

Based on information obtained during the recent site exploration and seismic Cone Penetration Test for determining shear wave velocity, the time-average shear wave velocity (VS30) for the upper 100 feet of soil at the site is estimated to be 850 ft/sec.

The Design Spectrum for the Safety Evaluation Earthquake, as specified in Caltrans Seismic Design Criteria with October 2019 interim revisions, version 2.0 (SDC v2.0) is the probabilistic response spectrum representing the horizontal ground motion at the site with a 5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the Design Spectrum in the form of the design Acceleration Response Spectrum (ARS).

Caltrans web-based tool ARS Online v3.0 was utilized to determine the design ground motion parameters, including the ARS, for the subject structure site. Based on the ARS Online v3.0 tool, the design PGA = 0.27g, and the deaggregated mean earthquake moment magnitude for PGA, M = 6.7 and the mean site-to-source distance for 1.0 second period spectral acceleration, R = 41.2 miles.

The Design Ground Motion Data Sheet, presenting the design ARS data, plot, and other relevant information, is attached.

The soil at the site is "Class S1" per the Sections 6.1 and 6.2.3 of the SDC, v2.0.

3.11.2 Other Seismic Hazards

The section must include information on the following seismic hazards:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Effects of Liquefaction, including
    i. Seismically-induced ground surface settlements at each support location
    ii. Downdrag at each support location with pile foundations (see *Downdrag* module)
    iii. Lateral spreading potential (see *Lateral Spreading* module)
d. Seismic slope stability
e. Tsunami risk (if applicable)

Discuss the findings and results of other seismic-design analyses, all applicable and necessary geotechnical seismic design recommendations (e.g., residual shear strengths for liquefied soil layers, seismic downdrag, lateral spreading loads/displacements, nominal bearing resistances of foundations with and without considering liquefaction for seismic retrofit projects, seismic lateral earth pressures, liquefaction mitigation measures, etc.).

Example: No Hazards

The site has been determined not to have potential for surface fault rupture, liquefaction, seismic-induced slope failure, or tsunami.

Example: No Surface Fault Rupture

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

Example: Surface Fault Rupture

The structure is located within an Alquist-Priolo Earthquake Fault Zone. Therefore, per MTD 20-10, the structure is susceptible to surface fault rupture hazards. Per the attached Fault Rupture Report dated March 15, 2020, the horizontal displacement is estimated to be one foot and the vertical displacement is estimated to be 4 inches.

Example: Liquefaction

Due to the presence of loose to medium dense alluvial material and shallow ground water beneath the site, the potential for soil liquefaction is present at the site. Liquefiable zone elevations at the abutment and pier locations are summarized in Table 1.

Example: Effects of Liquefaction

Liquefaction-induced settlement of the ground surface and pile downdrag are anticipated and summarized in Table 1. Implications of liquefaction on the pile tip elevations will be addressed in the Geotechnical Recommendations section.

Table 1: Liquefaction Potential at Old River Bridge

Support	Liquefaction Elevation (feet)	Estimated Seismic-induced Settlement (inches)	Downdrag Zone Bottom Elevation (feet)	Estimated Downdrag Load (kips/pile)
Abutment 1	Elev. 20 to 15 Elev. 0 to -10	3	-5	150
Pier 2	Elev. 10 to -5	4	-3	50
Abutment 3	Elev. 20 to 10	3	12	100

Note: Downdrag loads calculated for 24-inch CIDH concrete piles at the Abutments and 60-inch CISS at Pier 2.

Example: Lateral Spreading Potential

Due to the presence of liquefiable soils at shallow depths and relatively high design horizontal peak ground acceleration, an initial lateral spreading hazard assessment was performed at each abutment by ignoring all lateral resistance contributions from the foundation piles. The analysis was performed in accordance with Steps 1 through 4 of the Lateral Spreading Analysis Example of the Geotechnical Manual. Results of the analysis indicate a lateral spreading hazard potential at both abutments.

Additional lateral spreading analyses were performed for each abutment in accordance with MTD 20-15 and the Lateral Spreading Analysis Example module of the Geotechnical Manual. The pile restraining force versus displacement plots (MTD 20-15 Figure 5, Curve 3) developed for the two abutments are shown in Figures X and Y.

Due to the relatively flat ground surface conditions, lateral spreading potential does not exist at Pier 2.

Example: Seismic Slope Stability

Seismic slope stability analyses were performed to evaluate the overall stability at the proposed abutment slopes. The pseudo static analysis was performed with a horizontal seismic coefficient (k_h) equal to 0.15g.

Two-dimensional slope stability analyses were performed and the results are included in the appendix. The analyses found the minimum value of factor of safety at the proposed abutment slopes to be approximately 1.25 (resistance factor = 0.8), which meets the accepted minimums for stable abutment slopes (per AASHTO LRFD).

Example: No Tsunami Risk

The site is located about 0.25 miles from the nearest coastline. However, the ground surface elevation at the bridge location ranges from 100 to 120 feet above mean sea level. The site is not located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the X County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard does not exist at the site.

Example: Tsunami Risk

The site is located about 0.25 miles from the nearest coastline and the ground surface elevation at the bridge location ranges from 10 to 50 feet above mean sea level. The site is located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the X County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard exists at the site.

3.12 Geotechnical Recommendations

Provide complete and concise recommendations for bridge foundations by addressing the topics in the applicable portions (i.e., Shallow Foundations, Driven Pile Foundations, and/or CIDH Concrete Pile Foundations) of this section. Include recommendations for earth retaining structures connected to the bridge, or located within 150 feet of the bridge. Refer to *Foundation Reports for Earth Retaining Systems* for reporting requirements.

Present and/or discuss the following:

1. Identify all structures addressed in this section
2. Date of plans used for analysis (e.g., General Plan, Foundation Plan, Retaining Wall Layout Sheets)

Example

The following recommendations are for the proposed Dry Creek Bridge (Br. No. 54-1200) and associated wing walls as shown on the General Plan dated May 14, 2020, and a Foundation Plan received via electronic mail on June 28, 2020.

3.12.1 Shallow Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence foundation type selection, analysis, and design (e.g., scour, groundwater, ground improvement).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Foundation Data table (after MTD 4-1, Attachment 4, Table 1), from the Structure Designer
2. A description of the material on which the footing is to be placed
3. A description of the ground line conditions (e.g., flat, sloped)
4. Preliminary Foundation Data Tables
   a. End Supports (Abutments) table (after MTD 4-1 Attachment 2, Table 1)
   b. Intermediate Supports (Bents and Piers) table (after MTD 4-1 Attachment 2, Table 2)

Table X: Foundation Data

Support Location	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Abutment 1					
Bent 2					
Abutment 3					

Table X: Preliminary Foundation Data for Abutments

End Supports (Abutments)

Support Location:
Foundation Material (Soil or Rock)[1]:
Friction Angle or Undrained Shear Strength for Sliding:
Permissible Settlement (in):
Resistance Factor (Strength) – φ_b:
Resistance Factor (Seismic) – φ_b:

	Total Number of B' =			
No	Effective Footing Width	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B' (feet)	q_n (ksf)	q_{pn} (ksf)	q_R (ksf)
1				
2				
3				
4				
5				

1. Select “Soil” or “Rock” depending on design methodology used.
2. Based on L’ =____ ft.

Table X: Preliminary Foundation Data for Bents and Piers

Intermediate Supports (Bents and Piers)

Support Location:
Foundation Material (Soil or Rock)[1]:
Friction Angle or Undrained Shear Strength for Sliding:
Permissible Settlement (in):
Resistance Factor (Strength) – φ_b:
Resistance Factor (Seismic) –φ_b:

Total Number of unique *L′/B′* Ratios[2]	
Total Number of *B′*s per *L′/B′* Ratios[3]	

No	Effective Footing Width	Effective Footing Size Ratio	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B′ (ft)	*L′ / B′*	*qn* (ksf)	*qpn* (ksf)	*qR* (ksf)
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					
1					
2					
3					
4					
5					

1. Select “soil” or “rock” depending on design methodology used.
2. Indicates total number of curves used to show variations of q_n, q_{pn}, and q_R vs. *B’*.
3. Indicates total number of points on each curve to show variations of q_n, q_{pn}, and q_R vs. *B’*.

Example: Shallow Foundations (PFR)

At Abutments 1 and 2 support locations, spread footings are recommended. The subsurface information gathered for the site indicate that the abutment footings will be founded on dense sand. The foundation recommendations were designed in accordance with the AASHTO LRFD Bridge Design Specification with CA Amendments. The spread footings are located in close proximity to a descending slope and were designed as "footing on a slope." The following Foundation Geotechnical Data Tables provide preliminary recommendations for all support locations.

Present the following in the Foundation Report:

1. Information from the Structure Designer
    a. Foundation Data table (after MTD 4-1, Attachment 4, Table 1)
    b. Summary of Controlling Loads table (after MTD 4-1, Attachment 5, Table 1)
2. A description of the material on which the footing is to be placed.
3. A description of the ground line conditions (e.g., flat, sloped)
4. Foundation Design Recommendations for Spread Footing table (after MTD 4-1, Attachment 5, Table 2).
5. Spread Footing Data Table (after MTD 4-1, Attachment 5, Table 3).
6. Calculated resistance factor for overall global stability and local slope stability of the foundation (Service Limit State and Extreme Event Limit State).

If applicable, present the following additional items in the Foundation Report:

7. If spread footings are to be constructed below groundwater level, identify the type of excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).
8. If unsuitable native soil underlies the proposed footing, specify sub-excavation and replacement with structure backfill.
9. Discussion of the influence of the new footing on the adjacent structures and/or utilities.

Table X: Foundation Data

Support Location	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Abutment 1					
Bent 2					
Abutment 3					

Table X: Summary of Controlling Loads

Support Location	L (feet)	B (feet)	Controlling Loads						
			M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{total} (kips)	P_{perm} (kips)	*Load Combination*
Abutment 1				N/A	N/A				
Bent 2									
Abutment 3				N/A	N/A				

Table X: Foundation Design Recommendations for Spread Footing

Support Location	Footing Size (feet)		Bottom of Footing Elevation (feet)	Minimum Footing Embedment Depth (feet)	Total Permissible Support Settlement (inches)	Service Limit State	Strength Limit State (φ_b=)	Extreme Event Limit State (φ_b=1.0)
	B	L				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Abutment 1						(B' =)	(B' =)	N/A
Bent 2						(B' =)	(B' =)	(B' =)
Abutment 3						(B' =)	(B' =)	N/A

For each contact stress and bearing resistance in the table, include the associated effective footing width (B') in parentheses. See *Shallow Foundations* module for example.

Table X: Spread Footing Data Table

Support Location	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (φ_b=) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (φ_b=1.0) (ksf)
Abutment 1			N/A
Bent 2			
Abutment 3			N/A

Example: Shallow Foundations (FR)

At Abutments 1 and 4 support locations, spread footings are recommended. The foundation recommendations are based on the information provided by Structure Design in the following tables and were designed in accordance with the AASHTO LRFD Bridge Design Specification with CA Amendments. The spread footings are in close proximity to a 1.5:1 descending slope and were designed as "footing on a slope." The following Foundation Design Recommendations table and Spread Footing Data Table provide final recommendations for Abutments 1 and 4.

Groundwater will be encountered during construction of the footings at the proposed abutments, therefore show Structure Excavation Type D on the plans.

The subsurface information gathered for the site indicate that the Abutment 1 footing will be founded in sedimentary rock formation. At Abutment 4, unsuitable native soils underlie the proposed footings. It is recommended that the native materials be removed to a depth of 5 feet (Elev. 15 feet) below the bottom of footing, and be replaced with structure backfill or concrete to the bottom of footing elevation. The bottom of sub-excavation elevations for the abutments are listed in Table 1. The limits of the sub-excavation and replacement shown on the plans must conform to the limits specified in Standard Specification 19-5.03B for compaction of embankments under retaining wall footings without pile foundations.

Table 1: Bottom of Sub-Excavation Elevation

Support Location	*Bottom of Sub-Excavation Elevation (feet)*
Abutment 4	*15*

Global stability was analyzed at Abutment 4. The calculated resistance factors for global stability were found to satisfy current requirements for both the Service-I Limit State (static), as well as the Extreme Event Limit State (pseudo-static). Table 2 summarizes the global stability analysis results. Abutment 1 was not analyzed because it is founded on sedimentary rock formation.

Table 2: Global Stability Analysis Summary (Abutment 4)

Service Limit State	*Calculated Factor of Safety*	*Calculated Resistance Factor*	*2017 AASHTO LRFD Resistance Factor Requirement*
Service-I Limit State (Static)	*1.5*	*0.65*	*≤0.65*
Extreme Event Limit State (Pseudo-Static)	*1.11*	*0.89*	*≤0.90*

3.12.2 Deep Foundations

3.12.2.1 Driven Pile Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence type selection, analysis, and design (e.g., scour, liquefaction, lateral spreading, ground water).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Information provided by Structure Designer
    a. Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-2)
2. Preliminary Foundation Design Recommendations table, that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} and ϕ_{qp} = 0.7 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.3-1.
    b. Round the Required Nominal Resistance up to the nearest 10 kips.

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Table X: Preliminary Foundation Design Recommendations

Support Location	Pile Type	Cutoff Elevation (feet)	Required Nominal Resistance (Strength Limit State) (kips)		Preliminary Tip Elevation (feet)
			Compression (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	
Abutment 1					
Pier 2					
Abutment 3					

Add note if applicable:

- Cutoff elevations not provided by Structure Design and are estimated by Geotechnical Services

Present the following in the Foundation Report:

1. Information provided by Structure Designer
    a. Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-4)
    b. Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5)
2. Foundation Design Recommendations table (after MTD 3-1, Attachment 1, Table 3-6), that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} and ϕ_{qp} = 0.7 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.3-1.
    b. Round the Required Nominal Resistance (Strength and Extreme Limit State) and Factored Design Loads (Service Limit State) to the nearest 10 kips.
3. Pile Data Table (MTD 3-1, Attachment 1, Table 3-7)
4. The Required Nominal Driving Resistance is the side resistance of all penetrated soil, including scourable, downdrag, and liquefiable layers, plus the tip resistance. Show this value in the Foundation Design Recommendations table and the Pile Data Table.

If applicable, present the following:

5. Present the wall thickness for driven steel shells or cast-in-steel-shell piles in the Foundation Design Recommendation and Pile Data tables.
6. If the Required Nominal Resistance does not equal the Required Nominal Driving Resistance (e.g., scour susceptible layer, liquefiable layer), explain why.
7. For projects where a pile drivability study has been performed during the design phase (for pipe piles and steel shells), provide the minimum pile wall thickness determined by the pile drivability study.
8. If a Standard Plan pile or steel H-pile requires modification (e.g., increased wall thickness, adding driving tips or lugs), provide recommendations so that the modifications will be shown on the project plans.

    Example

    Install lugs on all steel "H" piles prior to driving. It is recommended that the pile detail sheets or abutment detail sheets show the lugs as illustrated in the Bridge Construction Records and Procedures Manual, Bridge Construction Memo 130-5.0, except that the lugs be located 10 feet from the pile tip.

    Example

    Show Modified Class 200, Alternative "W" steel pipe pile details on the project plans. The modified pipe pile must be shown with a flat circular steel plate or conical steel tip with a minimum thickness of ¾ inch welded to the pile tip, similar to the Alternative "V" pile tip detail shown in the Standard Plans.

9. If the design calculations account for liquefaction, discuss how the effect of liquefaction was incorporated in the pile foundation recommendations.
10. When the foundation soil is designated as Class S2 soil, the Structure Designer will request soil parameters for the lateral analyses for both non-liquefied and liquefied conditions. In some cases, complete lateral analyses may also be requested. Present the data in the Appendix.

11. If the design calculations account for seismic downdrag then add the following:

    "The design loads and design tip elevations were adjusted to account for seismic downdrag. The additional seismic downdrag loads calculated by Geotechnical Services were provided to Structure Design, and appropriate load factors were applied by Structure Design and incorporated into the Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5) provided by Structure Design to this Office."

12. If the design calculations account for static downdrag, discuss how the effect of static downdrag was incorporated in the pile foundation recommendations.
13. When a pile cap excavation is anticipated to extend below the groundwater surface elevation, the Geoprofessional must discuss with the Structure Designer and identify the "type" of structure excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).

    Example

    Show Type D excavation on the plans at Piers 2 and 3.

14. For CISS piles, state the top of soil plug elevation and the seal course thickness (if applicable) required for the tip resistance design of CISS piles.

    Example

    At Abutment 4, a soil plug is utilized to develop internal side resistance in the lower portion of the CISS pile for tip resistance design. The top of the soil plug elevation must be at elevation 252 feet. A seal course thickness of 5 feet is required to counteract the hydrostatic forces of the groundwater and to allow for the pile reinforcement and concrete to be poured in the dry.

Table X: Foundation Design Data Sheet

Support Location	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1							
Pier 2							
Abut 3							

Table X: Foundation Factored Design Loads

Support Location	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load per Support	Permanent Load per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. per Pile	Per Support	Max. per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut-Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Perm.		Comp. (ϕqs=0.7) (ϕqs=0.7)	Tension (ϕqs=0.7)	Comp. (ϕqs=1) (ϕqs=1)	Tension (ϕqs=1)			
Abut 1	Class 140 Alt "V"							N/A	N/A	(a-I) (c)		
Pier 2	CISS 24 x 0.5									(a-I) (b-I) (a-II) (b-II) (c)		
Abut 3	Class 140 Alt "V"							N/A	N/A	(a-I) (c)		

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement

If the design tip elevation for settlement is not calculated because the pile tip is in rock, add the following note:

- Design Tip Elevations for Settlement were not calculated because the pile are tipped in rock.

If applicable:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevation, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Abutment 1	Class 140 Alt. "V"			(a) (c)		
Pier 2	CISS 24 x 0.5			(a) (b) (c)		
Abutment 3	Class 140 Alt. "V"			(a) (c)		

Present the following Notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a) Compression, (b) Tension, (c) Settlement

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable, add the following note:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

3.12.2.2 CIDH Concrete Pile Foundations

Provide complete and concise recommendations by addressing the topics in the applicable portions of this section. Discuss any considerations which influence type selection, analysis, and design (e.g., scour, liquefaction, lateral spreading, groundwater, usage of casings or shells). Discuss how the pile was designed (e.g., side and tip resistance, locations where geotechnical resistance was ignored, downdrag zones).

Present the following in the Preliminary Foundation Report where the geotechnical investigation is complete:

1. Information provided by Structure Designer
    a. Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1, Table 3-2)
2. Preliminary Foundation Design Recommendations table, that includes the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} = 0.7 and ϕ_{qp} = 0.5 for side resistance and tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 and 10.5.5.2.4-1.
    b. Round the Required Nominal Resistance up to the nearest 10 kips.

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Table X: Preliminary Foundation Design Recommendations

Support Location	Pile Type	Cutoff Elevation (feet)	Required Nominal Resistance (Strength Limit State) (kips)		Preliminary Tip Elevation (feet)
			Compression (ϕqs=0.7) (ϕqp=0.5)	Tension (ϕqs=0.7)	
Abutment 1					
Pier 2					
Abutment 3					

Add note if applicable:

- Cutoff elevations not provided by Structure Design and are estimated by Geotechnical Services

Present the following in the Foundation Report:

1. Information provided by Structure Designer
    a. Foundation Design Information Sheet (MTD 3-1, Attachment 1, Table 3-4)
    b. Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5)
2. Foundation Design Recommendations table (MTD 3-1, Attachment 1, Tables 3-6 or Tables 3-8), with the following modifications:
    a. Report the resistance factors (in column header) using the appropriate notations, e.g., ϕ_{qs} = XX for side resistance, ϕ_{qp} = YY for tip resistance, see AASHTO LRFD BDS, CA Amendment 10.3 & 10.5.5.2.4-1.
    b. Round the Required Nominal Resistance (Strength and Extreme Limit State) and Factored Design Loads (Service Limit State) to the nearest 10 kips.
    c. If a CIDH concrete pile is supporting a single column, identify whether the pile is a Type I or Type II shaft in the “Pile Type” column.
    d. For situations where a Permanent Steel Casing is used:
        i. add column “Permanent Steel Casing Specified Tip Elevation (feet)”.
    e. For situations where a Driven Steel Shell is used as a Permanent Casing:
        i. add column “Driven Shell Specified Tip Elevation (feet)”.
        ii. add column “Required Nominal Driving Resistance (kips)”.
    f. For situations where a Rock Socket is used:
        i. Add column “Top of Rock Socket Elevation (feet)” to the Foundation Design Recommendations table and the Pile Data Table.
        ii. Add footnote: “The Bottom of Rock Socket Elevation is equal to the Specified Tip Elevation.” to the Foundation Design Recommendations table and the Pile Data Table.
3. Pile Data Table (MTD 3-1, Attachment 1, Tables 3-7 or 3-9), with the following modifications:
    a. Round the Nominal Resistance up to the nearest 10 kips.
    b. If a CIDH concrete pile is supporting a single column, identify whether the pile is a Type I or Type II shaft in the “Pile Type” column.
    c. For situations where a Permanent Steel Casing is used:
        i. add column “Permanent Steel Casing Specified Tip Elevation (feet)”.
    d. For situations where a Driven Steel Shell is used as a Permanent Casing:
        i. add column “Driven Shell Specified Tip Elevation (feet)”.
        ii. add column “Required Nominal Driving Resistance (kips)”.
    e. For situations where a Rock Socket is used:
        i. Add column “Top of Rock Socket Elevation (feet)” to the Foundation Design Recommendations table and the Pile Data Table.
        ii. Add footnote: “The Bottom of Rock Socket Elevation is equal to the Specified Tip Elevation.” to the Foundation Design Recommendations table and the Pile Data Table.

If applicable, present and/or discuss the following additional items:

4. If the design calculations account for liquefaction, discuss the how the effects of liquefaction was incorporated in the pile foundation recommendations
5. When the foundation soil is designated as Class S2 soil, the Structure Designer will request the soil parameters for the lateral analysis for both non-liquefied and

liquefied conditions. In rare cases, complete the lateral analysis may also be requested. Present the data in the Appendix.

6. If the design calculations account for seismic downdrag then add the following:

   "The design loads and design tip elevations were adjusted to account for seismic downdrag. The additional seismic downdrag loads calculated by Geotechnical Services were provided to Structure Design, and appropriate load factors were applied by Structure Design and incorporated into the Foundation Factored Design Loads (MTD 3-1, Attachment 1, Table 3-5) provided by Structure Design to this Office."

7. If the design calculations account for static downdrag, discuss the how the effect of static downdrag was incorporated in the pile foundation recommendations.
8. If Permanent Smooth-wall Steel Casing, Driven Steel Shell, or CMP is recommended, discuss the reasoning of their selection in the report. State if the structural capacity (from Structure Design) and/or geotechnical side resistance of the casing, shell, or CMP is used in pile design. If a CMP is to be utilized, state that the permanent steel casing must be a CMP.

   Example

   The structural capacity and geotechnical side resistance of the driven steel shell were used in the design of the pile.

   Example

   The structural capacity of the permanent smooth-wall steel casing was used in the design of the pile. The geotechnical side resistance of the permanent smooth-wall steel casing was not used in the design of the pile.

   Example

   The permanent casing may be specified as either smooth-walled steel or a CMP. Neither the structural capacity nor the geotechnical side resistance of the permanent casing was used in the design of the pile.

   Example

   The permanent casing must be specified as a CMP. The upper 20 feet of the CMP was used for the geotechnical side resistance of the pile design. The structural capacity of the CMP was not used in the design of the pile.

9. When a pile cap excavation is anticipated to extend below the groundwater surface elevation, the Geoprofessional must discuss with the Structure Designer and identify the "type" of structure excavation (Type A or Type D) required at all applicable support locations (See Bridge Design Aids, Section 11 - Estimating).

   Example

   Show Type D excavation on the plans at Piers 2 and 3.

Use the applicable Foundation Design Recommendations table and Pile Data Table listed below, and presented on the following pages.

- CIDH Concrete Pile in Soil without Casing
- CIDH Concrete Pile in Soil with Permanent Casing
- CIDH Concrete Pile in Rock Socket without Permanent Casing
- CIDH Concrete Pile in Rock Socket with Permanent Casing

CIDH Concrete Pile in Soil (no Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation. (feet)
						Strength/Construction		Extreme Event			
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression	Tension		
Bent 2	60” CIDH Concrete Piles									(a-I) (a-II) (c)	
Bent 3	60” CIDH Concrete Piles									(a-I) (a-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Soil (no Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension		
Bent 2	60” CIDH Concrete Piles			(a) (c)	
Bent 3	60” CIDH Concrete Piles			(a) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (c) Settlement.
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevation provided by Structure Design is lowest design tip elevation, and is therefore the Specified Tip Elevation.

CIDH Concrete Pile in Soil (Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Permanent Casing Specified Tip Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
						Strength/Construction		Extreme Event				
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0) (φqp=1.0)	Tension (φqs=1.0)			
Bent 2	60” CIDH Concrete Piles with Casing (Type II Shaft)										(a-I) (a-II) (c)	
Bent 3	60” CIDH Concrete Piles with Casing (Type II Shaft)										(a-I) (a-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Soil (Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Permanent Casing Specified Tip Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension			
Bent 2	60” CIDH Concrete Piles with Casing (Type II Shaft)				(a) (c)	
Bent 3	60” CIDH Concrete Piles with Casing (Type II Shaft)				(a) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (c) Settlement.
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (no Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation. (feet)
						Strength/Construction		Extreme Event				
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0)	Tension (φqs=1.0)			
Bent 2	60” CIDH Concrete Piles										(a-I) (a-II) (b-I) (b-II) (c)	
Bent 3	60” CIDH Concrete Piles										(a-I) (a-II) (b-I) (b-II) (c)	

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevation for settlement is not calculated because the pile tip is in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (no Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)
		Compression	Tension			
Bent 2	60” CIDH Concrete Piles				(a) (b) (c)	
Bent 3	60” CIDH Concrete Piles				(a) (b) (c)	

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement.
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (Permanent Casing)

Table X: Foundation Design Recommendations

Support Location	Pile Type	Cut Off Elevation (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Steel Casing Specified Tip Elevation (feet)
						Strength/Construction		Extreme Event					
			Total	Permanent		Compression (φqs=0.7) (φqp=0.5)	Tension (φqs=0.7)	Compression (φqs=1.0) (φqp=1.0)	Tension (φqs=1.0)				
Bent 2	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)										(a-I) (a-II) (b-I) (b-II)		
Bent 3	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)										(a-I) (a-II) (b-I) (b-II)		

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement
- Below the steel casing tip elevations, the CIDH concrete pile diameter is 48”
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

CIDH Concrete Pile in Rock Socket (Permanent Casing)

Table X: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Top of Rock Socket Elevation (feet)	Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Steel Casing Specified Tip Elevation (feet)
		Compression	Tension				
Bent 2	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)				(a) (b) (c)		
Bent 3	60” CIDH Concrete Piles with Permanent Casing (Type II Shaft)				(a) (b) (c)		

Present the following notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement.
- Below the steel casing tip elevations, the CIDH concrete pile diameter is 48”
- Bottom of Rock Socket Elevation = Specified Tip Elevation
- The specified tip elevation shall not be raised above the lowest design tip elevation.

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable:

- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations.

3.12.3 Dynamic Monitoring and Pile Load Testing

Identify support locations for dynamic monitoring. The control zones are identified in the Notes for Specifications.

Example: Dynamic Monitoring

At Piers 2, 4 and 6, dynamic monitoring is to be performed on the first CISS pile installed and will determine the pile acceptance criteria (SS 49-1.01D(4)). The control zones are identified in the Notes for Specifications section of this report.

In consultation with FTI staff, present and/or discuss the following for Pile Load Tests:

1) Control zones and associated support locations for the pile load tests.
2) Location, type and specified tip elevation of the load test pile and anchor piles in the Pile Load Test data table.
3) Type of load test
    a. Compressive (ASTM D 1143)
    b. Tensile (ASTM D 3689)
4) Purpose of test (select one)
    a. Proof test at Nominal Resistance
    b. Load to failure (provide estimate of maximum test load)
5) Identify piles to be dynamically monitored (Per the Standard Specifications, the load test pile and at least one anchor pile).
6) Restrike schedule if pile-setup is anticipated.

Example: Driven CISS Piles – Pile Load Test

Pile load tests in compression must be conducted on a non-production 48-inch CISS pile between Bent 4 of the left and right bridges, installed as required in the Pile Load Test Data table (Table 1). The control zone for the pile load test at Bent 4 will be Bents 2, 3, and 4 of the left and right bridges. During pile installation, the load test pile and one anchor pile will be dynamically monitored.

The compression test must be performed in two stages. For Stage 1 Load Test, load the test pile to the nominal axial geotechnical resistance after driving the test pile to the tip elevation and before removal of the soil plug. For Stage 2 Load Test, load the test pile to the "failure load" after removal of the soil plug, placement a 5-foot thick seal course, and placement of concrete in the test pile. The estimated maximum test load is 2700 kips.

Table 1: Pile Load Test Data

Support Location	*Pile Type*	*Ground Elevation (feet)*	*Cut-Off Elevation (feet)*	*Specified Top of Soil Plug Elevation (feet)*	*Specified Tip Elevation (feet)*	*Required Nominal Driving Resistance (kips)*
Bent 4	*CISS 48 x 1.0 (Load Test Pile)*	*98*	*108*	*40*	*30*	*2350*
	CISS 24 x 0.5 (Anchor Piles)	*98*	*101*	*N/A*	*40*	*800*

3.12.4 Approach Fills

This section may be omitted if the approach fill (fill within 150 feet of the bridge abutment) does not require special considerations for soft or otherwise unsuitable soil.

Present approach fill recommendations in this section. Reporting requirements for embankment construction on soft soil are presented in the *Embankment* module and the *Ground Improvement* module of the Geotechnical Manual.

In cases where settlement-related recommendations are presented, the Geoprofessional must collaborate with the author of the Geotechnical Design Report to assure that the recommendations are compatible. Issues to discuss may include:

- Anticipated settlement magnitude
- Rate of construction
- Settlement periods
- Surcharge locations and heights
- Prefabricated vertical drain locations, spacings, and lengths (see *Ground Improvement* module)
- Use of geosynthetics for separation and/or reinforcement (see *Geosynthetics* module)

3.13 Notes for Specifications

Omit this section for the Preliminary Foundation Report.

This section provides recommendations to the Specifications Engineer for inclusion and editing of Standard Special Provisions and developing NSSPs. Refer to the *Geotechnical Notes for Specifications* module for guidance on how to prepare this report section.

Note: This is a new section in this reporting standard. The information placed in this section was previously in the “Construction Considerations” section. This updated version of FR for Bridges separates communication to the Specifications Engineer (Notes for Specifications) and to Construction (Notes for Construction).

3.14 Notes for Construction

Omit this section for the Preliminary Foundation Report.

Notes for Construction are written to State construction personnel and contractors. Specific geologic conditions that are relevant to construction inspection should be cited in this section to ensure that both the intent of the geotechnical design is met and construction is successful. Include the following Notes if applicable.

3.14.1 Notes for Construction (Shallow Foundations)

1) Include the following instructions (edited for the project site conditions) to address potential disturbance of native material below the specified bottom of footing elevation(s).

Example: Footing on Soil

At all support locations, the spread footings are to be founded on the native alluvium. The structural concrete is to be placed neat against the undisturbed native alluvium at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed to a depth of 1 foot below the disturbance, and replaced at 95% relative compaction.

Example: Footing on Rock

At all support locations, the spread footings are to be founded on the weathered rock. The structural concrete is to be placed neat against the trimmed walls and undisturbed rock at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed and replaced to the bottom of footing elevation with concrete.

2) Include the following instructions to request footing inspections by the Geoprofessional.

Example

All support footing excavations are to be inspected and approved by the Office of Geotechnical Design X, Branch Y. The inspections are to be made after the excavation has been completed to the bottom of footing elevations and prior to placing concrete or rebar in the excavations. The contractor is to allow seven working days for the inspection of each footing excavation to be completed. The Structures Representative is to provide the Office of Geotechnical Design X, Branch Y a one-week notification prior to beginning the seven-day contractor waiting period.

(Note: If sub-excavation and replacement are required, modify the above example to require the inspection to be performed when the contractor completes the sub-excavation and prior to replacement.)

3.14.2 Notes for Construction (Driven Piles)

3) State how the nominal resistance was developed for CISS piles.

Example

The geotechnical resistance of the CISS piles was developed using external side resistance and tip resistance based upon a combination of the end area of the steel shell and the internal side resistance of the portion of the soil plug that will be left in place.

4) Provide cut-off criteria for pile acceptance (Standard Plan and H piles).

Example

Pile acceptance is to be based on Standard Specifications 49-2.01A(4)(c) "Department Acceptance." At Abutments 1 and 4 support locations, any pile that achieves 1½ times the required nominal driving resistance in compression, as shown on the contract plans, within 5 feet of the specified pile tip elevation, may be accepted and cut off with written approval from the Engineer. (i.e. 1½ times the required nominal driving resistance in compression is 580 kips at Abutment 1 and 600 kips at Abutment 4).

3.14.3 Notes for Construction (CIDH Concrete Piles)

5) State if the pile construction is expected to encounter groundwater, and if the slurry displacement method is anticipated for the CIDH concrete piling construction.

   Example

   It is anticipated that concrete placement for the CIDH concrete piles will occur below the groundwater surface, which may necessitate the slurry displacement method.

6) Report how the geotechnical resistance is derived, whether from side resistance and/or tip resistance. Present the highest "Top" elevation and lowest "Bottom" elevation for soil that contribute to pile side resistance in the "CIDH Concrete Pile Side Resistance Zone Elevations" table.

   Example: CIDH Geotechnical Resistance in Soil

   The calculated "Nominal Resistance" of the CIDH concrete piles was based on side resistance only. Tip resistance was not used. The zones used to calculate the side resistance of the CIDH concrete piles are shown in Table 1.

Table 1: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	*Top of Side Resistance Zone Elevation (feet)*	*Bottom of Side Resistance Zone Elevation (feet)*	*Specified Tip Elevation (feet)*
Pier 2	*165.4*	*151.4*	*145.0*
Pier 3	*165.4*	*153.4*	*147.0*
Pier 4	*168.5*	*156.0*	*150.0*

Example: CIDH Geotechnical Resistance in Rock Socket

The calculated "Nominal Resistance" of the CIDH concrete piles was based on side resistance only. Tip resistance was not used. The zones used to calculate the side resistance of the CIDH concrete piles are shown in Table 2.

If the actual top of rock elevation varies by more than X feet from the elevation presented in Table 2, the Office of Geotechnical Design Y must be contacted for further instruction.

Table 2: CIDH Concrete Pile Side Resistance Zones

Support Location	*Top of Rock Elevation (feet)*	*Top of Side Resistance Zone Elevation (feet)*	*Bottom of Side Resistance Zone Elevation (feet)*	*Specified Tip Elevation (feet)*
Pier 2	*67.5*	*65.5*	*50.5*	*48.5*
Pier 3	*67.5*	*65.5*	*52.5*	*50.5*
Pier 4	*70.5*	*68.5*	*55.0*	*53.0*

3.15 Report Copy List

Reports must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* section of the Offices of Geotechnical Design – Quality Management Plan.

3.16 Appendices

The Foundation Report appendices provide detailed information supporting foundation type selection, analyses, and recommendations. Reports prepared by Geotechnical Services staff must include the following (in the order presented, numerated as Appendix I, Appendix II, …):

All Foundation Reports:

- Appendix I: Ground Motion Data Sheet

If produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections: Do not include copies of published maps.
- Geophysical Test Reports
- Laboratory Test Data (including Corrosion Test Report) – Organized by test type. In addition to the raw laboratory test results, organize and provide summary tables and graphs developed for the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for lateral analysis and/or P-Y Curves

Optional:

- Photos of Rock Cores
- Photos relevant to the investigation findings, design recommendations, and construction. Photos that illustrate content presented in the text should be embedded in the report if feasible.

Reports prepared by consultants must include the following (in the order presented, numerated as Appendix I, Appendix II, …):

All Foundation Reports:

- Appendix I: Site Map showing project location
- Appendix II: Ground Motion Data Sheet
- Appendix III: Log of Test Borings (including as-built LOTB)
- Appendix IV: Field Exploration and Testing: Data acquired from field exploration and testing such as surface geologic mapping and surface geophysical surveys, logs from the Cone Penetration Test, Pressuremeter, Dilatometer, and in-situ Vane Shear Tests, Borehole Geophysical logging, indicator pile tests, Piezometer Readings, etc.
- Appendix V: Calculation Package
    - The objectives of each calculation, such as time rate of settlement or bearing resistance.
    - List calculation assumptions
    - The geotechnical model used for each calculation
    - The equations used and meaning of the terms used in the equations
    - Copies of the curves or tables used in the calculations and their source.
    - The load and resistance factors, or factors of safety, used for the design
    - If the calculations are performed using computer spreadsheets – step-by-step calculations for one example to demonstrate the basis of the spreadsheet. A computer spreadsheet is not a substitute for the step-by-step calculation.
    - Summary of the calculation results that form the basis of geotechnical recommendations, including a sketch of the design, if appropriate.
- Previous Caltrans review comments and responses

If produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections. Do not include copies of published maps.
- Photos of Rock Cores
- Photos that are relevant to the investigation findings, design recommendations, and construction. Photos that illustrate content presented in the text should be embedded in the report if feasible.
- Geophysical Test Reports
- Laboratory Test Data (including corrosion) – Organized by test type. In addition to the laboratory test results, summarize and provide summary tables and graphs developed for the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for lateral analysis and/or P-Y Curves

FOUNDATION REPORTS
For
EARTH RETAINING SYSTEMS
(ERS)

January 2021



DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

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1. INTRODUCTION

The intent of this document is to define the Department's standard of practice for preparation of the Structures Preliminary Geotechnical Report (SPGR), the Preliminary Foundation Report (PFR), and the Foundation Report (FR) for earth retaining systems (ERS). Standardized and consistent report presentations for projects statewide benefit the Department's staff, engineering consultants, bidders, and contractors. Geotechnical Services staff as well as any other organization preparing these reports must comply with the requirements presented herein.

1.1 Reporting for Caltrans Project Delivery

Geotechnical investigation and reporting generally occurs at three stages of the project development process:

- A Structures Preliminary Geotechnical Report (SPGR) to support Advanced Planning Studies, performed during the WBS 150.15 (K Phase).
- A Preliminary Foundation Report (PFR) to support Type Selection, performed during the WBS 160.10 (0 Phase) or 240.70 (1 Phase).
- A Foundation Report (FR) to support the design and construction of the ERS, performed during the WBS 240.80 (1 Phase).

Prepare a separate foundation report for each ERS. If requested by the client, multiple ERS may be placed in one report provided that the report is archived separately at each location along with the applicable LOTB.

Prepare reports to succinctly communicate information pertinent to the recommendations in accordance with the report preparation requirements. The following rules must be followed:

- Use proper grammar, spelling and punctuation.
- Present only useful specific information that is relevant to the recommendations.
- Reference or cite existing standards, specifications or policy only when clarifying, modifying, or disallowing the standard, specification or policy.
- Do not include unsubstantiated disclaimers.
- Provide titles for all figures and tables.
- Tables and figures must be included within the body of the report and located as near as possible to the place where they are first referenced.
- All depth references must have a corresponding elevation in parenthesis.

1.1.1 Reports Prepared by Caltrans Staff

Foundation Reports are written to the Structure Designer, Specification Engineer, and Structure Construction, and are part of the contract.

For reports prepared by Geotechnical Services staff, Foundation Reports must be prepared using the current departmental memorandum format with the subject line of "Foundation Report for ERS Name" or "Preliminary Foundation Report for ERS Name" or "Structures Preliminary Geotechnical Report for ERS Name". Do not include section numbers in the report. First-level and second-level section titles presented in this document (e.g., Geologic and Geotechnical Conditions, and Geology) must be included in the report. Other section titles are optional.

Do not include the Log of Test Borings (LOTB) and/or As-built LOTB as part of the FR. The Engineering Graphics Unit will send Microstation LOTB files and scanned copies of the As-built LOTB sheets to the Structure Designer for inclusion within the Contract Plans.

Sign and stamp reports in accordance with the *Communications and Reporting* section of the *Offices of Geotechnical Design – Quality Management Plan*.

1.1.2 **Reports Prepared by Consultants**

Foundation Reports must include the following: cover sheet, table of contents, main contents per this document, and appendices. The cover of the report and any addenda/amendments to the report must include the following information: Caltrans District, County, Route, Post Mile, Structure Number, Structure Name, and Expenditure Authorization (EA) number.

The LOTB and/or As-built LOTB must be submitted as part of the FR. Refer to the Caltrans Soil and Rock Logging, Classification, and Presentation Manual for direction on the preparation of the LOTB and As-built LOTB.

2. STRUCTURE PRELIMINARY GEOTECHNICAL REPORT (SPGR)

The SPGR is required during the early stages of a project to assist Structure Design in the preparation of an Advanced Planning Study and cost estimate for the District. Often the number, location, and type of ERS are not completely known. As a result, recommendations may be general, and detailed field investigations are usually not warranted. Typical fieldwork consists of a site visit only. The SPGR provides an overview of the existing foundations, site geology, seismic information, and recommendations regarding suitable and unsuitable wall types. If appropriate, the SPGR should also discuss the anticipated field and laboratory work required to support the PFR and FR.

The following topics should be addressed in all Structure Preliminary Geotechnical Reports (SPGR).

2.1 Introduction

Summarize the purpose, scope, and types of work performed to obtain the information supporting the preliminary recommendations. Reference the request memo and applicable plans by date so the reader knows on what plans the recommendations are based. Do not present an exhaustive list of tasks performed, a few sentences are sufficient.

2.2 Project Description

Describe the project location, existing and/or proposed ERS and structures, and pertinent project information, such as the reason for constructing the ERS. A table such as the one below may be used to present the information.

ERS Information Table

ERS ID No.	ERS Type	Begin Station. or PM	End Station. or PM	Length, (feet)	Max. Design Height (feet)	Notes

2.3 Exceptions to Policies and Procedures

List exceptions to Departmental policies and procedures relating to the SPGR. Approved *Request for Exception* forms must be included in the Appendix. Omit this section if there are no exceptions.

2.4 Geotechnical Investigation

Provide an overview of the geotechnical investigation(s) to support the preliminary ERS recommendations.

2.5 **Geotechnical Conditions**

2.5.1 **Geology**

Identify the pertinent geologic map and the prominent geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they influence the design and construction of the structure.

2.5.2 **Surface Conditions**

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

2.5.3 **Subsurface Conditions**

Provide a generalized description of the known subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials)

Do not re-create an As-built LOTB in detail in this section. A generalized discussion or table is sufficient.

Example

The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium. The topography is relatively flat and the site appears free of geologic hazards.

Based on the 1968 As-built Log of Test Borings located approximately 500 feet from the ERS, the alluvial soil at the site can generally be separated into three units. The upper unit consists of very loose to slightly compact silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of slightly compact to dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum explored depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

2.6 **Groundwater**

Report groundwater elevation(s) and dates of measurements. Use of a table is recommended if there are numerous borings and/or measurements.

Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation, (feet)	Date Measured	Depth to Groundwater (feet)	Groundwater Surface Elevation (feet)

Example: Groundwater Present

During the 1998 subsurface investigation at the adjacent bridge, groundwater was encountered in both borings. Groundwater levels varied from elevation 945 feet in February to elevation 938 feet in August. Recent groundwater measurements by the Department of Water Resources at a well located roughly 1300 feet north of ERS site are generally consistent with the 1998 measurements.

Example: Groundwater Not Present

During the 1998 subsurface investigation at the adjacent bridge, groundwater was not encountered in either boring within the explored depth of 100 feet (~ Elev. 900 feet).

Example: Groundwater Information Not Available

Groundwater information was not available based upon the literature search performed.

Example: Groundwater Information Available Nearby

Groundwater measurements available from a DWR monitoring well, located 800 feet northwest of the proposed ERS, had groundwater elevations that varied between 930 ft and 920 feet from 2015 to present.

2.7 As-Built Data

Include brief discussion of relevant As-Built data, such as:

- As-Built LOTB
- Existing or nearby ERS and foundation types
- Other man-made features
- Construction records such as pile driving logs, settlement monitoring data, etc.

2.8 Scour Data

For cases where the ERS is adjacent to a waterway, report the pertinent scour information, including the potential for scour and the predicted magnitude of scour. Otherwise, omit this section.

2.9 Corrosion Evaluation

Report and discuss pertinent site corrosion data.

Example: No information available

Historical corrosion data is not available. For preliminary design purposes the site should be considered non-corrosive based on the presence of predominantly cohesionless material. Corrosion samples will be obtained during the design phase to evaluate the corrosion potential of the site.

Example: Non-Corrosive

Three soil samples and one water sample were collected for corrosion testing during the 2011 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on current Caltrans' standards, the site is considered to be non-corrosive.

Example: Corrosive

During the 2011 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-11-001 and RC-11-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on current Caltrans' standards, and corrosion mitigation is required.

Table 1: Soil Corrosion Test Summary

Boring ID	Elevation (feet)	Minimum Resistivity (Ohm-Cm)	pH	Chloride Content (ppm)	Sulfate Content (ppm)	Corrosive?
RC-11-001	15.8 to 14.3	1544	7.24	N/A	N/A	No
RC-11-001	-4.2 to -3.2	683	7.94	384	432	No
RC-11-002	-69.1 to -70.6	73	6.86	**850**	**1500**	Yes
RC-11-002	-104.1 to -105.6	78	7.71	**1000**	**1600**	Yes

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

2.10 **Seismic Information**

Report all information required in Section 2.10.1 in the SPGR. Referencing a Seismic Report that was delivered separately is not acceptable. Information required in Section 2.10.2 should be summarized while referencing the reader to the applicable report (e.g., Fault Rupture Report).

2.10.1 **Ground Motion Hazard**

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)

b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., CPT or SPT correlations). See *Design Acceleration Response Spectrum (ARS)* module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
    - The horizontal peak ground acceleration (HPGA or PGA).
    - Deaggregated mean earthquake moment magnitude (M) for the PGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. A statement of whether the K_h used in the standardized ERS designs is applicable to the site.

Example

The site is susceptible to strong earthquake induced ground motions during the design life of the ERS.

Based on available subsurface information and Standard Penetration Test (SPT) correlations for determining shear wave velocity, the time-average shear wave velocity (VS30) for the upper 100 feet of soil at the site is estimated to be 980 ft/sec.

The Horizontal Peak Ground Acceleration (PGA) is the ground motion at the site with a 5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the ground motion. Adjustments for near-fault and basin effects were implemented, when applicable, as per Appendix B of the SDC v2.0.

Caltrans web-based tool ARS Online v3.0 was utilized to determine the preliminary horizontal PGA = 0.27g motion parameters, including the ARS, for the subject structure site.

2.10.2 **Other Seismic Hazards**

The section must include information on the following seismic hazards, as applicable at the site:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Seismically induced total and differential ground settlements
d. Lateral spreading potential (see *Lateral Spreading* module)
e. Seismic slope instability
f. Tsunami risk

Example

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

Groundwater was not encountered within the As-built borings drilled to depths ranging from 70 to 100 feet (~ Elev. 90 to 60 feet) from the existing ground surface. Dense and/or stiff soils were encountered in these borings below a depth of about 60 feet (~Elev. 100 feet) from the existing ground surface. Based on these groundwater and subsurface soil conditions, the project site is not considered susceptible to liquefaction or related seismic hazards, including seismic total or differential ground settlement, and lateral spreading.

The project site and the adjacent areas are relatively flat. The site is located more than 0.5 miles from the nearest coastline and is situated above elevation 40 feet, therefore the risk for tsunami does not exist (per MTD 20-13).

2.11 Geotechnical Recommendations

Provide preliminary recommendations for the ERS. Recommendations must include the following:

- ERS types considered, and advantages and disadvantages of each
- Recommended ERS and alternatives
- ERS location (begin and end station, if available) and geometry (length, height)
- Description of external loadings (surcharge, landslide, groundwater)
- Description of site constraints (environmental, right-of-way, utilities, traffic, construction, etc.)

2.12 Additional Field Work and Laboratory Testing

Describe the anticipated scope and types of fieldwork and testing that may be required to complete the geotechnical investigation. Discuss the potential need for entry permits, task orders, groundwater monitoring, access road construction, lane closures, etc.

2.13 Report Copy List

The SPGR must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* module.

3. PRELIMINARY FOUNDATION REPORT (PFR) and FOUNDATION REPORT (FR)

The PFR is prepared after completion of the SPGR and Advanced Planning Study, and prior to the Structure Type Selection. The ERS location, type, height, and length will be better defined, and the site investigation may be complete. The amount of information provided in the PFR will be relative to the information provided by Structure Design and the extent of geotechnical investigation completed.

The FR expands on data provided in the PFR and updates the foundation recommendations based upon final design details provided by Structure Design. The FR becomes part of the contract documents via its inclusion in the Information Handout per Standard Special Provision 2-1.06B, "Supplemental Project Information."

The following topics, if applicable, must be addressed in the Preliminary Foundation Report and Foundation Report.

3.1 Introduction

Summarize the scope and types of work performed to obtain the information supporting the foundation recommendations.

Example: Preliminary Foundation Report with 0-Phase Drilling

Per the request dated May 3, 2020, this Preliminary Foundation Report has been prepared for the proposed ground anchor wall. The purpose of this report is to summarize the investigations performed and to provide preliminary foundation recommendations. The recommendations presented in this report are based on the draft layout plan dated June 15, 2020, and a subsurface investigation consisting of borings along the wall layout line.

This Preliminary Foundation Report supersedes the Structure Preliminary Geotechnical Report for (Structure Name) dated (Date).

Example: Foundation Report

Per the request dated May 3, 2020, this Foundation Report has been prepared for the proposed ground anchor wall. The purpose of this report is to summarize the investigations performed and to provide foundation recommendations. The recommendations presented in this report are based on the layout plan dated June 15, 2020, and a subsurface investigation.

This Foundation Report supersedes the Preliminary Foundation Report for (Structure Name) dated (Date) and the Structure Preliminary Geotechnical Report for (Structure Name) dated (Date).

3.2 Project Description

Describe the project location, existing and/or proposed ERS, and pertinent project information, such as the reason for constructing the ERS and site constraints, including adjacent structures. Provide project vertical datum reference. A table such as the one below may be used to present the information.

ERS Information Table

ERS ID No.	ERS Type	Begin Northing/Easting Latitude /Longitude		End Northing/Easting (Latitude /Longitude)		Length, (feet)	Maximum Design Height (feet)	Notes

3.3 **Exceptions to Policies and Procedures**

Discuss exceptions to Departmental policies and procedures relating to the PFR/FR. Approved Request for Exception forms must be included in the Appendix. Omit this section if there are no exceptions.

3.4 **Geotechnical Investigation**

Provide an overview of the geotechnical investigation(s) performed to support the foundation recommendations including the number of boreholes/CPT soundings, the maximum depth and elevation, and the types of field and/or downhole testing (e.g., in-situ, geophysical).

Example

Geotechnical Investigation was done by reviewing the as-built borings from the 1966 investigation of the adjacent bridge and by drilling three borings along the proposed wall layout line in June 2020. The 1966 foundation investigation consists of one 3-inch mud rotary boring and eight 1-inch driven soil tube borings. In June 2020, three mud rotary borings were drilled to a maximum depth of 50 feet (~ Elev. 230 feet) using a CS2000 drill rig. Standard Penetration Test (SPT) testing was performed at regular intervals to evaluate the engineering properties of the earth materials. The type(s) and location(s) of field testing are shown on the LOTB sheets.

3.5 **Laboratory Testing Program**

Provide an overview of the laboratory testing program, if performed, to support the ERS recommendations. Briefly explain what the tests were used for (e.g. soil classification, settlement, strength parameters).

Example

During the most recent field investigation, soil samples were collected from borings RC-20-001 and RC-20-002 for soil classification and liquefaction evaluation. A summary of the test results will be provided in the Appendix, and the test sample locations are shown on the Log of Test Borings.

3.6 **Geotechnical Conditions**

3.6.1 **Geology**

Identify the pertinent geologic map and the prominent geologic unit(s) at the structure site. Describe relevant geologic features such as faults, bedding, major joint attitudes, and folds if they may influence the design and construction of the structure.

3.6.2 **Surface Conditions**

Describe site topography, surface water and drainage conditions, cuts and fills, geologic hazards such as landslides and rockfall, and land use history that may affect the proposed structure. Identify existing structures, facilities, and utilities near the proposed structure that may affect its design and construction.

3.6.3 **Subsurface Conditions**

Provide a generalized description of the known subsurface conditions. The information included within this section may include, but is not be limited to:

- Types of soil/rock, depths to generalized layer breaks, and corresponding elevations
- Pertinent soil/rock conditions such as unsuitable materials (collapsible, expansive foundation materials) or rock rippability.

Do not re-create the LOTB(s) in detail in this section. A generalized discussion or table is sufficient.

Example

The Geologic Map of Santa Ana 30' x 60' Quadrangle shows that the site is underlain by Quaternary alluvium. The topography is relatively flat and the site appears free of geologic hazards.

Based on the recent site investigation, the alluvial soil at the site can generally be separated into three units. The upper unit consists of very loose silty sand with gravel that extends from the ground surface to a depth of about 15 feet (~ Elev. 950 feet). The middle unit consists of dense sand to a depth of approximately 35 feet (~ Elev. 930 feet). The lowermost unit consists of dense to very dense gravelly sand and sandy gravel with isolated zones of sandy silt and gravel. This unit extends to the maximum explored depth of the borings, which is approximately 60 feet below the ground surface (~ Elev. 905 feet).

3.7 **Groundwater**

Report groundwater elevation(s) and dates of measurements. Use of the following table is recommended if there are numerous borings and/or measurements. Include discussions relating to the presence of wet or saturated soil when groundwater measurements were not made. Discuss surface water conditions that might influence the design or construction of the foundations. State the groundwater elevation(s) used for analyses and design.

Summary of Groundwater Data

Location or Boring ID	Ground Surface Elevation, (feet)	Date Measured	Depth to Groundwater (feet)	Groundwater Surface Elevation (feet)

Example

As-built LOTB's from the April 1968 subsurface investigation of the adjacent bridge indicate that groundwater was encountered in several borings at that time, and ranged from elevation 19.0 feet to elevation 21.2 feet (NAVD88 datum). During the 2020 subsurface investigation groundwater was measured in Boring RC-20-001 at elevation 15.3 feet, and in Boring RC-20-002 at elevation 13.9 feet. The groundwater elevation used for design was 21 feet.

3.8 As-Built Data

Include brief discussion of relevant As-Built data, such as:

- As-Built LOTB
- Existing or nearby ERS and foundation types
- Other man-made features
- Construction records such as pile driving logs, settlement monitoring data, etc.

3.9 Scour Data

For cases where the ERS is adjacent to a waterway, report the pertinent scour information, including the potential for scour and the predicted magnitude of scour. This information may come from hydraulics reports, geotechnical investigations, BIRIS records, etc. Otherwise, omit this section.

If the field investigation reveals geologic information that contradicts the hydraulics report for this project, then the Geoprofessional must discuss the findings in the PFR/FR and provide that information to author of the hydraulics report so that the scour recommendations can be re-evaluated.

Example: Scour Data Available

The ERS site is underlain by alluvial soil, which are considered potentially scourable. The Structure Hydraulics Branch provided scour information in a report dated January 15, 2020, which states the Long Term (Degradation and Contraction) scour extends to elevation 2285 feet, and the Short Term (Local) scour depth is 3 feet.

Example: Scour Data Unavailable

The ERS is adjacent to the Russian River. BIRIS records do not identify any historic scour issues. The Structures Hydraulics Branch has not yet provided a Hydraulic Report to this Office.

3.10 **Corrosion Evaluation**

Include and update the corrosion data from the SPGR based on new findings and field investigations. If corrosion testing was not completed during the geotechnical investigation, provide justification for the corrosion recommendations.

Example: Non-Corrosive

Three soil samples and one water sample were collected for corrosion testing during the 2020 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on current Caltrans' standards, the site is considered non-corrosive.

Example: Corrosive

During the 2020 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings RC-20-001 and RC-20-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered corrosive based on current Caltrans' standards, and corrosion mitigation is required.

Table 1: Soil Corrosion Test Summary

Boring ID	*Elevation (feet)*	*Minimum Resistivity (Ohm-Cm)*	*pH*	*Chloride Content (ppm)*	*Sulfate Content (ppm)*	*Corrosive?*
RC-11-001	*15.8 to 14.3*	*1544*	*7.24*	*N/A*	*N/A*	*No*
RC-11-001	*-4.2 to -3.2*	*683*	*7.94*	*384*	*432*	*No*
RC-11-002	*-69.1 to -70.6*	*73*	*6.86*	***850***	***1500***	*Yes*
RC-11-002	*-104.1 to -105.6*	*78*	*7.71*	***1000***	***1600***	*Yes*

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. With the exception of MSE walls, soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,100 ohm-cm.

3.11 **Seismic Information**

Update the seismic information required for the SPGR based on new findings and/or investigations. Summarize analyses and evaluations performed, and recommendations relating to seismic design.

3.11.1 **Ground Motion Hazard**

Include the following information:

a. Site coordinates (latitude and longitude in decimal degrees)
b. The estimated time-average shear wave velocity V_{S30} and how it was determined (e.g., geophysics, seismic CPT or SPT correlations). See *Design Acceleration Response Spectrum (*ARS) module.
c. For 5% probability of exceedance in 50 years (Return Period = 975 years).
    - The horizontal peak ground acceleration (HPGA or PGA).
    - Deaggregated mean earthquake moment magnitude (M) for the PGA and the mean site-to source distance (R) for the 1.0 second period spectral acceleration.
d. A statement of whether the K_h used in the standardized ERS designs is applicable to the site.

Example

The site is susceptible to strong earthquake induced ground motions during the design life of the ERS.

Based on available subsurface information and Standard Penetration Test (SPT) correlations for determining shear wave velocity, the time-average shear wave velocity (VS30) for the upper 100 feet of soil at the site is estimated to be 980 ft/sec.

The Horizontal Peak Ground Acceleration (PGA) is the ground motion at the site with a 5% probability of exceedance in 50 years (return period = 975 years). The USGS's 2014 NSHM is used as the basis to determine the ground motion. Adjustments for near-fault and basin effects were implemented, when applicable, as per Appendix B of the SDC v2.0.

Caltrans web-based tool ARS Online v3.0 was utilized to determine the horizontal PGA = 0.27g motion parameters, including the ARS, for the subject structure site.

3.11.2 **Other Seismic Hazards**

The section must include information on the following seismic hazards:

a. Surface fault rupture potential (see *Fault Rupture* module)
b. Liquefaction potential (see *Liquefaction Evaluation* module)
c. Effects of Liquefaction, including
    i. Seismically-induced ground surface settlement
    ii. Downdrag of pile foundations (see *Downdrag* module)
    iii. Lateral spreading potential (see *Lateral Spreading* module)
d. Seismic slope stability
e. Tsunami risk (if applicable)

Discuss the findings and results of other seismic-design analyses, all applicable and necessary geotechnical seismic design recommendations (e.g., residual shear strengths for liquefied soil layers, p-y, t-z and q-w curves, seismic downdrag, lateral spreading loads/displacements, nominal bearing resistances of foundations with and without considering liquefaction for seismic retrofit projects, seismic lateral earth pressures, liquefaction mitigation measures, etc.).

Example: No Hazards

The site has been determined not to have potential for surface fault rupture, liquefaction, seismic-induced slope failure, or tsunami.

Example: No Surface Fault Rupture

The structure is not located within an Alquist-Priolo Earthquake Fault Zone or 1000 feet from any Holocene or younger aged fault. Therefore, per MTD 20-10, the structure is not considered susceptible to surface fault rupture hazards.

Example: Surface Fault Rupture

The structure is located within an Alquist-Priolo Earthquake Fault Zone. Therefore, per MTD 20-10, the structure is susceptible to surface fault rupture hazards. Per the attached Fault Rupture Report dated March 15, 2020, the horizontal displacement is estimated to be one foot and the vertical displacement is estimated to be 4 inches.

Example: Liquefaction

Due to the presence of loose to medium dense alluvial material and shallow ground water beneath the site, the potential for soil liquefaction is present at the site. Liquefiable zone elevations are summarized in Table 1.

Example: Effects of Liquefaction

Liquefaction-induced settlement of the ground surface and pile downdrag are anticipated and summarized in Table 1. Implications of liquefaction on the pile tip elevations will be addressed in the Geotechnical Recommendations section.

Table 1: Liquefaction Potential at Retaining Wall 3

Location	*Liquefaction Elevation (feet)*	*Estimated Seismic-induced Settlement (inches)*	*Downdrag Zone Bottom Elevation (feet)*	*Estimated Downdrag Load (kips/pile)*
Station 0+00 to 0+50	*Elev. 20 to 15 Elev. 0 to -10*	*3*	*-5*	*150*
Station 0+50 to 0+80	*Elev. 10 to -5*	*4*	*-3*	*50*
Station 0+80 to 1+30	*Elev. 20 to 10*	*3*	*12*	*100*

Note: Downdrag loads calculated for 24-inch CIDH concrete piles.

Example: Lateral Spreading Potential

Due to the presence of liquefiable soils at shallow depths and relatively high design horizontal peak ground acceleration, an initial lateral spreading hazard assessment was performed by ignoring all lateral resistance contributions from the foundation piles. The analysis was performed in accordance with Steps 1 through 4 of the Lateral Spreading Analysis Example of the Geotechnical Manual. Results of the analysis indicate a lateral spreading hazard potential.

Additional lateral spreading analyses were performed in accordance with MTD 20-15 and the Lateral Spreading Analysis Example module of the Geotechnical Manual. The pile

restraining force versus displacement plot (MTD 20-15 Figure 5, Curve 3) is shown in Figures X and Y.

Example: Seismic Slope Stability

Seismic slope stability analyses were performed to evaluate the overall stability. The pseudo static analysis with a horizontal seismic coefficient (kh) equal to 0.15g was performed.

Two-dimensional slope stability analyses were performed and the results are included in the appendix. The analyses found the minimum value of factor of safety at the ERS to be approximately 1.25 (resistance factor = 0.8), which meets the accepted minimums for stable abutment slopes (per AASHTO LRFD).

Example: No Tsunami Risk

The site is located about 0.25 miles from the nearest coastline. However, the ground surface elevation at the ERS location ranges from 100 to 120 feet above mean sea level. The site is not located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the X County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard does not exist at the site.

Example: Tsunami Risk

The site is located about 0.25 miles from the nearest coastline and the ground surface elevation at the ERS location ranges from 10 to 50 feet above mean sea level. The site is located within the tsunami inundation zone shown in California Official Tsunami Inundation Map for the X County (Interactive Map accessed on mm/dd/year).

Based on the above information and per MTD 20-13, a tsunami hazard exists at the site.

3.12 Geotechnical Recommendations

Provide complete and concise recommendations by addressing the applicable topics of this section. At the beginning of the recommendations section, discuss the following:

1. Date of plans used for analysis (e.g., General Plan, Retaining Wall Layout Sheets)
2. Description of the recommended ERS
    a. Location (begin and end station, length, and alignment)
    b. Design Height (maximum and minimum)
    c. Describe the representative cross-sectional geometry and external loads. Reference the plan sheets when possible.
3. Geotechnical design parameters
    a. Soil and rock parameters used for geotechnical and structural analyses, presented in the table below. If applicable, provide parameters for short term and long-term design.
    b. Ground water conditions used for both short term and long-term analyses, including results of seepage or flow analyses.
    c. Description of external loads (surcharge, landslide)
        i. Surcharge load locations, magnitudes, types (line, uniform, etc.) and inclinations

ii. Landslide geometry (depth, location), failure mode, and material properties (strength parameters, unit weights). For structures that stabilize slopes, provide maps and cross sections of the slope modeling.

Design Soil Parameters

Layer No.	Layer boundaries	Group Name	Engineering Parameters
1	Finished grade to elev. 300	Silty Sand (fill)	Φ = 34 degrees, γ = 120 pcf
2	Elev. 285 to 300	Silty Sand	Φ = 33 degrees, γ = 113 pcf
3	Elev. 272 to 285	Poorly-graded Sand	Φ = 34 degrees, γ = 120 pcf
4	Elev. 250 to 272	Silty Sand	Φ= 34 degrees, γ = 114 pcf

3.12.2.1 Conventional Retaining Walls

Foundation recommendations for conventional retaining walls (standard detail or modified standard plan) founded on spread footings must include the following:

- Factored gross nominal bearing resistances (strength and extreme event limit states)
- Total and differential settlement as a result of application of the net bearing pressure (service limit state). Differential settlement should be examined both along the alignment of the ERS and between the front and back of the ERS. Effects of ERS construction on adjacent ground and/or existing structures, utilities and other structures, both above and below ground.
- Permissible net contact stress (service limit state)
- Calculated resistance factor for overall global stability and local slope stability (service and extreme event limit states). Provide the method of analysis.
- Susceptibility of foundation material to erosion and recommended mitigation
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- Surface and subsurface drainage systems – describe locations and system configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type.
- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- Foundation improvements required to meet geotechnical design objectives, such as sub-excavation, foundation preloading and surcharge delay periods.
- Minimum unbonded ground anchor length for Type 7 walls that incorporate ground anchors.

The geotechnical analysis results, ERS configuration, and factored load demands (provided in the standard plans or standard detail sheets) must be presented in the table below. A summary *Spread Footing Data Table* must also be provided.

Design Data for Retaining Wall (ERS ID)

ERS Station Limits (feet)	Design Height (feet)	Bottom of Footing Elevation (feet)	Footing Width (feet)	Minimum Footing Embedment Depth (feet)	Limit State	Effective Footing Width (B') (feet)	Gross Uniform Bearing Stress (psf)	Factored Bearing Resistance (psf) Φ_b (q_N) Φ_b =	Net Bearing Stress (psf)	Calculated Settlement at Net Bearing Pressure (inches)	Total Permissible Settlement (inches)
					Service		N/A	N/A			
					Strength IA				N/A	N/A	N/A
					Strength IB				N/A	N/A	N/A
					Extreme I				N/A	N/A	N/A
					Extreme II				N/A	N/A	N/A

Spread Footing Data Table (ERS ID)

ERS Station Limits (feet)	Design Height (feet)	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (φ_b=) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (φ_b=0.8) (ksf)

Foundation recommendations for conventional retaining walls (standard plan, standard detail or modified standard plan) founded on piles must include the following:

- Design tip elevation for service limit state (settlement)
- Design tip elevations for strength limit state (compression and tension)
- Design tip elevations for extreme event limit state (compression and tension)
- Susceptibility of foundation material to erosion and recommended mitigation
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and development of pile down drag loads. Provide recommended mitigation measures.
- Discuss the groundwater condition anticipated over the design life of the ERS. Describe the drainage system location and configuration. Reference the Standard

Specifications for permeable material and geosynthetic filter fabric type. If horizontal drains or underdrains are recommended, include details and specifications.

- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- Foundation improvements required to meet geotechnical design objectives, such as sub-excavation, foundation preloading and surcharge delay periods.

The geotechnical analysis results and factored load demands (provided by the structure designer) must be presented in the following tables.

Pile Foundation Design Recommendations

ERS Station Limits (feet)	Pile Type	Cut-off Elevation (feet)	Service-I Limit State Load per Segment (kips)		Total Permissible Segment Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevations (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp. (φ_{qs}=1) (φ_{qp}=1)	Tension (φ_{qs} =1)			
										(a-I) (b-I) (a-II) (b-II) (c)		
										(a-I) (b-I) (a-II) (b-II) (c)		

Present the following notes under the Foundation Design Recommendations table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a-I) Compression (Strength), (b-I) Tension (Strength), (a-II) Compression (Extreme Event), (b-II) Tension (Extreme Event), (c) Settlement

If the design tip elevation for settlement is not calculated because the pile tip is in rock, add the following note:

- Design Tip Elevations for Settlement were not calculated because the piles are tipped in rock.

If applicable:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevation, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

Pile Data Table

ERS Station Limits (feet)	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Nominal Driving Resistance (kips)
		Compression	Tension			
				(a) (b) (c)		
				(a) (b) (c)		

Present the following Notes under the Pile Data Table. Edit to include only those load cases provided in the table:

- Design tip elevations are controlled by (a) Compression, (b) Tension, (c) Settlement

If the design tip elevations for settlement are not calculated because the pile tips are in rock, add the following note:

- Design Tip Elevations for Settlement not calculated because the piles are tipped in rock.

If applicable, add the following note:

- The specified tip elevations shall not be raised above the design tip elevations for Tension, Settlement and Lateral Load.
- The lateral design tip elevations provided by Structure Design are the lowest design tip elevations, and are therefore the Specified Tip Elevations. The Required Nominal Driving Resistances are based on the lateral design tip elevations.

3.12.2.2 Non-gravity Cantilever Systems

Non-gravity cantilever systems include sheet pile walls, soldier pile walls with or without lagging, tangent pile walls, and secant pile walls.

Foundation recommendations for non-gravity cantilever retaining systems (soldier pile, secant, and tangent or slurry diaphragm retaining walls) must include the following:

- Soil and/or rock parameters for the internal and structural design (unit weight, friction angle and cohesion for soil, and unit weight and shear strength of the rock mass). This applies to the foundation and retained materials.
- Recommended material models, per AASHTO Bridge Design Specifications Section 3.11.5.6, "*Lateral Earth Pressures for Non-Gravity Cantilevered Walls*", for developing the lateral earth pressure distribution (active and passive).
- For designs utilizing vertical piles that carry vertical loads, the factored nominal bearing resistance for piles having a specified tip elevation or embedment length
- For designs utilizing soldier piles, recommendation for minimum pile embedment depth and/or pile tip elevations based on global stability or erosion considerations.
- Total and differential settlement in the retained zone as a result of the placement of fills (service limit state) and pile lateral deflection.
- Calculated resistance factor for overall (global) and local stability (service and extreme event limit states). Provide the method of analysis.

- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- Recommended arching factors for all soil and rock types in which soldier piles are embedded.
- A recommendation for lagging or wall face embedment below finish grade based on erosion, local stability and global stability considerations.
- Discuss the groundwater condition anticipated over the design life of the ERS. Describe the drainage system location and configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type. If horizontal drains or underdrains are recommended, include details and specifications.
- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- Down drag loads on the piles as a result of the addition of overburden pressure.

3.12.2.3 Anchored Pile Systems

The anchored systems category includes ground anchored pile systems, anchored diaphragm walls, and soil nail walls.

Ground Anchored Pile Systems

Anchored pile systems include a row of drilled vertical soldier piles and sub-horizontal drilled ground anchors installed into an anchorage zone behind the retained soil or rock. Recommendation for anchored pile systems must include:

- Soil and/or rock parameters for the internal and structural design (unit weight, friction angle and cohesion for soil, and unit weight and shear strength of the rock mass). This applies to the foundation and retained materials.
- Recommended material models, per AASHTO Bridge Design Specifications Section 3.11.5.7, "*Apparent Earth Pressure (AEP) for Anchored Walls*", for developing the lateral earth pressure distribution (active and passive).
- For designs utilizing vertical piles that carry vertical loads, the factored nominal bearing resistance for piles having a specified tip elevation or embedment length.
- For designs utilizing soldier piles, recommendation for minimum pile embedment depth and/or pile tip elevations based on global stability or erosion considerations.
- Total and differential settlement in the retained zone as a result of the placement of fills (service limit state) and pile lateral deflection.
- Calculated resistance factor for overall (global) and local stability (service and extreme event limit states). Provide the method of analysis.
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- Recommended arching factors for all soil and rock types in which soldier piles are embedded.
- A recommendation for lagging or wall face embedment below finish grade based on erosion, local stability and global stability considerations.
- Discuss the groundwater condition anticipated over the design life of the ERS. Describe the drainage system location and configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type. If horizontal drains or underdrains are recommended, include details and specifications.

- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- The minimum unbonded ground anchor lengths to satisfy static and seismic overall stability (consider providing in tabular format)
- Recommended ground anchor inclination.
- For ERS constructed to retain slope failures, provide the total required earth retention force and the associated global stability resistance factor.
- Recommended location of ground anchors subject to performance testing.
- Down drag loads on the piles as a result of the addition of overburden pressure.

Anchored Diaphragm Walls

Anchored diaphragm walls utilize a reinforced concrete diaphragm as the facing with no pile foundation. The facing typically bears on foundation soil, embedded below a toe bench. Recommendations for anchored diaphragm wills must include:

- Soil and/or rock parameters for the internal and structural design (unit weight, friction angle and cohesion for soil, and unit weight and shear strength of the rock mass). This applies to the foundation and retained materials.
- Total and differential settlement in the retained zone as a result of the placement of fills (service limit state) and wall lateral deflection.
- Calculated resistance factor for overall (global) and local stability (service and extreme event limit states). Provide the method of analysis.
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- A recommendation for lagging or wall face embedment below finish grade based on erosion, local stability and global stability considerations.
- Discuss the groundwater condition anticipated over the design life of the ERS. Describe the drainage system location and configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type. If horizontal drains or underdrains are recommended, include details and specifications.
- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- The minimum unbonded ground anchor lengths to satisfy static and seismic overall stability (consider providing in tabular format)
- Recommended ground anchor inclination.
- For ERS constructed to retain slope failures, provide the total required earth retention force and the associated global stability resistance factor.
- Recommended location of ground anchors subject to performance testing.
- Down drag loads on the wall elements as a result of the addition of overburden pressure.

Soil Nail Walls

Soil Nail Wall recommendations must be reported in accordance with this document and the *Soil Nail Wall* module of the Geotechnical Manual.

3.12.2.4 Mechanically Stabilized Embankment (MSE)

MSE recommendations must be reported in accordance with this document and the applicable Geotechnical Manual module.

- *Mechanically Stabilized Embankment (Caltrans Pre-Designed)*
- *Mechanically Stabilized Embankment (Non-Standard Design)*

3.12.2.5 Prefabricated Modular Walls

Prefabricated modular wall types typically used by Caltrans include crib walls, gabion walls, and mortarless concrete block gravity walls. Typically crib walls utilize a geometry shown in the Caltrans Standard Plans. Geotechnical Services typically determines the geometry and detailing of gabion walls and mortarless concrete block walls.

Foundation recommendations for all types of Prefabricated Modular Walls must include the following:

- Soil and/or rock parameters for the internal and structural design (unit weight, friction angle and cohesion for soil, and unit weight and shear strength of the rock mass). This applies to the foundation and retained materials.
- Factored gross nominal bearing resistances (strength and extreme event limit states)
- Total and differential settlement as a result of application of the net bearing pressure (service limit state). Differential settlement should be examined both along the alignment of the wall face and between the front and back of the system. Effects of ERS construction on adjacent ground and/or existing structures, utilities and other structures, both above and below ground.
- Permissible net contact stress (service limit state)
- Calculated resistance factor for overall (global) and local stability (service and extreme event limit states). Provide the method of analysis.
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- Foundation improvements required to meet geotechnical design objectives, such as sub-excavation, foundation preloading and surcharge delay periods.
- The minimum embedment depth of the prefabricated modular wall and toe bench width based on erosion, overall stability, bearing resistance and settlement analyses.
- Minimum base width to meet overall stability requirements
- Discuss the groundwater condition anticipated over the design life of the ERS, and if a blanket drain is required. Describe the drainage system location and configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type.
- Slope or batter of the face used in slope modeling, grades and slopes analyzed.
- Corrosiveness of foundation and retained soils and/or rock and water sources or drainage systems.
- Foundation improvements required to meet geotechnical design objectives, such as sub-excavation, foundation preloading and surcharge delay periods.
- Type of crib or configuration of the gabions (such as stepped front, or smooth front and stepped rear).
- Infill considerations, or facing closure recommendations

- Indicate the weight of the mature landscaping included in the ERS modeling and stability calculations.

Foundation recommendations for Geotechnical Services designed gabion and mortarless concrete block walls include all those listed above for prefabricated modular walls, and the following:

- Cross sections showing wall geometry. For gabion walls this includes the thickness and number of baskets, the basket dimensions and the size of the rock infill.
- Wall batter
- Analysis of base sliding
- Active earth pressure coefficient (k_a) and passive earth pressure coefficient (k_p)
- Maximum external loading
- Design criteria for the General notes on the contract plans
- Information for estimating purposes
- Specifications requirements/additions including manufacturers information when necessary

3.12.2.6 Other ERS Technologies

Reports for ERS not previously discussed should include applicable topics listed for any of the ERS discussed here, as well as additional ERS specific information necessary to prepare a design that is in compliance with LRFD specifications.

3.13 Notes for Specifications

Omit this section for the Preliminary Foundation Report.

This section provides recommendations to the Specifications Engineer for inclusion and editing of Standard Special Provisions and developing NSSPs. Refer to the *Geotechnical Notes for Specifications* module for guidance on how to prepare this report section.

Note: This is a new section in this reporting standard. The information placed in this section used to be placed in the “Construction Considerations” section. This updated version of FR for ERS separates communication to the Specifications Engineer (Notes for Specifications) and to Construction (Notes for Construction).

3.14 Notes for Construction

Omit this section for the Preliminary Foundation Report.

Notes for Construction are written to State construction personnel and contractors. Specific geologic conditions that are relevant to construction inspection should be cited in this section to ensure that both the intent of the geotechnical design is met and construction is successful.

Address topics when applicable, such as:

- Recommended construction inspection to be performed by Geotechnical Services personnel
- Instructions to contact Geotechnical Services if a geologic condition is not encountered. For example, the top of rock elevation varies from what was assumed.

3.15 **Report Copy List**

Reports must be addressed to the Structure Designer and copies provided to those listed under Report Distribution in the Communications and Reporting section of the Offices of Geotechnical Design – Quality Management Plan.

3.16 Appendices

The Foundation Report appendices provide detailed information supporting ERS type selection, analyses, and recommendations. Reports prepared by Geotechnical Services staff must include the following (in the order presented, numerated as Appendix I, Appendix II, …), if produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections: Do not include copies of published maps. Limit to 8 ½ x 11.
- Geophysical Test Reports
- Laboratory Test Data (including Corrosion Test Report) – Organized by test type. In addition to the raw laboratory test results, organize and provide the summary tables and graphs of the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for Lateral analysis and/or P-Y Curves

Optional:

- Photos: Relevant to the investigation findings, design recommendations, and construction. Photos that supplement text should be embedded in the report if feasible.
- Photos of Rock Cores

Reports prepared by consultants must include the following (in the order presented, numerated as Appendix I, Appendix II, …):

All Foundation Reports:

- Appendix I: Site Map showing project location
- Appendix II: Log of Test Borings (including as-built LOTB)
- Appendix III: Field Exploration and Testing: Data acquired from field exploration and testing such as surface geologic mapping and surface geophysical surveys, logs from the Cone Penetration Test, Pressuremeter, Dilatometer, and in-situ Vane Shear Tests, Borehole Geophysical logging, indicator pile tests, Piezometer Readings, etc.
- Appendix IV: Calculation Package
    - The objectives of each calculation, such as time rate of settlement or bearing capacity.
    - List assumptions used to simplify the calculation
    - The developed geotechnical model for each calculation
    - The equations used and meaning of the terms used in the equations
    - A copy of the curves or tables used in the calculation and their source or reference.
    - The load and resistance factors, or factor of safety, used for the design
    - If the calculation is performed using computer spreadsheets – step-by-step calculation for one example to demonstrate the basis of the spreadsheet. A computer spreadsheet is not a substitute for the step-by-step calculation.
    - Summary of the calculation results that form the basis of geotechnical recommendations, including a sketch of the design, if appropriate.
- Previous Caltrans review comments and responses

If produced during the investigation:

- Approved "Request for Exception" forms
- Field-generated Geologic Map and Cross-Sections: Do not include copies of published maps. Limit to 8 ½ x 11.
- Photos: Relevant to the investigation findings, design recommendations, and construction. Photos that supplement text should be embedded in the report if feasible.
- Photos of Rock Cores
- Geophysical Test Reports
- Laboratory Test Data (including corrosion) – Organized by test type. In addition to the laboratory test results, summarize and provide the summary tables and graphs of the interpretation of laboratory test results.
- Pile Drivability Study
- Soil parameters for Lateral analysis and/or P-Y Curves

FOUNDATION REPORTS
For
BUILDINGS and MISCELLANEOUS STRUCTURES

January 2020



DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

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1. INTRODUCTION

1.1 Intent of this Document

The intent of this document is to define the Department's standard of practice for preparation of the Foundation Report (FR) for structures requiring foundations at maintenance stations, safety roadside rest areas, CHP truck inspection facilities, toll bridge plazas, and toll operations buildings. The plans package for these facilities including the specifications is prepared by the Office of Transportation Architecture (OTA). Structures that may require design support from Geotechnical Services include buildings, retaining walls, canopies, material storage bunkers, tanks, power generation equipment, light towers and sound walls. Standardized and consistent report presentations benefit the Department's staff, engineering consultants, bidders, and contractors. Geotechnical Services staff as well as any other organization preparing these reports must comply with the requirements presented herein.

This document addresses report content and foundation engineering practice (investigations, design procedures, etc.) that is specific to OTA designed buildings and miscellaneous structures. The structures and facilities addressed by this document are design in accordance with the *California Building Code* (CBC). At the time of the preparation of this document, a digital copy of the 2016 CBC was available at the following website:

https://archive.org/details/publicsafetycode?and[]=title:(2016%20California)

1.2 Reporting for Caltrans Project Delivery

Buildings and miscellaneous structures are most often funded from the Minor A or B Program. As a result, foundation investigations and reporting generally occurs only at the design or 1phase of a project (WBS 240.80). Planning and preliminary design reports are infrequently requested for building facilities. If a Structure Preliminary Geotechnical Report or a Preliminary Foundation Report is requested for a building facility, adhere as closely as possible to the format presented in this document.

1.3 Report Format

A foundation report can present foundation recommendations for all structures located at a facility such as a maintenance station, CHP inspection facility, or toll plaza.

1.3.1 Reports Prepared by Caltrans Staff

Foundation Reports are written to the OTA Designer and Structure Construction. They are also provided to bidders via the Information Handout. Specifications are prepared by the OTA designer.

For reports prepared by Geotechnical Services staff, Foundation Reports must be prepared using the current departmental memorandum format with the subject line of "Foundation Report for *Facility Name*" or "Preliminary Foundation Report for *Facility Name*". Do not use the section numbers in the report. Section titles must be used.

The Log of Test Borings (LOTB) and/or As-built LOTB are not to be submitted as part of the FR. Microstation LOTB files and scanned copies of the As-built LOTB sheets will be sent to the Office of Transportation Architecture for inclusion within the Contract Plans.

Signing and sealing requirements are presented in the *Communications and Reporting* section of the *Offices of Geotechnical Design – Quality Management Plan*.

1.3.2 Reports Prepared by Consultants

Foundation Reports must consist of the following: cover sheet, table of contents, main contents per this document, and appendices. The cover of the report and any addenda/amendments to the report must include the following information: Caltrans District, County, Route (if applicable), Post Mile (if applicable), and Expenditure Authorization (EA) number.

The LOTB and/or As-built LOTB must be submitted as part of the FR. Refer to the *Caltrans Soil and Rock Logging, Classification, and Presentation Manual* for direction on the preparation of the LOTB and As-built LOTB.

2. FOUNDATION REPORT (FR)

The following topics, if applicable, must be addressed in the Foundation Report.

2.1. Scope of Work

Summarize the scope and types of work performed to obtain the information supporting the foundation recommendations. Include a statement that the current report supersedes all previous reports (referenced by title and date).

Example: Foundation Report
Per the request dated January 10, 2016, this Foundation Report has been prepared for the proposed Little City Maintenance Station. The purpose of this report is to summarize the investigations performed and to provide foundation recommendations for the crew building and the vehicle maintenance building at the Little City Maintenance Station. The recommendations presented in this report are based on the site plans dated December 10, 2015, a subsurface investigation, proposed foundation configurations and load demand information provided by the Office of Transportation Architecture.

2.2. Project Description

Describe the proposed buildings and miscellaneous structures. Discuss the previous land use. Discuss if the site has been graded and/or disturbed. Discuss the approximate depth of fill at the location(s) of the proposed structure(s). Provide pertinent project information relating to the planned foundation improvements. The datum used to reference the elevations in the report is also included.

Example
The maintenance station site is located in Little City on State Route 21at PM R3.8. A new crew building and vehicle maintenance building are necessary. All elevations referenced within this report

are based on the North American Vertical Datum of 1988 (NAVD 88), unless otherwise noted. To convert an elevation at this site from National Geodetic Vertical Datum of 1929 (NGVD 29) to NAVD 88, add 2.3 feet to the NGVD 29 elevation.

Based on the Site Plan (dated December 10, 2015) and Foundation Report request, the proposed buildings will be supported by continuous footings founded below the exterior perimeter walls. The superstructure will be masonry and timber. The preferred foundation type is shallow footing.

2.3. Exceptions to Policies and Procedures

Discuss exceptions to Departmental policies and procedures relating to the FR. Approved *Request for Exception* forms must be included in the Appendix.

2.4. Field Investigation and Field Testing Program

Provide a very brief overview of the field investigation performed to support the foundation recommendations, including the number of boreholes, the number of CPT soundings, and a brief description of geophysical testing. The borehole and CPT logs will be provided on a LOTB.

Example
In May 2016, a subsurface investigation was performed consisting of two hollow stem auger borings drilled to a maximum depth of 40.0 feet. Additionally, the As-built LOTB indicates that three borings were drilled to a maximum depth of 45 feet in April 1968.

A rule of thumb for determining the number of boreholes is to drill one borehole per 10,000 square feet of building footprint. Where footings are selected, borehole depths should be a minimum of 25 feet below the bottom of the footing, and extend to a sufficient depth to evaluate settlement and liquefaction. Consider employing SPT sampling at 2.5 foot intervals to a depth of 15 feet below the bottom of the footings. Also consider advancing the boreholes using hollow stem augers so that groundwater observations can be made during and shortly after the drilling of the borehole. Review the *Shallow Foundations for Bridges* section of the Geotechnical Manual for further guidance.

2.5. Laboratory Testing Program

Provide an overview of the laboratory testing program, if performed, to support the foundation recommendations. Briefly explain how the tests contributed to report findings (e.g. soil classification, settlement, strength parameters).

Example
During the most recent field investigation, soil samples for particle analysis and Atterberg limits were collected from borings RC-16-001 and RC-16-002 for soil classification and liquefaction evaluation. The summary of the results will be provided in the Information Handout (referenced in the Special Provisions, Supplemental Project Information section), and the test sample locations will be shown on the Log of Test Borings.

2.6. Site Geology and Subsurface Conditions

Based on the field investigation, provide an updated generalized description of the project site geology and known subsurface conditions.

Do not re-create the As-built LOTB in detail in this section. A generalized discussion is sufficient.

Present only factual information in this section, not how it relates to design and construction. Discussion of the site geology, geological features, and subsurface conditions as they relate to the foundation design and construction must be placed in the *Foundation Recommendations* and *Construction Considerations* sections, respectively.

Example
The maintenance station site is located within the Mojave Desert Geomorphic Province of California. The "Geologic Map of the San Bernardino Quadrangle" (Bortugno and Spittler, 1998) shows that the maintenance station site is underlain by Quaternary alluvium.

During the 2016 investigation, boring RC-16-001 was drilled near the northeast corner of the proposed crew building. Boring RC-16-002 was drilled near the southwest corner of the proposed vehicle maintenance building.

The soil borings revealed that the site is underlain by interbedded layers of predominantly dense to very dense poorly-graded sand with silt and well-graded sand with gravel, cobbles, and boulders, to the maximum depth drilled at the site of 40.0 feet (elev. 2377.9 feet). Soil descriptions from the 2016 subsurface investigation are presented on the Log of Test Borings.

At the northeast corner of the maintenance building (Boring A-16-001), very hard boulders up to 4 feet in size were identified. However, larger boulders up to 8 feet were exposed in the adjacent natural slope.

2.7. Groundwater

Report observed groundwater elevation(s) and date(s) of measurements. Use of a table is recommended if there are numerous borings and/or measurements. Include discussions relating to the presence of wet or saturated soil when groundwater measurements were not made. Discuss surface water conditions that might influence the design or construction of the foundations. The presence of shallow groundwater is of importance for determining the need for a vapor barrier below the slab of occupied space.

State the groundwater elevation(s) used for foundation analyses and design.

Example
As-built LOTB's from the April 1968 subsurface investigation indicate that groundwater was encountered in several borings at that time, and ranged from elevation 2381.9 feet to elevation 2382.2 feet (NAVD88 datum). During the 2016 subsurface investigation groundwater was measured in Boring A-16-001 at elevation 2381.3 feet and in Boring A-16-002 at elevation 2383.9 feet. The groundwater elevation used for design was 2385.0 feet.

2.8. As-built Foundation Data

If not addressed elsewhere in the FR, include discussion of relevant As-built data, such as the type of foundation systems used for pre-existing structures at the site. Include as much specific data as possible: foundation types, elevations, widths and loading. Discuss the potential of encountering buried obstructions such as abandoned foundations and utilities.

2.9. Corrosion Evaluation

Report and discuss pertinent site corrosion data. Corrosion test only soils and groundwater that will be in contact with foundation elements.

Example: No information available
Historical corrosion data is not available. The site should be considered non-corrosive based on the presence of predominantly cohesionless material.

Example: Non-Corrosive
Two soil samples were collected for corrosion testing during the 2016 subsurface investigation. Corrosion test results for those samples are shown below in Table 1. Based on current Caltrans' standards, the site is considered to be non-corrosive.

Table 1: Soil Corrosion Test Summary

Boring ID	*Elevation (ft)*	*Minimum Resistivity (Ohm-Cm)*	*pH*	*Chloride Content (ppm)*	*Sulfate Content (ppm)*
A-16-001	*2409.2 to 2410.7*	*1544*	*7.24*	*N/A*	*N/A*
A-16-002	*2415.3 to 2416.8*	*683*	*7.94*	*384*	*432*

Caltrans currently defines a corrosive environment as an area where the soil has either a chloride concentration of 500 ppm or greater, a sulfate concentration of 1500 ppm or greater, or has a pH of 5.5 or less. Soil and water are not tested for chlorides and sulfates if the minimum resistivity is greater than 1,000 ohm-cm.

Example: Corrosive
During the 2016 subsurface investigation four soil samples were collected for corrosion testing. Corrosion test results for the samples collected from borings A-16-001 and A-16-002 are shown below in Table 1. Due to chloride content being greater than 500 ppm in two of the samples tested, the site is considered to be corrosive based on current Caltrans' standards, and corrosion mitigation is required.

2.10. Seismic Design Information and Recommendations

Seismic design and site seismicity are determined per California Building Code (CBC) Section 1613. This section utilizes ASCE 7, *Minimum Design Loads for Buildings and Other Structures*. Geotechnical Services provides OTA the soil profile Site Classification, and the spectral accelerations for the short or 0.2 second period (S_S) and 1 second period (S_1). The Foundation Report should also provide the seismic parameters that are derived from the short period and 1 second period spectral accelerations per the CBC, S_{MS}, S_{M1}, S_{DS} and S_{D1}. These parameters are determined per the formula

provided in CBC Sections 1613.3.3 and 1613.3.4, and as shown later in this section. The developed data should be provided in the Foundation Report as shown below, in Table 2.

Table 2: Maximum Considered Earthquake Response Spectral Parameters

S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}
1.580	*0.562*	*1.580*	*0.977*	*1.053*	*0.651*

Step 1: The calculations require the soil profile at a project location be assigned a Site Classification. Per Section 1613.3.2 of the CBC, Site Classifications are determined in accordance with ASCE chapter 20. The following Table provides the Site Classifications from ASCE 7. Provide the Site Classification in the Foundation Report.

Table 3: Site Classification

Site Class	Weighted Average of V_S	Weighted Average of N or Ns	Weighted Average of S_u
A. Hard rock	>5,000 ft/sec	NA	NA
B. Rock	2,500 to 5,000 ft/sec	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/sec	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/sec	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/sec	<15	<1,000 psf
	Any profile with more than 10 feet of soil having the following characteristics: • Plasticity index greater than 20 • Moisture content greater than or equal to 40%, and • Undrained shear strength less than 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1 of ASCE7	Liquefiable soils, peat, high plasticity clay		

The weighted average values used in Table 3 are for the 100 feet of the soil or rock immediately below the footings. If subsurface data is not available to this depth, use judgement to extrapolate or infer appropriate values. The site class can be determined by one of three methods: 1) using the shear wave velocity, 2) SPT blow count corrected to 60% hammer energy, or 3) a combination of the undrained shear wave velocity for cohesive soils and the SPT blow count for cohesionless soils.

The equation for calculating the weighted average of V_S for the upper 100 feet is:

$$V_S = \frac{100 \text{ feet}}{\frac{D_1}{V_1} + \frac{D_2}{V_2} + \frac{D_3}{V_3} + \ldots + \frac{D_n}{V_n}}$$

; Where D is the layer thickness (feet) and V is the shear wave velocity (feet/sec) for that layer.

The equation for calculating the weighted average of N for the upper 100 feet is:

$$N = \frac{100 \text{ feet}}{\frac{D_1}{N_1} + \frac{D_2}{N_2} + \frac{D_3}{N_3} + \ldots + \frac{D_n}{N_n}}$$

; Where D is the layer thickness (feet) and N is the SPT blow count corrected for hammer energy for that layer.

When using the undrained shear strengths of cohesive soil to determine the site classification, use the following equation for calculating the weighted average of S_u for the portion of the upper 100 feet of the soil column that is cohesive soil:

$$S_u = \frac{d_c \text{ feet}}{\frac{D_1}{S_{u1}} + \frac{D_2}{S_{u2}} + \frac{D_3}{S_{u3}} + \ldots + \frac{D_n}{S_{un}}}$$

; Where d_c is the total thickness of cohesive soil layers, D is the layer thickness (feet) and S_u is the undrained shear strength (psf) for that layer (not to exceed 5,000 psf).

Use the following equation for calculating the weighted average of N for the portion of the upper 100 feet of the soil column that is cohesionless soil:

$$N_S = \frac{d_s \text{ feet}}{\frac{D_1}{N_{s1}} + \frac{D_2}{N_{s2}} + \frac{D_3}{N_{s3}} + \ldots + \frac{D_n}{N_{sn}}}$$

; Where d_s is the total thickness of cohesionless soil layers, D is the layer thickness (feet) and Ns is the SPT blow count corrected for hammer energy for that layer.

Step 2: Spectral accelerations for the 0.2 second or short period (S_S), and 1 second period (S_1) are determined from CBC Section 1613.3.1 (Figures 1613.3.1(1) and 1613.3.1(2)). For most locations in California, these figures are not of sufficient clarity to determine the S_S and S_1 coefficients. It is recommended that the URL below be used to access the OSHPD website to determine S_S and S_1. While this website does calculate all of the parameters shown in Table 2, the seismic parameters generated on the website should be checked against hand calculations following the methods provided in CBC Sections 1613.3.3 and 1613.3.4, as shown below.

https://seismicmaps.org/

The calculator on the website requires that the project location be entered either by coordinates or by locating the project location on a map. Enter the appropriate design code reference document. This is tied to the version of the CBC that OTA is using for design of the structure. For the 2016 edition of the CBC the reference document is ASCE 7-10. The Risk Category for occupied maintenance structures is considered to be Risk Category II. This is the category that applies to residential, commercial, and industrial buildings.

Step 3: Calculate the maximum considered earthquake response acceleration for short periods (S_{MS}) and at 1-second periods (S_{M1}) per the following equations and Tables.

$S_{MS} = F_a * S_s$

$S_{M1} = F_v * S_1$

Where:

F_a = Site coefficient defined in Table 4

F_v = Site coefficient defined in Table 5

S_s = The mapped spectral acceleration for short periods.

S_1 = The mapped spectral acceleration for a 1-second period.

Table 4: Values of Site Coefficient F_a

Site Class	Mapped Spectral Accelerations at Short Period				
	S_s <= 0.25	S_s = 0.50	S_s = 0.75	S_s = 1.00	S_s >= 1.25
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at short period Ss
b. Values shall be determined in accordance with Section 11.4.7 of ASCE 7.

Table 5: Values of Site Coefficient F_v

Site Class	Mapped Spectral Accelerations at 1-Second Period				
	S_s <= 0.1	S_s = 0.2	S_s = 0.3	S_s = 0.4	S_s >= 0.5
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	Note b	Note b	Note b	Note b	Note b

a. Use straight-line interpolation for intermediate values of mapped spectral response acceleration at 1-second period S_1
b. Values must be determined in accordance with Section 11.4.7 of ASCE 7.

Step 4: Calculate the 5 percent damped design spectral response acceleration at short periods (S_{DS}) and at 1-second period (S_{D1}) per the following equations.

$S_{DS} = 2/3 * S_{Ms}$

$S_{D1} = 2/3 * S_{M1}$

Where:

S_{Ms} = The maximum considered earthquake spectral response accelerations for short period.

S_{M1} = The maximum considered earthquake spectral response accelerations for 1-second period.

In the Foundation Report, provide seismic hazards for the proposed foundations (e.g., fault rupture evaluation, liquefaction and seismic settlement evaluation). Provide mitigation measures for seismic hazards. For deep foundations and upon request, provide p-y and t-z curves.

Example: Liquefaction
Due to the presence of loose to medium dense alluvial material and high ground water beneath the site, the potential for soil liquefaction under strong ground shaking is present. The liquefiable zone elevations at locations within the project limits are provided below in Table 6.

Table 6: Liquefaction Potential at Little City Maintenance Station

Location	*Liquefaction Elevation (ft)*	*Estimated Seismic-induced Settlement (in)*
NE corner of crew building	*Elev. 20 to 15 and Elev. 0 to -5*	*3*
Center of crew building	*Elev. 10 to -5*	*4*
SW corner of crew building	*Elev. 20 to 10*	*3*

Example: Lateral Spreading Potential
It is anticipated that lateral spreading may occur at the proposed location of the crew building. Mitigation of this settlement may include a deep foundation consisting of piles or ground improvement.

Example: Seismic Settlement
Liquefaction-induced settlement of the ground surface is estimated to range from 3 to 5 inches. Mitigation of this settlement may include a deep foundation consisting of piles or ground improvement.

2.11. Foundation Recommendations

Building and miscellaneous structure foundations are designed per the California Building Code Chapter 18. Calculations are performed using Allowable Stress Design Methodology. Provide complete and concise foundation recommendations by addressing the topics in the applicable portions of this section. Discuss the recommended foundations and any special considerations which influence their design and selection (ground improvement, liquefaction, etc.).

Example: Shallow Foundations
The following recommendations are for the proposed Little City Maintenance Station, as shown on the Site Plan dated March 14, 2016. At the Crew Building and the Vehicle Maintenance Building, continuous footings are recommended. The subsurface information gathered for the site indicates that the footings will be founded in alluvial soil. The following foundation recommendations were designed in accordance with the 2016 California Building Code.

Example: Deep Foundations
The following recommendations are for the proposed Little City Maintenance Station, as shown on the Site Plan dated March 14, 2016. Based on the subsurface information gathered at the site, driven precast concrete piles (Alt. "X") are recommended at both the Crew Building and Vehicle Maintenance Building. The following foundation recommendations were designed in accordance with the 2016 California Building Code.

2.11.1 Spread Footings and Continuous Footings

While the CBC provides *Presumptive Load Bearing Values* in Table 1806.2, it is Caltrans practice that a site investigation be conducted and that the gross nominal bearing resistance be computed in accordance with the analytical methods provided in the Geotechnical Manual Section *Shallow Foundations for Bridges*. The allowable soil pressure is computed by applying a Factor of Safety of 3 to the gross nominal bearing resistance. For shallow foundations, spread footings and continuous (strip) footings, OTA requires allowable soil pressures in units of psf.

Present and/or discuss the following:

1. A description of the material on which the footing is to be placed.
2. The soil or rock strength parameters and unit weight used for the bearing resistance calculations.
3. The minimum footing embedment required for the recommendations to be valid. The CBC Section 1809.4 specifies the minimum embedment into undisturbed material must be 12 inches. Bearing resistance, frost penetration and erosion considerations influence this recommendation.
4. Summary of foundation elevations or range of elevations, allowable soil pressures and footing sizes. The CBC Section 1809.4 specifies the minimum footing width must be 12 inches.
5. The influence of new footing construction on the adjacent structures and/or utilities, if applicable.

Report the footing allowable soil pressure recommendations in the following table:

Table 7: Footing Design Recommendations for Crew Building

Footing Type	*Footing Width (feet)*	*Minimum footing embedment below grade (feet)*	*Approximate Footing Elevation (feet)*	*Ultimate Soil Pressure (psf)*	*Allowable Soil Pressure (psf)*
Continuous	*2.0*	*1.5*	*7.24*	*3300*	*1100*
Continuous	*3.0*	*2.0*	*6.74*	*4200*	*1400*
Square	*6.0*	*2.0*	*6.74*	*5100*	*1700*
Square	*8.0*	*2.0*	*6.74*	*6600*	*2200*

Section 1806.1 of the CBC states that the presumptive *vertical foundation pressure* provided in Table 1806.2 may be increased for load combinations that include short term or transient loads used in the alternative basic load combinations of Section 1605.3.2. The permitted increase is 33%. It is interpreted that the same provision applies to allowable soil pressure values calculated from site specific foundation data. Reiterate this allowance in the Foundation Report.

Perform total and differential settlement calculations for building structures in conformance with the methods outlined in the Geotechnical Manual Section *Shallow Foundations for Bridges*. Settlement calculations should be performed using the Service Loads. Present the immediate settlement and long term settlement values.

Lateral load demands on footings are resisted by a combination of friction along the bottom of the footings and lateral passive bearing pressure along footing vertical surfaces. Report the coefficient of friction for sliding of the foundation along subgrade soils per Section 1806.3. The coefficient of friction values provided in Table 1806.2 are presumptive, and may be exceeded if data from a field investigation supports doing so. Also consult Section 3.10.1 of FHWA NHI-01-023, *Shallow Foundations* for guidance to determine the lateral sliding resistance.

Presumptive values of lateral bearing pressure are provided in Table 1806.2 of the CBC. The presumptive lateral bearing pressure values provided in Table 1806.2 are conservative. Passive lateral earth pressures can be computed using Rankine earth pressure theory, however the results may yield unconservative designs and should be used with caution because the displacement required to achieve these pressures is excessive. In order to limit the lateral movement of shallow foundations to tolerable values, the recommended lateral passive bearing pressure values are less than passive lateral earth pressures. It is recommended that a 50% reduction be considered. If the possibility exists that the soil adjacent to the foundation will be excavated in the future, no lateral bearing pressure should be used to resist lateral loads.

Section 1806.1 of the CBC states that the presumptive *lateral bearing pressure* provided in Table 1806.2 may be increased for load combinations that include short term or transient loads used in the alternative basic load combinations of Section 1605.3.2. The permitted increase is 33%. It is interpreted that the same provision applies to lateral passive bearing pressure values calculated from site specific foundation data. Reiterate this allowance in the Foundation Report.

Per CBC Section 1809.5, unless founded on rock, footings are either founded at depths below frost penetration or are constructed in compliance with ASCE 32. Frost depth is determined by consulting the local building codes. Observations and knowledge of frost depth from local practice at the project site, are the best sources of information. In general, penetration of frost into the ground is a potential design factor in Northern California, the Sierra Nevada Mountains and east of the Sierra Nevada Mountains (FHWA NHI-01-023, *Shallow Foundations*, Section 2.5.1). Conditions for the exception to this specification are found in CBC Section 1809.5.

The presence of expansive soil beneath footings should be investigated and analyzed. Building footings are relatively lightly loaded and may be susceptible to damage caused by expanding and contracting foundation soils. If evaluation of the uplift pressure indicates that it exceeds the contact stress resulting from the service loads on the footings, mitigation should be considered. The first strategy that should be considered is taking measures to maintain constant moisture content in the soils beneath the footings. Other typical mitigation measures include remove-and-replace the expansive soil, and treatment-in-place to eliminate the expansion potential.

Notes to the Structure Designer (Shallow Foundations)

Provide the following (applicable) information to aid in preparation of the contract plans.

1. Footings below groundwater level.

Example
Groundwater will be encountered during construction of the continuous footings at the location of the proposed Crew Building. Dewatering is anticipated for construction.

2. Footings with sub-excavation and replacement with structure backfill.

Example
At the Vehicle Maintenance Building, unsuitable native soils were identified in the subsurface investigation and possibly underlie the proposed continuous footing. Therefore, it is recommended that the native materials be removed to a depth of 2.0 feet below the bottom of footing, and be replaced with structure backfill compacted to 95% relative compaction, or concrete to the bottom of footing elevation. The bottom of sub-excavation elevations are listed in Table 1. The limits of the sub-excavation and replacement must conform to the limits specified in Standard Specification 19-5.03B for compaction of embankments under retaining wall footings without pile foundations.

Table 8: Little City Maintenance Station – Bottom of Sub-Excavation Elevation

Support Location	*Bottom of Sub-Excavation Elevation (ft)*
Vehicle Maintenance Building	*4334.4*

3. Location of footings adjacent to existing footing or utilities

Example
Avoid transferring load from proposed foundations to existing foundations by configuring their bearing surfaces below an imaginary 1.5 horizontal to 1 vertical plane that is projected upward from the bearing surface of the existing footing.

Construction Considerations (Shallow Foundations)

Provide the following (applicable) information in this section for use by the OTA Design Engineer, Structure Construction Representative, Contractor, and others.

1. Include the following instructions to address potential construction disturbance of native material below the specified bottom of footing elevation(s).

Example
At the Crew Building, the continuous footings are to be constructed on native alluvium at the bottom of the excavation. The structural concrete is to be placed neat against the undisturbed native alluvium at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed to a depth of 1.0 feet below the disturbance, and recompacted to 95% relative compaction.

Example
At the Crew Building, the continuous footings are to be constructed on the weathered rock at the bottom of the excavation. The structural concrete is to be placed neat against the trimmed walls and undisturbed rock at the bottom of the footing excavation. Should the bottom of the footing excavation be disturbed, then the disturbed material must be removed and replaced to the bottom of footing elevation with concrete.

Example
Gravel, cobbles and boulders were observed in the foundation soils. Excavations for the continuous footings may encounter gravel, cobbles and boulders that extend beyond the planned limits of the footings. All soils disturbed during excavations are to be removed and replaced with structure backfill compacted to 95% relative compaction.

2. Include instructions for the removal of obstructions, and their replacement with lean concrete or structure backfill compacted to 95% relative compaction.

2.11.2 Deep Foundations

Deep foundations consisting of driven piles are rarely warranted for building foundation use. The use of CIDH piles is extremely rare. Building and canopy load demands are low when compared to bridge structures, therefore adequate bearing resistance can at most locations be provided with continuous (strip) and spread footings. Settlement behavior may warrant deep foundations, particularly when differential settlement with relationship to adjoining or connecting features that have minimal foundation loads and therefore settle very little. This includes utilities, walkways and driveways.

If the design team determines that the expense of deep foundations is warranted for performance and maintenance reasons, the Geotechnical Designer should compute deep foundation recommendations with the methods outlined in the Geotechnical Manual Sections *Driven Pile Foundations and CIDH Pile Foundations*. However, deep foundations for buildings are designed utilizing the CBC which specifies that Allowable Stress Design methodology is used. Therefore, the allowable bearing load or pile design load is determined by applying an appropriate Factor of Safety to the calculated pile ultimate or nominal bearing resistance. A Factor of Safety value of 2 is typically utilized. CBC Section 1810.3.3.1.7 specifies that a minimum value of 2 is used for the Factor of Safety.

Pile design tip elevations for settlement are calculated per the Geotechnical Manual Sections *Foundation Reports for Bridges, Driven Pile Foundations* and *CIDH Pile Foundations.* If site conditions will result in more than 0.25 inch differential settlement for the specified tip elevations report this value to the OTA designer.

Piles may be subject to lateral loads, particularly if they are not fully embedded. If the upper portions of piles are unbraced in air or water, or the upper portions of the piles are embedded in weak soils, the OTA designer may request assistance with a lateral pile analysis. Often the OTA engineer will determine the design tip for lateral load, and include it in the plans. Consult the CBC for lateral load and resistance requirements.

Also consider foundation conditions that may impact pile design and performance such as downdrag and liquefaction.

The following Foundation Recommendations Table and Pile Data Table are included in the Foundation Report. These tables have been modified from those found in Memos to Designers 3-1 to reflect Allowable Stress Design practice, not LRFD.

Table 9: Little City Maintenance Station – Foundation Design Recommendations

Foundations Design Recommendations									
Location	Pile Type	Cut-off Elevation (ft)	Service Load per Pile (kips)	Total Permissible Pile Settlement (inches)	Nominal Resistance (kips)		Design Tip Elevations (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance Required (kips)
					Compression (FS = 2.0)	Tension (FS = 2.0)			
Crew Building	Class 90 Alt."X"	4.8	60	1	120	20	-30 (a) -15(b) -20(c)	-30	120

Notes:

1) *Design tip elevations are controlled by: (a) Compression, (b) Tension, (c) Settlement, and (d) Lateral Load, respectively.*
2) *The specified tip elevation must not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*
3) *The nominal driving resistance required is equal to the nominal resistance needed to support the factored load plus driving resistance from the unsuitable penetrated soil layers (very soft, liquefiable, scourable, etc.), if any, which do not contribute to the design resistance. Unsuitable soil layers extend to elevation -10 ft.*
4) *Design tip elevation for Lateral Load is typically provided by OTA*

Table 10: Little City Maintenance Station – Pile Data Table

Pile Data Table						
Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (ft)	Specified Tip Elevation (ft)	Nominal Driving Resistance (kips)
		Compression	Tension			
Crew Building	Class 90 Alt. "X"	120	20	-30(a) -15(b) -20(c)	-30	120

Notes:

1) *Design tip elevations for are controlled by: (a) Compression, (c) Settlement, (d) Lateral Load*
2) *The specified tip elevation must not be raised above the design tip elevations for Tension, Settlement, and Lateral Load.*
3) *Unsuitable soil layers (very soft, liquefiable, scourable, etc.) that do not contribute to the design nominal resistance exist to elevation -10 ft.*

Incorporate appropriate report content that follows the guidance in the *Notes to the Designer* and *Construction Considerations* Sections of the Geotechnical Manual document *Foundations Reports for Bridges*. This includes considerations for pile modification from the standard plan details, pile acceptance criteria, and predrilling limits.

2.11.3 Concrete Slabs on Grade

Building floor slabs are lightly loaded and rarely warrant sub-excavation and material replacement beneath the floor slab. Sub-excavation beneath slabs should rarely be recommended. If recommended, provide detailed justification.

State the proposed bottom of slab elevation, and provide the modulus of subgrade reaction for slab foundation soils. The first step to determining the modulus of subgrade reaction is to determine the representative subgrade modulus (k) for a 12 inch plate load test using the following table (Lindeburg, Civil Engineering Reference manual, 15th Edition). Select a k value from the table, where higher values within each range are appropriate for stiffer soils.

Table 11: 12 inch plate subgrade modulus

Group symbol	Range of subgrade modulus, k (psi/in)
GW	300-500
GP	250-400
GM	100-400
GC	100-300
SW	200-300
SP	200-300
SM	100-300
SM-SC	100-300
SC	100-300
ML	100-200
CL	50-200
OL	50-100
MH	50-100
CH	50-150
OH	25-100

Modify the k value to reflect the dimensions of the foundation. Using the following formulae, determine and provide in the Foundation Report the appropriate subgrade modulus: k_s, $k_{s,rect}$ or $k_{s,cont}$.

- Square footings or floor slabs $k_s = k*(((B+1)/(2*B))^2)$
- Rectangular footings or floor slabs $k_{s,rect} = k_s*((1+(B/L))/1.5)$
- Continuous footings $k_{s,cont.} = 0.67* k_s$

Where:

B = the foundation width

L = the length of a rectangular foundation

The recommendation for a vapor barrier beneath occupied buildings is provided by Geotechnical Services (GS) in the Foundation Report. This will be based on the predicted height of ground water and the height of capillary rise beneath the slabs of occupied buildings. If it's anticipated that moisture can reach the bottom of the floor slab, then a vapor barrier should be recommended. OTA will specify the vapor barrier configuration and material that is in conventional use at that time. OTA also has a standard practice and detail for base courses that are placed beneath concrete slabs.

The presence of expansive soil beneath slabs should be investigated and analyzed. Expansive soils meet the characteristics set forth in CBC Section 1803.5.3. Soils having an expansion index greater than 20 as determined by ASTM D4829 are considered expansive. Also consult Section 6.2 of FHWA NHI-01-023, *Shallow Foundations*. Floor slabs are lightly loaded and susceptible to damage caused by expanding and contracting foundation soils. If evaluation of the uplift pressure indicates that it exceeds the contact stress resulting from the dead weight of the slab, mitigation should be considered. The first strategy that should be considered is taking measures to maintain constant moisture content in the soils beneath the slab. Other typical mitigation measures include pre-saturation, remove-and-replace the expansive soil and chemical treatment to eliminate the expansion potential.

2.11.4 Retaining Walls and Earth Retaining Structures (ERS)

Retaining walls employed in non-roadway settings do not typically utilize the Standard Plan concrete cantilever retaining wall design details. The Office of Transportation Architecture (OTA) designer will most often start with the standard plan wall concrete cantilever design as a template, and modify the design with the intent of economizing. This is achieved by not designing the retaining wall for loads that are not applicable, such as traffic surcharges. The OTA designer requests that the Geotechnical Designer provide *lateral soil loads*. Table 1610.1 of Section 1610 of the CBC is used to determine the *lateral soil load*, at-rest or active, whichever is applicable. The results of the geotechnical investigation can be also used to determine the *lateral soil load*. The values provided in Table 1610.1 apply to moist drained soils. If retaining wall backfill is undrained, then the retaining wall is designed for hydrostatic pressure in addition to the buoyant soil being retained. Lateral pressures resulting from expansive soils must also be identified. Retaining wall and ERS foundation calculations and recommendations follow the guidelines provided in the Geotechnical Manual sections for the specific wall type and *Foundations Reports for ERS.*

2.11.5 Site Grading

If requested, the Foundation Report provides recommendations for cut slope and fill slope inclinations. If the site requires the placement of more than 3 feet of fill at locations of proposed occupied structures, embankment caused settlement should be analyzed. Recommendations for settlement periods will be the most common mitigation measure to address predicted unacceptable settlement and differential settlement. Fill surcharge may be considered in rare cases.

Provide recommendations as to the suitability of on-site materials for use as structure backfill.

2.11.6 Radio Communications Towers and Infrastructure

The equipment for radio communications are purchased as a package from the manufacturer. A prescriptive foundation design is included with the equipment. OTA will check the design of the foundation elements. In order to check the communications tower foundation and lateral load design, OTA may request that GS provide bearing resistances and lateral soil resistances. Compute passive lateral earth pressure distributions using Rankine Earth Pressure theory. Guidance can be found in the Geotechnical Manual Section for Non-gravity Cantilever Retaining Walls. Presumptive *lateral bearing pressure* values are provided in CBC Section 1806, Table 1806.2. Per Section 1806.3.4, the table values can be doubled for short-term loads if 0.5 inch displacements do not adversely affect performance or maintenance.

2.12. Additional Considerations

Provide any recommendations that have not already been addressed in any of the preceding sections.

2.13. Supplemental Project Information

Standard Special Provision 2-1.06B, "Supplemental Project Information", discloses to bidders and contractors a list and location of pertinent information available for their inspection prior to bid opening. Documents and information are presented in the following table:

Table 12: Supplemental Project Information

Means	*Description*
Included in the Information Handout	*Foundation Report for Little City Maintenance Station* *Lab Test Results for Little City Maintenance Station* *Cone Penetration Test Results for Little City Maintenance Station*
Available for inspection at the District Office	
Available for inspection at the Transportation Laboratory	

The Geoprofessional must identify and forward all pertinent geotechnical information (in PDF) to the OTA Designer so that it can be properly included in the construction contract via SSP 2-1.06B. Include all relevant geotechnical reports and laboratory test results. Items available for inspection at the Transportation Laboratory or District Office are typically core samples.

2.14. Report Copy List

The FR must be addressed to the OTA Designer and copies provided to those listed under Report Distribution in the *Communications and Reporting* section of the Geotechnical Design Quality Management Plan, with the exception that the report is not distributed to the Structures Office Engineer.

2.15. Appendices

The Foundation Report appendices provide detailed information supporting foundation type selection, analyses, recommendations, and construction considerations.

Reports prepared by Geotechnical Services staff must include:

Appendix I: Exceptions to Policy

- Attach all approved "Request for Exception" forms.

Reports prepared by consultants must include:

Appendix I: Site Map showing project location
Appendix II: Log of Test Borings (including as-built LOTB)
Appendix III: Field Exploration and Testing

- Data acquired from field exploration and testing such as surface geologic mapping and surface geophysical surveys, logs from the Cone Penetration Test, Pressuremeter, Dilatometer, and in-situ Vane Shear Tests, Borehole Geophysical logging, Piezometer Readings, etc.

Appendix IV: Laboratory Test Results

- Soil and rock laboratory test results.

- Corrosion test results.

Appendix V: Analyses and Calculations.

- Engineering analyses and calculations supporting the foundation recommendations including all QC/QA signature sheets.

Appendix VI: Exceptions to Policy

- Attach all approved "Request for Exception" forms.

Geotechnical Design Reports

February 2021



DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES

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INTRODUCTION

This document provides the standards for the District Preliminary Geotechnical Report (DPGR), Preliminary Geotechnical Design Report (PGDR), and the Geotechnical Design Report (GDR). DPGR, PGDR, and GDR are issued for geotechnical components involving Caltrans assets other than bridge foundations and special design earth retaining systems.

GEOTECHNICAL REPORTING FOR CALTRANS PROJECT DELIVERY

Geotechnical reports are produced in the following phases:

- **Phases K and 0**

  DPGR *(WBS 150.15.20 or WBS 160.10.80)* – The DPGR provides preliminary recommendations to District Project Engineers in Design and Planning and is used to develop the Project Initiation Document (PID), Environmental Impact Report (EIR), Environmental Impact Statement (EIS), Project Study Report (PSR), and/or Project Report (PR).

  PGDR *(WBS 160.10.82)* – The PGDR provides preliminary geotechnical design recommendations to District Project Engineers. The PGDR describes existing site conditions and provides preliminary geotechnical recommendations before the end of Project Approval and Environmental Document (PA&ED).

- **Phase 1**

  GDR *(WBS 230.05.70.15)* – The GDR provides subsurface information from the geotechnical investigation, analysis and design, recommendations, and notes for editing the Special Provisions.

GENERAL CONTENT AND PRACTICE REQUIREMENTS

Technical Content

DPGR, PGDR, and GDR must meet the requirements identified in this document and the following FHWA checklist and guidelines:

- FHWA 2003 ED-88-053, *Checklist and Guidelines for Review of Geotechnical Reports and Preliminary Plans and Specifications*
- FHWA 2016 GEC 014, *Assuring Quality in Geotechnical Reporting Documents*

Identify Special-Design Project Components

Throughout project development, continually verify whether a project component meets the design parameters shown on the Standard Plans. When a project component does not satisfy the design parameters shown on the Standard Plans, notify the Project Engineer that the project component needs to be special designed.

For Earth Retaining Systems (ERS), verify whether the horizontal seismic coefficient of the site does not exceed the standard design parameters. For sign post foundations and

sound wall foundations, verify whether the subsurface engineering properties and ground geometry satisfy the design parameters as shown on the Standard Plans.

Review Project Documents and Contract Package

For DPGR and PGDR, review project documents and plans to ensure the information presented in the report is consistent with project documents and plans before issuing the report.

For GDR, review the draft Plans and Special Provisions, to ensure geotechnical recommendations provided in the GDR have been incorporated and information provided in the GDR is consistent with the contract Plans and Special Provisions. There must be no inconsistencies between the GDR, Plans, and Special Provisions, because the GDR is part of the contract package.

Report Signing and Stamping

For geotechnical design performed by Caltrans, sign and stamp the reports following *Communication and Reporting section* of the *Offices of Geotechnical Design – Quality Management Plan*.

Report Format

For DPGR and PGDR, organize the report as follows: *Introduction, Geotechnical Investigation, Geotechnical Conditions, Geotechnical Design Evaluation, Recommendations, and References (Optional)*.

For GDR, organize the report as follows: *Introduction, Geotechnical Investigation, Geotechnical Conditions, Analysis and Design, Recommendations, Notes for Specifications, Notes for Construction (Reserved), and References (Optional)*. Section Numbering is not required.

Include subsections and the subsection titles outlined in this document when presenting corresponding information.

Not every topic covered in this document is required to be included in a report. A report may be only two or three pages, or may be tens of pages, depending on the scope and complexity of the project. To address project-specific unique design issues that are not covered in this document, add additional sections or subsections.

A report must provide relevant and useful geotechnical design information for the Project Engineer and the Specifications Engineer to produce the Plans, Specifications and Estimates (PS&E) package. When presenting the results of an analysis or design, include geotechnical models, diagrams and photos showing essential design and geotechnical features to make the presentation self-contained.

For geotechnical design performed by Caltrans, prepare the report using current departmental memorandum format with the subject line of "*Report Name for Structure/Project/location Name*", for example "Geotechnical Design Report for Highway 5 Widening", "Preliminary Geotechnical Design Report for …", or "District Preliminary Geotechnical Report for …". Refer to *Communication and Reporting* section of *Offices of Geotechnical Design – Quality Management Plan* for details. The reports should include

project name, District, County, Route, begin and end Post Miles, Project ID, Expenditure Authorization (EA) number, project component name, author, and date.

Geotechnical Project File Content

Geotechnical Design Performed by Caltrans

Caltrans PS&E package should include only one set of subsurface information, either in the Boring Records format as an appendix in the report, or in the Log of Test Boring (LOTB) format in the Plans. To prevent potential inconsistency in the contract, a PS&E package must not include both the Boring Records and LOTBs.

Include Boring Records in the appendix of the Geotechnical Design Report when LOTBs are not included in the Plans. Do not include LOTBs or As-built LOTBs in the Geotechnical Design Report. For As-built LOTB sheets, send the MicroStation files or scanned copies of the As-built LOTB sheets to the Project Engineer to be included in the Contract Plans.

Geotechnical Design Performed by Consultant

The report must consist of the following: cover sheet, table of contents, main contents per this document, and appendices. The cover of the report and any addenda/amendments to the report must include the following information: project name, Caltrans District, County, Route, begin and end Post Miles, Project ID, Expenditure Authorization (EA) number, project component ID, project component name, author, and date.

Include calculation package in the appendix of the GDR.

Submit either the Boring Records, or the LOTBs and As-built LOTBs as part of the GDR. Refer to the Caltrans *Soil and Rock Logging, Classification, and Presentation Manual* for direction on the preparation of the LOTBs and As-built LOTBs.

Submit electronic copy of cone penetration test (CPT) data (both original and interpreted) and graphs in PDF format.

Exceptions

For geotechnical design performed by Caltrans, exception to the standards presented in this document requires prior approval under *Offices of Geotechnical Design – Quality Management Plan*.

DISTRICT PRELIMINARY GEOTECHNICAL REPORT AND PRELIMINARY GEOTECHNICAL DESIGN REPORT

The main function of a DPGR is to provide preliminary geotechnical design and recommendations. A DPGR typically includes evaluation of geologic hazards, existing site conditions, seismicity, and feasibility of identified geotechnical options. The assessment and recommendations provided in the DPGR can influence the scope, cost, and selection of project components.

Caltrans project deliverables that require a DPGR as supporting document are the PID, EIR/EIS, and the PSR/PR.

The main function of a PGDR is to present evaluation of project alternatives under consideration for project scoping and cost estimating, and geotechnical design considerations.

A PGDR typically provides geotechnical input for:

- Evaluation of geologic hazards, existing site conditions, seismicity, geotechnical options and feasibility study
- Assessment of emergency repair needs for landslide, rockfall, and scour
- Preliminary geotechnical analysis of project components

The expected objective, scope, and content of each DPGR and PGDR depend on the nature of each project. Adapt the scope and content of DPGR and PGDR to the specific needs of the project.

Cover the following subjects in the DPGR and PGDR. When a partial or complete geotechnical investigation has been performed, include applicable sections of the GDR in the DPGR and PGDR.

1 INTRODUCTION

Include a concise introduction of the project and its location on the State Highway System. State the type and purpose of the report. Refer to the request or contract received for the preparation of the report. Provide additional information relevant to the background of the project.

1.1 Project Description

Provide a brief description of the project based on information supplied in the Project Report. Include Project Datum reference, and a Vicinity Map showing the project site locations.

Describe the components of the project that require geotechnical information and recommendations. Maps, drawings, or plans are effective in showing the locations and alignments of the project components.

Use tables to provide summary information for project components, such as: the begin and end of slope, sound wall, and ERS alignment, or the location of the culvert, sign post, and infiltration basin. Refer to Table 1 as an example.

Table 1 – Description of the Proposed Slope, ERS, etc.

ID No.	Slope or ERS Type	Begin	End	Length (feet)	Maximum Design Height (feet)	Notes
		Sta. or PM	Sta. or PM			

When presenting locations based on station and offset, reference the highway or project alignment, instead of individual component alignments. For project components that traverse the highway, such as culverts, reference the component alignment.

1.2 Exception to Policy

List exceptions to Departmental policies and procedures relating to the DPGR, or PGDR. Include the approved Request for Exception form(s) in the appendix.

2 GEOTECHNICAL INVESTIGATION

Summarize existing information, including as-built plans, information from previous geotechnical investigations, existing reports, published geologic reports and maps.

Describe geotechnical, geologic, and any site reconnaissance performed.

3 GEOTECHNICAL CONDITIONS

Describe site geology, surface, and geotechnical conditions that affect the geotechnical design of project components based on the results of the geotechnical investigation.

3.1 Geology

Describe project site geology and known subsurface conditions. Include:

- Description of the regional geologic settings relevant to the project
- General geology of the site relevant to the project
- Geologic hazards – landslide or slope failure, rockfall, or debris flow at the site
- Expansive, collapsible, or other unsuitable materials
- Geologic maps showing:
  - Relevant geologic features, such as faults, bedding, major joint attitudes, and folds
  - Locations of geologic cross-sections
  - Layout line of project components

3.2 Topsoil – Soil Survey Review

Describe the soil survey review performed and the assessment. Provide a list of soil survey reports and maps reviewed and the base map used, such as the maps of California Soil/Vegetation Survey, or USDA Soil Survey. When the maps describe the soils as erodible, provide the depth and lateral distribution of the erodible soil, and engineering classification.

3.3 Surface Conditions

Describe site topography, surface water and drainage conditions, significant natural features, and land use history. Describe the performance of existing cut, fill, and natural slopes. Describe soil erosion and scour, and historical maintenance issues, that may affect the proposed project components.

Identify existing and proposed above-ground structures, facilities, and utilities near each proposed project component that may affect the design and construction of the project component.

3.4 Subsurface Conditions

Summarize subsurface conditions based on information obtained from literature study, and previous and current geotechnical investigation. Identify existing and proposed underground facilities and utilities near each proposed project component that may affect the design and construction of the project component.

3.5 Groundwater

Summarize the groundwater conditions from available information. Provide a summary table of measured groundwater table elevation, following Table 2.

Table 2 – Measured Groundwater Table

Location or Borehole No.	Ground Surface Elevation (feet)	Groundwater Table or Piezometric Elevation		Date Measured	Notes
		Depth (feet)	Elevation (feet)		

Provide preliminary interpreted groundwater table elevations for design. Provide:

- Direction and gradient of groundwater, artesian conditions, and confined aquifers
- Perched water tables
- Potentially significant seasonal variations
- Observed influences on the groundwater table

- Describe features observed to infer groundwater, streams, springs, seeps, and vegetation.

3.6 Seismic Hazards

Provide preliminary seismic information.

In PGDR, include preliminary seismic hazard assessment for the entire project site and, where appropriate, to each relevant project components.

For project components with long alignment or covering a large area, divide the project component into segments based on differences in:

- site-to-source distance,
- subsurface conditions, and
- characteristic features of the project component.

Provide seismic hazard evaluation for each segment of the project component.

3.6.1 Site Seismic Parameters

- Geospatial information (latitude/longitude in decimal degrees) of representative location(s) where V_{S30} values are evaluated.
- The time-averaged shear-wave velocity (V_{S30}) for the top 30 m of the earth materials, how it was determined (e.g., CPT or SPT correlations, Seismic CPT, and geophysical methods).

Refer to *Design Acceleration Response Spectrum* module to determine V_{S30}.

3.6.2 Ground Motion Parameters

Obtain design Horizontal Peak Ground Acceleration (HPGA), mean earthquake moment magnitude (M), and mean site to fault source distance (R) based on procedures described in *Design Acceleration Response Spectrum* module. Summarize seismic design information in Table 3 below.

Table 3. Recommended Ground Motion Parameters for Geotechnical Design

Project Component ID	Site Parameters			Design Ground Motion Parameters (Return Period = 975 years)		
	Locations		Shear-Wave Velocity V_{S30}, (m/sec)	Horizontal Peak Ground Acceleration (HPGA)(1) (g)	Mean Earthquake(1) M, Moment Magnitude	Mean Site-to-Fault Source Distance(1) R, (km)
	Latitude, degrees	Longitude, degrees				
	XXX.XXXX	XXX.XXXX	XXX.X	X.XX	X.XX	XX.XX

1. Based on Caltrans web tool ARS Online (Version 3.xx)

3.6.3 Parameters for Seismic Slope Stability Analysis

- Design horizontal seismic coefficient for seismic slope stability analysis for projects components that require slope stability assessment

3.6.4 Fault Rupture

Refer to the *Fault Rupture* module.

3.6.5 Liquefaction

Refer to the *Liquefaction Evaluation* module.

3.6.6 Liquefaction-Induced Lateral Spreading

Refer to the *Liquefaction-Induced Lateral Spreading* module.

4 GEOTECHNICAL DESIGN EVALUATION

Describe the geotechnical design evaluation performed and a summary of the results. These may include:

- Feasibility and cost ranking of mitigation options for landslide, slope instability, or rockfall
- Review of geotechnical options for ERS, slopes, or sound walls
- Review of geotechnical issues for environmental or project reports

Discuss how project may affect geotechnical design considerations, such as right-of-way, traffic control, environmental constraints, allowable construction window, and constructability.

5 RECOMMENDATIONS

Provide recommendations such as:

- Scope of geotechnical monitoring, investigation, and design – for the DPGR that assist in development of the PID
- Summary of geotechnical design considerations – for the DPGR that assist the development of environmental impact reports or PSR
- Comparison of feasible options – for the PGDR that assist in type selection

6 REFERENCES

Cite references following the author-date format. Refer to *ASCE Publishing in ASCE Journals, A Guide for Authors* for details.

Examples:
Arizona Dept. of Commerce. (2005). “Community profile: Hualapai Indian Reservation.” <http://www.azcommerce/com/doclib/commune/hualapai.pdf> (Mar. 17, 2014).

Irish, J. L., and Resio, D. T. (2013). “Method for estimating future hurricane flood probabilities and associated uncertainty.” J. Waterway, Port, Coastal, Ocean Eng., 10.1061/(ASCE)WW.1943-5460.0000157, 04013015.

Karam, G. N. (1991). “Effect of fiber volume on the strength properties of short fiber reinforced cements with application to bending strength of WFRC.” Proc., 6th Technical. Conf. of the American Society for Composites, A. Smith, ed., Vol. 1, Technomics, Lancaster, PA, 548–557.

Smith, R. L., Bailey, R. A., and Ross, C. A. (1970). Geologic map of the Jemez Mountains, New Mexico, U.S. Geol. Surv. Misc. Invest. Map, I-571.

REPORT COPY LIST

Refer to *Report Distribution* in *Communication and Reporting* section of *Offices of Geotechnical Design - Quality Management Plan*.

APPENDICES

1. **Site Map and Cross-Section** – should be embedded in the report and readable. Only place large format figures in the appendices.
2. **Geologic Map and Cross-Section** – should be embedded in the report and readable, with an appropriate legend. Only place large format figures in the appendices.
3. **Subsurface Profiles** – should be embedded in the report and readable. Only place large format figures in the appendices.
4. **Photos** – photos that enhance readability should be embedded in the report. Only place supporting photos in the appendices.
5. **Calculation Package**

   For reports prepared by Caltrans, do not include calculation package in the appendix. The calculation package, both digital and scanned files, should be stored in Project Record Management System.

   For reports prepared by consultants, include the calculation package in the appendix.

   Include:

   a. The objectives of each calculation, such as time rate of settlement or bearing capacity
   b. List of all given information and assumptions used to simplify the calculation, and the source of the information
   c. Developed geotechnical models for the calculation
   d. Equations used and meaning of the terms used in the equations
   e. Copies of the curves or tables used in the calculation and their source
   f. Load and resistance factors, or factor of safety, used for each calculation
   g. For the calculation performed using computer spreadsheets – step-by-step calculation for one example to demonstrate the basis of the spreadsheet. A computer spreadsheet is not a substitute for the step-by-step calculation.
   h. Summary of the calculation results that form the basis of geotechnical recommendations, including a sketch of the design, if appropriate
6. **Exception to Policy –** Include the approved *Request for Exception* form.

GEOTECHNICAL DESIGN REPORT

Cover the following subjects in the GDR.

1 INTRODUCTION

Include a concise introduction of the project and its location on the State Highway System. State the type and purpose of the report. Refer to the request or contract received for the preparation of this report. Provide additional information that are relevant to the project.

1.1 Project Description

Provide a brief description of the project based on information supplied in the Project Report. Include Project Datum reference and a Vicinity Map showing the project location(s).

Describe the components of the project covered in the GDR. Maps, drawings, or plans are effective in showing the locations and alignments of the project components.

Use the following table as a template to provide summary information for project components, such as: the begin and end of slope, sound wall, and ERS alignment, or the location of the culvert, signpost, or infiltration basin.

Obtain from the Project Engineer permanent geospatial information of the project components, such as latitude and longitude (in degrees and accurate to 6 decimal places), or northing and easting. Enter this information into the table. For project components that traverse the highway, such as culverts, reference the component alignment, in addition to the latitude and longitude or northing and easting.

To ensure the geospatial information presented in this table is consistent with those shown on the Plans, verify the geospatial information with the Project Engineer before issuing the report.

Table 4 – Description of the Proposed Slope, ERS, etc.

ID No.	Slope or ERS	Begin		End		Length (feet)	Design Height (feet)	Notes
		Northing/Easting (Latitude/Longitude)		Northing/Easting (Latitude/Longitude)			Max.	

1.2 Exception to Policy

List exceptions to Departmental policies and procedures relating to the GDR. Include the approved Request for Exception forms in the appendix.

2 GEOTECHNICAL INVESTIGATION

Describe the geotechnical and geologic investigation performed, including literature study of available information.

2.1 Subsurface Exploration

Describe tasks performed for subsurface exploration such as trenching, boring, Standard Penetration Test (SPT), Cone Penetration Test (CPT), geophysical tests, and other in-situ tests, such as pressuremeter test. Follow the *Soil and Rock Logging, Classification and Presentation Manual* when presenting results of the subsurface exploration.

Summarize the boreholes and CPTs performed in Table 2.

Table 5. Borehole/CPT Summary List

Borehole /CPT No.	Adjacent Project Component	Northing/Easting (Latitude/Longitude)		Project Center Line Station/Offset, ft (Optional)		Ground Surface Elevation, ft	Depth, ft	Date Completed

Briefly describe the type, installation, and monitoring of in-situ instrumentation, such as slope indicator and survey monitoring points, and recorded data.

Refer to the *Geotechnical Investigations* module for Caltrans practice on subsurface exploration.

2.2 Laboratory Test

List soil and rock laboratory tests and corrosion test performed.

2.3 Geologic Mapping

Describe geologic mapping performed.

3 GEOTECHNICAL CONDITIONS

Provide geotechnical conditions of the sites. The information provided in this section is the basis for geotechnical analyses and design.

3.1 Geology

Describe project site geology and subsurface conditions, including a general description of regional geology relevant to the project. Present information only, not how it relates to design and construction. A generalized discussion is sufficient. Include:

- Soil and rock types, geologic units, and geologic properties
- Depth to bedrock

- For rock slopes – rock structure, weathering degree, fractures intensity, bedding and joints, rock hardness and strength, and potential for seepage
- Geologic hazards – landslide, slope failure, rockfall, or debris flow
- Heaving or expansive materials
- Collapsible foundation materials
- Other unsuitable materials

Include geologic maps showing:

- Geologic structures, units, and significant features, such as faults, bedding, major joint attitudes, and folds
- Locations of geologic cross-sections
- Layout line of project components

Include geologic cross-sections that illustrate the relationship of geologic conditions to the project components.

3.2 Surface Conditions

Describe site topography, surface water and drainage conditions, significant natural features, and land use history. Describe the performance of existing cut, fill, and natural slopes. Describe soil erosion and scour, and historical maintenance issues, that may affect the proposed project components.

Identify existing and proposed above-ground structures, facilities, and utilities near each proposed project component that may affect the design and construction of the project component.

3.3 Subsurface Conditions

Summarize subsurface conditions based on information obtained from geotechnical investigation, and literature. Identify existing and proposed underground facilities and utilities near each proposed project component that may affect the design and construction of the project component.

Exercise judgment on whether a subsurface profile/cross-section is needed for a project component.

For essential project components, including cuts, fills, landslides, structures, and walls, provide interpreted longitudinal subsurface profiles/cross-sections along the layout line or perpendicular to the layout line of the project components and representative cross-sections of subsurface profiles where analysis and evaluation were performed.

Present interpreted subsurface profiles/cross-sections used in the analyses and design.

Keep the interpretive information in the profiles and cross-sections to a minimum. Delineate stratification lines with dashed lines and question marks where not reasonably

certain. Note the potential for variability in the interpreted stratification in the report, if a detailed stratification is provided.

3.4 Groundwater

Describe the groundwater conditions and describe gradient and direction of groundwater if possible.

Provide a summary of measured groundwater as in Table 3. Describe the method and equipment used, and literature study, to determine ground water table level and observe flow and seepage.

Table 6 – Measured Groundwater Table

Location or Borehole No.	Ground Surface Elevation (feet)	Groundwater Table or Piezometric Elevation		Date Measured	Notes
		Depth (feet)	Elevation (feet)		

Present interpreted groundwater table elevations for design, historical high groundwater table elevation, and whether fluctuation of the groundwater should be expected. Describe features observed to infer groundwater, streams, springs, seeps, and vegetation.

3.5 Seismic Hazards

Provide seismic hazard assessment for the entire project site and, where appropriate, to each relevant project components.

For project components with long alignment or covering a large area, divide the project component into segments based on differences in:

- site-to-source distance,
- subsurface conditions, and
- characteristic features of the project component.

Provide seismic hazard evaluation for each segment of the project component.

3.5.1 Site Seismic Parameters

- Geospatial information (latitude/longitude in decimal degrees) of representative location(s) where V_{S30} values are evaluated.
- The time-averaged shear-wave velocity (V_{S30}) for the top 30 m of the earth materials, how it was determined (e.g., CPT or SPT correlations, Seismic CPT, and geophysical methods). Refer to the *Design Acceleration Response Spectrum* module for how to determine V_{S30}.

3.5.2 Ground Motion Parameters

Obtain design Horizontal Peak Ground Acceleration (HPGA), mean earthquake moment magnitude (M), and mean site to fault source distance (R) based on procedures described in the *Design Acceleration Response Spectrum* module. Summarize seismic design information in Table 4 below.

Table 7. Recommended Ground Motion Parameters for Geotechnical Design

Project Component ID	Site Parameters			Design Ground Motion Parameters (Return Period = 975 years)		
	Locations		Shear-Wave Velocity V_{S30}, m/sec	Horizontal Peak Ground Acceleration (HPGA) (1), g	Mean Earthquake (1) M, Moment Magnitude	Mean Site-to-Fault Source Distance (1) R, km
	Latitude, degrees	Longitude, degrees				
	XXX.XXXX	XXX.XXXX	XXX.X	X.XX	X.XX	XX.XX

1. Based on Caltrans web tool ARS Online (Version 3.xx)

3.5.3 Parameters for Seismic Slope Stability Analysis

Design horizontal seismic coefficient for seismic slope stability analysis for projects components require slope stability assessment

3.5.4 Fault Rupture

Refer to the *Fault Rupture* module.

3.5.5 Liquefaction

Refer to the *Liquefaction Evaluation* module.

3.5.6 Liquefaction-Induced Lateral Spreading

Refer to the *Liquefaction-Induced Lateral Spreading* module.

4 Analyses and Design

Document the analyses and design performed. For a project includes multiple project components, organize this section based on project components, instead of types of analyses, is recommended for clarity.

4.1 Project Design Information

Document and date project design information provided by other design team members, and design decisions made by the project design team that set the criteria and parameters for geotechnical design. These may include:

- Maximum allowable total and differential settlement, and lateral displacement
- Existing foundation
- Underground utilities

- Available construction easement and access
- Environmental constraints, such as construction noise and vibration restrictions
- Allowed construction season
- Allowed traffic control schedule

These criteria and parameters can affect the selection of component types and construction methods or require special analysis methods or design steps.

4.2 Soil/Rock Engineering Properties

Provide a summary table of the engineering properties that corresponds to the soil/rock layers shown in the interpreted subsurface profiles (Section 3.3), such as:

- Soil shear strength, unit weight, and compressibility
- Rock shear and compressive strengths, modulus, composition, weathering, bedding, and degree of fracturing

Table 5 shows an example of the presentation of essential interpreted soil engineering properties.

Table 8 – Soil Engineering Properties

Layer No.	Depth of the Top of Layer, feet	General Soil Description	SPT N Value	Interpreted Soil Engineering Properties		
				Unit Weight (g), pcf	Apparent Cohesion (*C*), psf	Apparent Friction Angle (ϕ), degrees

For engineering properties interpreted from laboratory tests, include the analysis of the test results and how those test results apply to the specific site conditions. For engineering properties and profiles correlated and interpreted from in-situ testing data, such as those obtained using SPT or CPT, include the correlation formula used. For engineering properties derived from back-analysis based on measured data or observed evidences, include the analyses and assumptions used.

Include information, data, and deliberation that substantiate the interpreted soil/rock engineering parameters.

4.3 Geotechnical Model and Analyses

Discuss the geotechnical model used for analysis and design. Include:

- The design element, such as a wall, footing, or slope, located in context to the original and finish grades
- Subsurface profiles derived from the information presented in Section 3.3 *Subsurface Condition*
- External loads

For slope stability or numerical analysis, the computer-generated output cross-section with soil/rock engineering properties used for the analysis, may be substituted as the geotechnical model.

Include relevant plans and cross-sections used for the geotechnical models in the appendix.

Present the geotechnical analysis and design performed. Include analysis methods used, assumptions made, calculation steps, and a summary of results. These may include:

- Immediate and time-dependent settlement, displacement, and deformation analyses
- Bearing capacity calculation
- Slope stability analyses – analysis approach, assessment of failure mechanisms, and back-analyses performed
- Analyses performed to determine soil strength properties and geotechnical nominal resistances
- For Standard-Plan ERS, verification of the geotechnical site conditions and interpreted soil engineering properties that meet or exceed those assumed and shown in the Standard Plans
- Evaluation of seismic hazards and seismic design parameters
- Evaluation of corrosivity of the sites that need corrosivity evaluation – include corrosion test results in the Appendix.

5 RECOMMENDATIONS

To ensure completeness of the recommendations, refer to corresponding modules in the Geotechnical Manual, and FHWA manuals and the Checklist. Also, consult with the project team members for their needs of geotechnical information and recommendations.

5.1 Embankment

(Refer to the Geotechnical Manual)

5.2 Slope Stabilization and Landslide Mitigation

(Refer to the Geotechnical Manual)

5.3 Erosion Control and Scour Mitigation

- Recommendations for geotechnical erosion protection measures, such as erosion control mats or blankets, flattening the slope, geosynthetics reinforced slope surface, and diverting surface water
- Recommendations for slope protection measures, such as riprap, gabion baskets, geosynthetics wrapped slope

Refer to *Caltrans Erosion Control Toolbox* provided by Caltrans Landscape Architect Design for erosion control measures that are not directly based on geotechnical methods.

5.4 Rockfall Hazard Mitigation

(Refer to the Geotechnical Manual)

5.5 Excavated Slope

(Refer to the Geotechnical Manual)

5.6 Reinforced Soil Slope

- Elevation view of the slope that shows layout of geosynthetic reinforcements, embedment lengths, elevations, and beginning and ending stations
- Cross-section view of the slope that shows surficial stability reinforcements, geotechnical erosion control details, and subdrain details
- List of geosynthetic reinforcements, with corresponding Long-Term Strength (LTS)
- Required engineering properties of backfill material

5.7 Standard Plan Earth Retaining System

(Refer to the Geotechnical Manual)

5.8 Sound Walls

(Refer to the Geotechnical Manual)

5.9 Ground Improvement

(Refer to the Geotechnical Manual)

5.10 Trenchless Method for Culvert Installation

(Refer to the Geotechnical Manual)

5.11 Best Management Practices for Stormwater and Leach Fields

Obtain from Project Engineer the locations and depths that need percolation test. Provide:

- Recommendations for the design of Best Management Practices (BMPs) for stormwater
- Summary and discussion of percolation tests in the following table format
- Summary and discussion of fines content tests at trench invert in the following table format

Table 9 – Summary of Percolation Tests

Basin No.	Borehole No.	Percolation Rate, minutes/inch (CTM 749 or 750)	Soil at Test Depth

Table 10 – Fines Content at Trench Invert

Trench No.	Clay Content, %	Combined Fines Content, %

5.12 Sign and Light Post Foundations

(Refer to the Geotechnical Manual)

5.13 Source Material Site Evaluation

- Summary of locations of the source material, gradation, and compaction tests
- Acceptability of the source materials

5.14 Instrumentation and Monitoring

- The needs for long-term instrumentation monitoring or construction performance evaluation
- List and layout of instruments, monitoring schedule, and data format – electronic or hard copy
- Methods of evaluation, threshold values for action, and responsible parties

6 NOTES FOR SPECIFICATIONS

Refer to the *Geotechnical Notes for Specifications* module.

7 REFERENCES

Cite references following the author-date format. Refer to *ASCE Publishing in ASCE Journals, A Guide for Authors* for details.

Examples:

Arizona Dept. of Commerce. (2005). "Community profile: Hualapai Indian Reservation." <http://www.azcommerce/com/doclib/commune/hualapai.pdf> (Mar. 17, 2014).

Irish, J. L., and Resio, D. T. (2013). "Method for estimating future hurricane flood probabilities and associated uncertainty." J. Waterway, Port, Coastal, Ocean Eng., 10.1061/(ASCE)WW.1943-5460.0000157, 04013015.

Karam, G. N. (1991). "Effect of fiber volume on the strength properties of short fiber reinforced cements with application to bending strength of WFRC." Proc., 6th Technical. Conf. of the American Society for Composites, A. Smith, ed., Vol. 1, Technomics, Lancaster, PA, 548–557.

Smith, R. L., Bailey, R. A., and Ross, C. A. (1970). Geologic map of the Jemez Mountains, New Mexico, U.S. Geol. Surv. Misc. Invest. Map, I-571.

REPORT COPY LIST

Refer to *Report Distribution* in *Communication and Reporting* section of *Offices of Geotechnical Design - Quality Management Plan*.

APPENDICES

1. **Site Map and Cross-Section** – should be embedded in the report and readable. Only place large format figures in the appendices.
2. **Geologic Map and Cross-Section** – should be embedded in the report and readable, with an appropriate legend. Only place large format figures in the appendices.
3. **Subsurface Profiles** – should be embedded in the report and readable. Only place large format figures in the appendices.
4. **Borehole Location Map** (with approximate borehole locations) – should be embedded in the report. Only place large format figures in the appendices. Overlaying borehole locations and project component alignments on Google Earth image or a map with relevant topographic features is recommended.
5. **Boring Records** – Generate Boring Records using current Data Interchange for Geotechnical and Geoenvironmental Specialists (DIGGS) schemas and data dictionaries (ex. gINT) is recommended.
6. **Digital Photos of Rock Cores** – In addition to presenting the digital photos of rock cores in the appendices, the digital photo files should also be archived in Caltrans archive (GeoDOG).
7. **Laboratory Test Data** (including Corrosion Test Report) – In addition to the laboratory test results, summarize and provide the summary tables and graphs of the interpretation of laboratory test results. Note unreliable data. Note: Digital files should be archived in Caltrans archive.
8. **Geophysical Test Data** – digital files should be archived in Caltrans archive.
9. **In-Situ Test Data** –digital files should be archived in Caltrans archive.
10. **Photos** – photos that enhance readability should be embedded in the report. Only place supporting photos in the appendices.
11. **Calculation Package**

    For reports prepared by Caltrans, do not include calculation package in the appendix. The calculation package, both digital and scanned files, should be stored in Project Record Management System.

    For geotechnical design performed by consultants, include calculation package in the appendix.

    Include:

    a. The objectives of each calculation, such as time rate of settlement or bearing capacity
    b. List and descriptions of all given information and assumptions used to simplify the calculation, and the source of the information

c. The developed geotechnical model (Section 4.2 Geotechnical Model and Analyses) for each calculation
d. Equations used and meaning of the terms used in the equations
e. Copies of the curves or tables used in the calculation and their source or reference.
f. Load and resistance factors, or factor of safety, used for the design
g. If the calculation is performed using computer spreadsheets – step-by-step calculation for one example to demonstrate the basis of the spreadsheet. A computer spreadsheet is not a substitute for the step-by-step calculation.
h. Summary of the calculation results that form the basis of geotechnical recommendations, including a sketch of the design, if appropriate.

12. **Exception to Policy –** Include the approved *Request for Exception* form.

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1. Introduction

Geotechnical engineering has been commonly recognized as the engineering work that includes geotechnical investigation, analysis, design, and preparing and issuing geotechnical reports. However, issuing the geotechnical reports should not be the end of involvement for the Geoprofessional. To ensure the design is properly contracted and constructed, the Geoprofessional should also assist in the development of the construction contract Plans, Specifications, and Estimate (PS&E) and assist the Resident Engineer with administration of the construction contract.

To assist the development of project specifications, i.e. Special Provisions, the Geoprofessional should be familiar with geotechnical-related sections of the Standard Specifications and Standard Special Provisions (SSPs).

Standard Specifications, including Revised Standard Specifications, provide uniform contract clauses that apply to all Caltrans construction contracts while the SSPs provide the means for the Specification Engineer to draft project specific contract clauses and compile them into a project's Special Provisions.

To ensure consistency and maintain contract quality, a SSP can only be edited according to the instructions embedded in each SSP. Refer to the geotechnical related SSPs in Appendix 2 for embedded instructions for each SSP. The Geotechnical Design Reports and Foundation Reports included in the final PS&E package must provide recommended edits to SSPs that are consistent with the instructions and contents of the Standard Specifications and SSPs.

Edits beyond the instructions embedded in a SSP make that SSP a Nonstandard Special Provision (NSSP). NSSPs require a time-consuming approval process and should be avoided.

For the development of a PS&E package, the Geoprofessional should:

- Provide instructions for recommended edits to SSPs in the Notes for Specifications section of the geotechnical report.

- Draft or assist in the preparation of NSSPs and assist the Specification Engineer with obtaining approval from the Specification Owner of the section of the Standard Specifications, and concurrence from the Division of Construction for geotechnical NSSPs. For the Specification Owner Roster, refer to Caltrans HQ/Design/Office Engineer web site.

- Review the draft PS&E package received from the District or Structure Office Engineer (OE) to verify that:

    - The project plans have incorporated the geotechnical recommendations provided in the Geotechnical Design Report or Foundation Report, and

- The project Special Provisions have incorporated the instructions and recommendations for SSP edits or NSSP development provided in the Geotechnical Design Report or Foundation Report.

In a contract package, the hierarchy of contract parts in descending order is:

1. Special provisions
2. Project plans
3. Revised standard specifications
4. Standard specifications
5. Revised standard plans
6. Standard plans
7. Supplemental project information (with Geotechnical Design Reports and Foundation Reports included)

Therefore, it is critical to provide concise and precise instructions and recommendations for edits of SSPs, which will be compiled into the Special Provisions.

Subsequent sections in this document provide instructions on how to provide instructions and recommendations to the Specification Engineer for SSP editing and assist with NSSP preparation:

Section 2 Notes for Specifications – general instructions

Section 3 Geotechnical Related SSPs and Suggested Instructions – instructions for recommended specification edits and examples of content for *Notes for Specifications* section

Section 4 Nonstandard Special Provisions (NSSPs) – Key issues related to the development of the NSSPs

Appendices

Appendix 1 – List of Typical SSPs for Common Geotechnical Related Construction Items

Appendix 2 – Printout of Geotechnical Related SSPs – with the instructions for specifications edits shown

2. Notes for Specifications

To provide clarity in geotechnical reporting, two new sections, the *Notes for Specifications* and *Notes for Construction*, have been created to replace the previous *Construction Considerations* section in geotechnical reports.

In the *Notes for Specifications* section, provide concise instructions and recommendations to the Specification Engineer to edit SSPs and develop Nonstandard Special Provisions (NSSPs). Do not quote the Standard Specifications in the geotechnical report. Standard Specifications are applicable to all construction contracts. Referring to or quoting from Standard Specifications creates duplication and potential to misstate the Standard Specifications.

To advise reviewing the entire geotechnical report before using the information provided, provide the following note at the beginning of the *Notes for Specifications* section:

"*Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.*"

To create a clear and concise communication channel between the Geotechnical Designers, Specification Engineers, and Project Engineers, identify SSP section numbers and provide the instructions for the recommended edits. For projects with multiple construction components, list applicable SSPs and provide instructions and recommendations for edits under each construction component. An SSP may be edited for different parts of the project. For example, a large project may include multiple earth retaining systems, highway embankments, and bridges. Conditions, such as groundwater and unsuitable materials, can vary at each construction component location. Each construction component would require different edits for the same SSP.

3. Geotechnical Related SSPs and Suggested Instructions

This section does not provide a comprehensive compilation of geotechnical related SSPs. In addition, SSPs are updated on an as-needed basis and new SSPs are periodically developed and inserted into the SSP list. For up-to-date SSPs, refer to the latest SSP list provided by *Caltrans HQ/Design/Office Engineer*, which can be accessed through the following Caltrans internal web link: *https://design.onramp.dot.ca.gov/2018-construction-contract-standards*.

For questions on geotechnical related SSPs and specifications related issues, contact Geotechnical.Specification@dot.ca.gov .

For each geotechnical related SSP shown below, instructions are provided for the instructions and recommendations that should be included in the *Notes for Specifications* section. The instructions followed by "➢" are action items, while the instructions followed by "❖" are for information and clarification. These instructions were developed based on the edit instructions embedded in each of the SSPs. When reviewing the instructions provided in this section, cross reference the geotechnical related SSPs with the embedded instructions for each SSP compiled in Appendix 2.

This section is organized in the sequential order of contract items listed in the Standard Specifications and the SSPs.

2 Bidding

2-1.06B Supplemental Project Information

- Provide a list of materials, rock cores, and other information may be available for inspection at the Transportation Laboratory or other District offices. For materials and information available at District office, provide address and contact phone number.

  Do not self-reference the title of the geotechnical report. Specification Engineers will compile the list of documents and reports to be included in the Supplemental Project Information that received from the project engineer.

- Use the following table, which is also used by the Specification Engineers, to list the Supplemental Project Information:

Supplemental Project Information

Means	Description
Included in the *Information Handout*	*This field will be filled in by the Specification Enger*
Available as specified in the *Standard Specifications*	*This field will be filled in by the Specification Engineer*
Included with the project plans	
Available for inspection at the Transportation Laboratory	*Rock cores, etc.*
Available for inspection at the District Office Telephone no.: _____	
Available for inspection at: _____ _____ Telephone no.: _____	*Rock cores, etc.*
Available for inspection at: http://www.dot.ca.gov/_____	

- ❖ During geotechnical review of the Draft PS&E package, verify that the list and contents of geotechnical reports and additional geotechnical information, such as rock cores information, are included in the Supplemental Project Information / Information Handout of the Draft PS&E package.

19 Earthwork

19-3 Structure Excavation and Backfill

19-3.01D(2) Soil Nail Wall and Ground Anchor Wall Zones

- ❖ Showing wall zones on the plans is preferred by contractors and Structure Representatives. Work with structure designers and plans detailers to delineate the wall zones on the plans.
- If it was determined not to show wall zones on the plans, provide wall zones for each soil nail wall and ground anchor wall using the following table:

Wall zone	Beginning station	End station	Upper elevation (feet)	Lower elevation (feet)
1				
…				

19-3.03A Removal of Unsuitable Material below the Bridge and Earth Retaining System Footings

- Provide instructions in the *Recommendations* section or during design to have the plans show where the removal of unsuitable material below the bottom of bridge or earth retaining system footings is required.
- Include a note for inclusion of this SSP.
- If material below the bottom of excavated unstable material must be compacted, provide the locations that need to be compacted in the following table:

Structure Name and Number	Begin Station (or Abutment Number)	End Station (or Bent Number)

19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation

- Include a note for inclusion of this SSP if surface water or groundwater is expected during structure excavation but no seal course is recommended and shown (Structure excavation Types D or DH).
- If surface water or groundwater is expected during structure excavation, and you determine that seal course is required, provide a recommendation for seal course (Structure excavation Type A) in the Recommendations section of the Foundation Report.

19-3.03B(2) Soldier Pile Retaining Walls are not Built Completely from the Top Down

- No actions or recommendations are needed from Geotechnical Design but refer to the Standard Specifications and SSPs for details.

19-3.03E(3) Minimum Compacted Backfill behind the Lagging of a Soldier Pile wall is other than 5 feet above the first Row of Ground Anchors

- Provide the required minimum height of compacted backfill above the level of ground anchors before drilling.

19-3.04 Structure Excavation and Backfill Payment Clauses

- ❖ No actions or recommendations are needed from Geotechnical Design but refer to the Standard Specifications and SSP for details.

19-6 Embankment Construction

19-6.02B Geosynthetic Reinforced Embankment

- ➢ Provide the identifications, such as Type 1, Type 2, of geosynthetic reinforcement materials and their Long-Term Strengths (LTS) used in the design. Use different line types and associated legend on the elevation and cross-section views on the plans to show the embedment lengths, elevations, and terminating stations of geosynthetic reinforcement layers.
- ➢ If backfill material must have different requirements than those specified in the Standard Specifications, provide recommended particle size distribution, plasticity index, and pH. Note that the LTS of geosynthetic reinforcement material can be significantly affected by the properties of the selected backfill materials.

19-6.03B Subsidence of the Ground Surface is Anticipated, and Embankment, or Imported Borrow is Measured by Theoretical Basis

- ➢ Indicate if subsidence of ground surface is anticipated due to embankment construction and refer to the *Recommendations* section for the estimated subsidence profile.
- ❖ Provide estimated subsidence profile in the *Recommendation*s section so that the additional quantity and cost of embankment and import borrow, if used, can be estimated.

19-6.03D Settlement Periods and Surcharges

- ➢ If settlement periods are required at bridges, provide instructions and the settlement period in the following table. Provide surcharge height in the following table if applicable. If only settlement periods are required, and no surcharge is required, enter "0.0" in the "Surcharge height" column.

Structure Number	Support Location	Surcharge height (feet)	Settlement period (days)

- ➢ If settlement periods are required for roadway embankments at the earth retaining structures, provide instructions and the settlement period in the following table. Provide surcharge height in the following table if applicable. If only settlement periods are required, and no surcharge is required, enter "0.0" in the "Surcharge height" column.

Earth Retaining Structure Number	Begin Station	End Station	Surcharge height (feet)	Settlement period (days)

19-6.04 Payment Quantity for Anticipated Subsidence – for Embankment Construction

- ❖ No actions or recommendations are needed from Geotechnical Design but refer to the Standard Specifications and SSP for details.
- ❖ If subsidence is anticipated during embankment construction, the magnitude of anticipated subsidence should be provided in the Geotechnical Design Report, so that the payment quantity can be estimated by others. Refer to Geotechnical Design Report Guidelines.

19-6 Borrow Material

19-7.02B Local Borrow

- ❖ If requested, in the *Recommendations* section of the Geotechnical Design Report provide geotechnical assessment of local borrow identified by District.

19-7.02C Imported Borrow

- ❖ If requested, in the *Recommendations* section of the Geotechnical Design Report provide geotechnical assessment of imported borrow identified by District.

46 Ground Anchors and Soil Nails

46-1 General

46-1.01A Ground Anchors and Soil Nails Work Sequence

- ➢ Provide concise instructions on specific work sequencing concerns related to ground anchors and soil nails construction. These may include the timing of ground anchor stressing relative to the construction of other elements due to bearing capacity consideration.

46-1.03E Research Investigation

- ➢ Provide concise instructions for research activities, such as survey markers, slope indicator casings and other equipment.
- ❖ Refer to the SSP for the assistance needed from the contractor.

46-2 Ground Anchors

46-2.01C Alternative Number of Ground Anchors

- ❖ This SSP is listed for information only. Structure designers are responsible for providing information for this SSP when an alternative number of ground

anchors that provide the same horizontal and vertical components and distribution of the design force is allowed.

46-2.01D(2)(b)(i)Locations and Number of Ground Anchor Performance Tests

- ❖ During design, provide locations of ground anchors subject to performance testing. During Geotechnical Review of Draft Structure PS&E, review the plans to verify the locations are shown on the plans.
- ➢ When the locations cannot be determined during design and shown on the plans, provide the minimum number of ground anchors subject to performance testing. Refer to instructions in the SSP for the minimum number of performance tests that should be performed. During construction, assist the Structure Representative with selecting the locations.

46-2.03A Expected Difficult Ground Anchor Installation due to Geotechnical Issues

- ➢ Indicate if:
  - Difficult ground anchor installations are expected due to geotechnical issues, such as loose sands, caving, presence of groundwater, soft clay, cobbles and boulders. Refer to LOTBs or observed surface conditions if available and applicable.
  - Voids may be encountered that may affect drilling and grouting, such as along a landslide plane or voids encountered during the geotechnical investigation or observed on the surface. Lost circulation during geotechnical investigation and voids on the ground surface also suggest the presence of subsurface voids. Refer to the SSP for details.
  - Provide structure numbers, stations and elevations and use the following table if necessary:

Structure Number	Begin station	End station	Elevation	Condition

- ➢ Indicate and provide locations where ground anchors are to be installed under a spread footing or existing structure.

- ❖ For example, if cave in of the drilled hole could negatively affect an existing structure. The top level of ground anchors will be installed with drilled holes advanced with drill casing.

46-3 Soil Nails

46-3.01D(2)(b)(ii)(3) 2% of Total Number of Production Soil Nails

- ❖ Verify the value equal to 2 percent of the total number of production soil nails for each wall or the entire project is provided in the SSP. This is the number of additional proof tests required at locations to be determined by the Engineer. (Note: The Engineer is the Resident Engineer as defined in Standard Specifications 1-1.07B Glossary.)
- ❖ The locations of these proof test nails are to be determined during construction or to be placed at zones that require special attention.

46-3.03A Expected Difficult Soil Nail Installation due to Geotechnical Issues

- ➢ Indicate if:
  - Difficult soil nail installations are expected due to geotechnical issues, such as loose sands, caving, presence of groundwater, soft clay, cobbles and boulders. Refer to LOTBs or observed surface conditions if available and applicable.
  - Provide structure numbers, stations and elevations. Use the following table if necessary:

Structure Number	Begin station	End station	Elevation	Condition

49 Piling

49-1 General

49-1.01D(3) Load Test Piles

- ➢ Provide load test control zones in the following table for driven piles with a diameter greater than 36 inches or if you recommended load tests.

Structure Number	Control zone	Load test pile support location

49-1.01D(4) Dynamic Monitoring of Driven Piles

- ➢ If dynamic monitoring of pile redriving should be performed other than 1 day after the pile is driven, provide the number of days. Refer to the Standard Specifications and SSP for details.

- Use the following table to provide the locations of supports and control zones that require dynamic monitoring to develop bearing acceptance criteria curves:

Structure Number	Control zone	Dynamic monitoring support location

Notes:

1) Dynamic monitoring is required:
   - for driven piles with a diameter of 18 inches and greater, or
   - if the required nominal driving resistance exceeds 600 kips.
2) You may consider additional dynamic monitoring for piles that do not satisfy the conditions identified in Note 1 above:
   - At sites with subsurface condition that the modified Gates formula needs to be verified.
   - At locations closer to or at the support locations, while the load test and dynamic monitoring will be performed at a location within the same control zone but away from the support locations. According to SSP 49-1.01D(3), in control zones where load test is required (for piles with a diameter D >36"), dynamic monitoring will be performed on both load test pile and anchor piles.
   - To verify the integrity of the piles when subject to the dynamic loads under proposed driving system.

49-1.01D(5) Test Borings to Verify Top of Rock Elevation if the Top of Rock Elevations Vary or are Uncertain – for Piling to be Embedded into Rock

- ❖ Use this SSP as the last resort. The preferred option is for Caltrans Geotechnical Design staff to perform this work during construction.
- Indicate if test borings are required during construction to verify top of rock. The intent of this specification is to verify top of rock, not to gather additional geotechnical data.
- Provide locations where test borings should be performed using the following table:

Structure Number	Location

- Provide the minimum depths below the specified pile tip elevation if the required minimum depths are other than 20 feet below the specified tip elevation shown. Refer to corresponding Standard Specifications and SSP for details.

49-1.03 Expected Difficult Pile Installation

- Indicate if difficult pile installation is expected. Provide the locations of the piles, and a list of site conditions based on factual observation using the following table. Refer to the LOTBs, Foundation Report, observed ground conditions, artesian groundwater condition, or exposed excavated surface, for the conditions listed in the table.

Pile Location		Conditions
Structure Number	Support Location	

49-2 Driven Piling

49-2.01A(3)(a) Pile and Driving Data Form Submittal

- ❖ No actions or recommendations are needed from Geotechnical Design but refer to the SSP for expected review of the Pile and Driving Data Form submittals.

49-2.01A(3)(b) Require Driving System Submittal

- Indicate if the requirement for driving system submittals for each of the selected support locations or control zones is included in the *Recommendations* section of the geotechnical report.
- ❖ Each control zone requires a driving system submittal.
- Provide pile type, monitored support locations, and control zones using the following table:

Structure Number	Pile Type	Monitored Support Location	Control Zone

49-2.01C(2) Conditions Require Revising Specifications for Driving Equipment

- Indicate and provide instruction if any of the following conditions exist and revising the specifications is required:
  - Installing sheet piles that have no geotechnical capacity along the length of the pile. Where vibratory hammers for sheet piles may be used; provide the locations using the following table:

Structure Number	Abutment Number	Bent Number

49-2.01C(3) If Drilling for Driven Piles or Casings is Allowed or Restricted

- No action for paragraphs 1, 2, and 4.
- Use the following table to provide the locations of drilling allowed, and the bottom of the drilled hole elevation to attain the specified tip elevation.

Structure Number	Support Location	Bottom of Hole Elevation, feet

Note: According to the Standard Specifications, a "drilled hole" has a diameter less than the least dimension of the pile.

- Indicate if center-relief drilling may be necessary for open-ended CISS and steel pipe piles; provide the exclusion zone above the specified pile tip where center-relief drilling is not allowed.

Note: For open-ended steel pipe piles and CISS, center-relief drilling is not allowed before driving piles.

49-2.01C(4) Predrilling for Driven Piles in Existing Embankment is Required

- Indicate and provide the following table if:
  - piles are to be installed in existing fills with thickness greater than 5 feet, or
  - predrilling is required due proximity to an obstruction, such as underground utilities, or other structure.

Structure Number	Support Location	Bottom of Hole Elevation, feet	Condition

Note: According to Standard Specifications, a predrilled hole must be at least 6 inches larger than the greatest dimension of the pile cross section and used in embankments constructed under the contract, or near an obstruction.

- Indicate if predrill holes are needed for piles to be driven through existing concrete footings that are to remain structurally functional.

49-2.01C(5) Pile Set Period is Allowed

- Indicate if pile set period is allowed. Use the following table and provide:
  - The locations where pile set period is allowed, and
  - The set period (hours) based on the soil type at the site.

Structure Number	Support Location	Set Period (hours)

- Notify the Specification Engineer that this SSP needs to be revised as follows:
  - Add the following sentence after the first sentence in the first paragraph: "*Leave the top of the pile one foot above cut off for restrike.*"
  - Replace "*12 hours*" in the last sentence of the first paragraph with the set period (hours) provided.
- Refer to the SSP for details.

49-3 Cast-In-Place Concrete Piling

49-3.02A(1) CIDH Piles Designed as End Bearing

- Indicate if there are CIDH concrete piles designed as end bearing. Provide the locations of the piles using the following table.

Structure Number	Support Location

49-3.02B(6)(c) List of Synthetic Slurry for CIDH piles at least 24" in diameter

- No actions or recommendations needed from Geotechnical Design but refer to the SSP for details of the synthetic slurry.
- Standard Specifications, section 49-3.02B(6)(c): "*Do not use synthetic slurries in holes drilled in primarily soft or very soft cohesive soils as determined by the Engineer.*"

49-3.02B(6)(d) Allow the Use of Water Slurry for CIDH Piles with Full-Depth Casing

- By default, water slurry is not allowed in the Standard Specifications.
- Indicate, if in the rare scenario that you allow the use of water as slurry, when a casing is used for the entire length of the drilled hole.

49-3.02C(1) Drilling Sequence for CIDH Piling Center-to-Center Spacing less than 4 Pile Diameters

- Indicate if a drilling sequence is required when the CIDH piling center-to-center spacing is less than 4 pile diameters.

49-3.02C(4) CIDH Piles Designed as End Bearing

- Indicate inclusion of this SSP if there are CIDH concrete piles designed as end bearing.

49-3.02C(6) Not-Allowed Installation Methods for Permanent Steel Casing

- Provide a list of methods not allowed for installing permanent steel casings if any. Refer to the SSP for details.

49-3.02C(7) Permanent Casing for Type II CIDH Piling

- Indicate if the permanent casing must be corrugated metal pipe.
- Indicate if slurry cement backfill is allowed instead of grout. In the rare case that groundwater is below the tip of the casing, and the pile design does not rely on side resistance, then slurry cement backfill instead of grout may be allowed.
- If there are permanent casing installation methods not allowed, provide the list.
- ❖ Refer to the SSP for details.

49-3.03C(2) Clean-Out Depth for Open-Ended CISS Piles

- Indicate if other than 8 feet of the material at the bottom of piles must remain and provide the length.
- ❖ Refer to the SSP for typical practice.

49-4 Steel Soldier Piling

49-4.03B Conventional Drilling Equipment for Soils may not be Suitable and Bedrock or Boulders is shown in LOTBs; Pile Substitutions are Allowed

- Indicate if factual evidence of bedrock or boulders with soil matrix foundation material is shown on LOTBs and conventional drilling equipment for soils may not be suitable for drilling holes.
- ❖ No action for the second paragraph.

49-5 Micropiling

The SSP for micropiles divides into two categories:

Category A: Micropiles resisting primarily axial forces due to axial loading at the top of the micropile. This typically includes bridge foundations and may occasionally include some walls. For Category A, the contractor determines the length, load testing is performed, and typically pressure grouting is used.

Category L: Micropiles subject to primarily shear and bending forces along their length due to lateral loading. This typically includes earth stabilization structures and most walls. For Category L, the State determines the length, load testing is not performed, and gravity grouting is used.

- Provide the category of micropiles for the project here, so that the SSP can be edited accordingly.

49-5.01D(4)(c)(i) Load Tests – Compression, Tension, or both; Proof, Verification, or both; Factored Test Load and Service Load

- Indicate if micropiles need to be load tested in compression.
- Indicate if micropiles need to be load tested in tension.
- Indicate if micropiles need to be load tested in both tension and compression.
- Indicate if micropiles need to be proof load tested.
- Indicate if micropiles need to be verification load tested.
- Indicate if micropiles need to be both proof and verification load tested.
- Indicate if creep test is required during the load tests.
- Review the PS&E package and verify that Factored Test Load (FTL) and Service Load (SL) for respective load tests (compression, tension, or both) are provided by Structure Design.

49-5.01D(4)(c)(ii) Verification Load Tests, Representative Supports or Wall Zones; Creep Test; Adjustment of Load Increment

- For bridges, provide location of each verification test micropile and the control zone that the verification test micropile represents using the following format:

Structure Number	Verification Test Micropile Location	Control Zone

Notes:

If load tests are required, a tension load test is preferred because it is cheaper to perform and typically provides a conservative assessment of pile compressive nominal resistance.

- For walls and slope stabilization structures, provide location of each verification test micropile and limits of the wall zone that the verification test micropile represents using the following table:

Structure Number	Verification Test Micropile Location	Wall Zone Limits	
		Begin Station	End Station

Notes:

1. Load tests are seldom required for slope stabilization and ERS applications.
2. If load tests are required, a tension load test is preferred since it is cheaper to perform and typically provides a conservative assessment of pile capacity.

- Provide an instruction as to whether creep test is required during verification load test.
- Provide recommended adjustment of load increments between 1.00SL and 1.00 FTL, if needed.
- ❖ Refer to the SSP for details.

49-5.01D(4)(c)(iii) Number of Proof Load Tests; Timing of the First Proof Test; Creep Test; Adjustment of Load Increment

- Provide the number of micropiles per bridge support or wall zone to be proof load tested.
- Provide the percentage of micropiles that must be installed per bridge support or wall zone before the first proof test is performed.
- Provide an instruction on whether creep test is required during proof load test.
- Provide recommended adjustment of load increments between 1.00SL and 0.80 FTL, if needed.
- ❖ Refer to the SSP for details.

49-5.01D(5)(b) Verification Load Test Acceptance Criteria

- ❖ Note: Defer to Structure Design for the maximum axial movement allowed.

49-5.01D(5)(c) Proof Load Test Acceptance Criteria

- ❖ Note: Defer to Structure Design for the maximum axial movement allowed.

49-5.03A Minimum Spacing and Elapsed Time when Installing the Next Micropile

- If the required minimum center-to-center spacing is greater than 5 feet, provide the minimum center-to-center spacing from an open hole.
- If the required grout set time is greater than 12 hours, provide the minimum elapsed time before start drilling the adjacent micropile.

  Note: The default minimum center-to-center spacing is 5 feet, and at least 12 hours after initial grout has set.

49-5.03B Verification Test – Provide Geotechnical Equivalence using Isolation

- Provide the locations where the verification test micropile must be lengthened and isolated.
- Provide the amount to be lengthened and the elevation of the bottom of isolation.

  Notes:

  1. Isolation may be required due to the ground surface elevation being significantly higher than the cutoff elevation of the production micropiles, due to sloping strata layers, or due to other situations that cause a lack of geotechnical equivalence between the verification test micropile and the production micropiles that it represents.

2. The isolation should ensure that the verification test accounts for the same engaged length and ground materials as the production micropiles. Verification test movement criteria may need to be adjusted to be comparable to the proof test movement criteria, due to a longer elastic length.

58 Sound Walls

58-2.03A Friction Angle of the Foundation Material which Determines the Dimensions of the Sound Wall Foundation

- ➢ Verify that the Geotechnical Design Report provides the friction angle of the foundation material, which will determine the dimensions of the sound wall foundation, either piles or trench footings.
- ➢ Provide the friction angle of the foundation materials using the following table.

Sound Wall Number	Station Limits	Friction Angle (ϕ) of Foundation Materials, degrees

- ❖ Refer to the SSP and Standard Plans for details.

61-2 Culvert and Drainage Pipe Joints

61-2.01D(3) Drainage Pipe Settlement and the Characteristics of Surrounding Soils

- ❖ Pay attention to projects with culverts and drainage pipes, especially under new embankment.
- ➢ Indicate if the expected settlement of the pipe is greater than 3 inches, which will trigger field leakage tests of the pipe. Provide estimated lapse time after pipe installation and backfill before the leakage test is performed.
- ➢ Indicate if the materials around the pipeline are fine cohesionless (sand or silt) soils.
- ❖ Refer to the SSP for details.

65 Concrete Pipe

65-2 Rock or Unyielding Material under the Concrete Pipe

- ❖ Pay attention to projects with reinforced concrete pipes under new embankment.
- ➢ Indicate if rock or other unyielding material may be encountered in the excavation immediately below the bottom of the pipe bedding. The rock or the unyielding material immediately below the bottom of the pipe bedding should be excavated and backfill with structure backfill material.

68 Subsurface Drains

68-2.02F(1) Permeable Material for Underdrains

- If Class 1 permeable material is not to be used for underdrains, provide the recommended permeable material, Class 2 or 3, for underdrains. Refer to Standard Specifications for details.

68-5 Permeable Material Blanket

- Provide the recommended permeable material for permeable material blanket.
- ❖ Refer to the SSP for details.

69 Overside Drains

69-2 Geomembrane and Cushion Fabric

- For each site or application, provide the selection of
  - Geomembrane Class
  - Smooth or Textured Geomembrane, and
  - Cushion Fabric Class

71 Existing Drainage Facilities

71-3.03 Grout Port Locations – Contact Grouting to Rehabilitate Existing Drainage Structures

- If requested, provide recommendations for the grout port locations.

72 Slope Protection

72-2.02C Nonstandard Fabric for Rock Slope Protection

- Provide recommended fabric specifications if the fabric selected is not described in section 96-1.02I Rock Slope Protection Fabric.

96 Geosynthetics

96-1.02B Specify Filter Fabric Class

- Provide the locations/application to receive filter fabric, and the class of filter fabric.
- ❖ The selection of filter fabric should be based on the gradation of the soils at the job site. Refer to the SSP for details.

Examples

Example 1 – Geosynthetic Reinforced Embankment

Notes for Specifications

Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.

19-6.02B Geosynthetic Reinforced Embankment

- Backfill gradation requirements

Sieve size	Percentage passing
1-1/2"	100
3/4"	75 – 100
No. 4	40 – 100
No. 40	0 – 80
No. 200	0 – 60

- Backfill quality characteristics requirements

Quality characteristic	Test method	Requirement
Plasticity index (max)	California Test 204	25
pH	California Test 643	5 – 9

- Geosynthetic reinforcement Long Term Strength requirements:

Geosynthetic reinforcement type	Long Term Strength (lb/ft)
GR 1	3,600
GR 2	2,800
GR 3	1,600

19-6.03B Subsidence of the Ground Surface is Anticipated, and Embankment, or Imported Borrow is Measured by Theoretical Basis

- Subsidence of ground surface is anticipated due to embankment construction. Please refer to the *Recommendations* section of the report for the estimated subsidence profile.

Example 2 – MSE Wall

Notes for Specifications

Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.

19-3.03A Removal of Unsuitable Material below the Footings

- Include this SSP.
- At the locations shown in the following table, material below the bottom of excavated unstable material must be compacted:

Structure Number	Begin Station	End Station
53-1537	07+00	10+80
	14+70	18+90
	21+30	25+50

19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation

- Groundwater is expected during excavation. Include this SSP.

19-6.03D Settlement Periods and Surcharges

Structure Number	Begin Station	End Station	Surcharge height (feet)	Settlement period (days)
53-1537	05+00	11+70	5	60
	11+70	18+10	10	60
	18+10	26+50	7	30

Example 3 – Ground Anchors

Notes for Specifications

Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.

46-2.03A Expected Difficult Ground Anchor Installation due to Geotechnical Issues

Structure Number	Begin Station	End Station	Elevation	Conditions
24-0824	10+00	10+80	All ground anchors	Perched water Cobbles and boulders; as observed on the slope face Loose to medium dense fine sand; refer to LOTBs.
	11+00	11+50	80’ – 96’	Loose fine sand with gravel; refer to LOTBs
24-0825	50+00	51+00	70’ – 90’	Boulders; as observed on the slope face

Example 4 – Driven Piles

Notes for Specifications

Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.

49-1.01D(4) Dynamic Monitoring of Driven Piles

Locations of supports and control zones that require dynamic monitoring are:

Structure Number	Control Zone	Dynamic Monitoring Support Location
24-1390	Abut 1	Abut 1
	Bents 2 – 5	Bent 3

49-1.03 Expected difficult pile installation

Pile Location		Conditions
Structure Number	Support Location	
24-1390	All Supports	Groundwater; refer to LOTBs

49-2.01A(3)(b) Require Driving System Submittal

Driving System Submittals are required as described in the *Recommendations* section.

Pile type, support location, and control zone are:

Structure Number	Pile Type	Support Location	Control Zone
24-1390	CISS (24 x 0.5")	Abut 1	Abut 1
		Abut 7	Abut 7

49-2.01C(5) Pile Set Period is Allowed

The support locations that allow pile set period and the minimum set period are shown in the following table:

Structure Number	Support Location	Set Period (hours)
24-1390	Bent 2, 3, 4	36

This SSP needs to be revised as follows:

- Add the following sentence after the first sentence in the first paragraph: "*Leave the top of the pile one foot above cut-off for restrike.*"
- Replace "*12 hours*" in the last sentence of the first paragraph with the minimum set period (hours) provided, or "as shown in the following table".

Example 5 – CIDH Piles

Notes for Specifications

Before using the information provided in this section, please review the report to comprehend the contents and intent of the geotechnical design.

19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation

Type D structure excavation is recommended at the following support locations:

Structure Number	Support Location
24-0390	Pier 2, 3, 4

49-1.03 Expected difficult pile installation

Pile Location		Conditions
Structure Number	Support Location	
24-0391	All Supports	Presence of groundwater; refer to LOTBs and the Foundation Report.
	Pier 3	Boulders or concrete slabs are expected during CIDH pile installation; refer to the Foundation Report.

4. Nonstandard Special Provisions (NSSPs)

If there is a potential need for NSSPs, notify the Specification Engineer as soon as possible. The development, review, and approval of an NSSP can take several months. The scenarios that require NSSPs include:

- The need for construction items that are not in the *Coded Contract Items (Bid Items)* list, such as rock dowels or lightweight cellular concrete, and
- The need for construction methods that are not included in the Standard Specifications or Standard Special Provisions, such as jet grouting or stone columns.

The Geotechnical Designer must provide technical content for an NSSP and work with the Specification Engineer to develop an NSSP that is biddable and buildable. The Geotechnical Designer is professionally responsible for the technical content of the NSSP.

For questions on geotechnical related NSSPs and associated issues, contact Geotechnical.Specification@dot.ca.gov .

For a specified product, there must be at least two manufacturers that can provide the product meeting the requirements. Notify the District, if there is potentially only one manufacturer that can provide the product that meets the requirements. Assist the Project Engineer in obtaining the approval of using sole source products/materials.

For sole source products/materials, the Project Engineer must complete a Public Interest Finding (PIF), to satisfy the State proprietary requirements and obtain an approval memo issued by the District Director/Deputy Director or DES Chief for proprietary products being used on the Department's contract. As of October 28th, 2019, FHWA no longer requires a PIF and has rescinded their proprietary requirements.

The following are short contract clauses that may be needed for projects but are currently under the NSSP status and to be incorporated into SSPs or Standard Specifications in the near future.

- 49-3.02C(3) Types of Temporary Casing not Allowed

Section 49-3.02C(3). Use for temporary steel casings. List methods of installation, such as: vibration, oscillation, rotation, drilling and placing the casing, not allowed per the Foundation Report.

Add to section 49-3.02C(3):

Do not use __ to install temporary steel casings.

Appendix 1 – List of Typical SSPs for Common Geotechnical Related Construction Items

The following is a list of typical SSPs for common geotechnical related construction items. These lists are not comprehensive. For any given project, there could be other SSPs that address various geotechnical related construction issues.

Embankment

19-6.03B	Subsidence of the Ground Surface is Anticipated, and Embankment, or Imported Borrow is Measured by Theoretical Basis
19-6.03D	Settlement Periods and Surcharges
19-7.02B	Local Borrow
19-7.02C	Imported Borrow

Geosynthetic Reinforced Embankment

19-6.02B	Geosynthetic Reinforced Embankment
19-6.03B	Subsidence of the Ground Surface is Anticipated, and Embankment, or Imported Borrow is Measured by Theoretical Basis
19-6.03D	Settlement Periods and Surcharges

Spread Footing

19-3.03A	Removal of Unsuitable Material below the Footings
19-3.03B(1)	Surface Water or Groundwater is Expected during Structure Excavation
90-1.02G(6)	Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H	Concrete in Direct Contact with a Corrosive Environment

Standard Plan Earth Retaining Systems

19-3.03A	Removal of Unsuitable Material below the Footings
19-3.03B(1)	Surface Water or Groundwater is Expected during Structure Excavation
47-2.01A, 47-3.01, 47-6.01A, and 47-6.01C(2)	Alternative Earth Retaining System for Caltrans Standard Walls, MSE, or Crib Wall Designs
51-1.01A	Alternative Earth Retaining Systems for Type 1 – 5 Retaining Walls
90-1.02G(6)	Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H	Concrete in Direct Contact with a Corrosive Environment

Non-gravity cantilever Earth Retaining Systems (Solider Pile wall with laggings)

19-3.03B(1)	Surface Water or Groundwater is Expected during Structure Excavation
49-4.03B	Drilled Holes through Rock Material and Pile Substitutions
90-1.02G(6)	Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H	Concrete in Direct Contact with a Corrosive Environment
49-4.01C	Drilling through Cobbles, Boulders, or Hard Rock is Anticipated
49-4.03A	Drilling through Cobbles, Boulders, or Hard Rock is Anticipated
49-4.03B	Drilled Holes through Rock Material and Pile Substitutions

Soil Nail Walls

19-3.01D(2) Soil Nail Wall Zones
19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation
46-1.01A Soil Nails Work Sequence
46-1.03E Research Investigation
46-3.01D(2)(b)(ii)(3) 2% of Total Number of Production Soil Nails
46-3.02A Corrosivity of the Site
46-3.03A Expected Difficult Soil Nail Installation due to Geotechnical Issues

Ground Anchor Walls

19-3.01D (2) Ground Anchor Wall Zones
19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation
19-3.03E(3) When Minimum Compacted Backfill behind the Lagging of a Soldier Pile Wall is other than 5 Feet above the Ground Anchors
46-1.01A Ground Anchors Work Sequence
46-1.03E Research Investigation
46-2.01C Alternative Number of Ground Anchors
46-2.01D(2)(b)(i)Locations and Number of Ground Anchor Performance Tests
46-2.01D(2)(b) and 46-2.01D(2)(c) Ground Anchors as Wall Footing Anchors
46-2.02B Ground Anchor Bearing Plates on Materials Other than Concrete
46-2.03A Expected Difficult Ground Anchor Installation due to Geotechnical Issues
46-3.02A Corrosivity of the Site

Mechanically Stabilized Embankment Wall

19-3.03A Removal of Unsuitable Material below the Footings
19-3.03B(1) Surface Water or Groundwater is Expected during Structure Excavation
47-2.01A, 47-3.01, 47-6.01A, and 47-6.01C(2) Alternative Earth Retaining System for Caltrans Standard Walls, MSE, or Crib Wall Designs
47-3.02B(3) Corrosivity and Marine Environment of the Site

Sound Walls

58-2.03A Friction Angle of the Foundation Material that Determines the Dimensions of the Sound Wall Foundation

Driven Piles

49-1.01D(3) Load Test Piles
49-1.01D(4) Dynamic Monitoring of Driven Piles
49-1.01D(5) Test Borings to Verify Top of Rock Elevation if the Top of Rock Elevations Vary or are Uncertain – for Steel Piling to be Embedded into Rock
49-1.03 Expected Difficult Pile Installation
49-2.01A(3)(b) Require Driving System Submittal
49-2.01C(2) Conditions require Revised Specifications for Driving Equipment
49-2.01C(3) Drilling for Driven Piles or Casings is not Allowed or Restricted
49-2.01C(4) Predrilling for Driven Piles is Required
49-2.04B(1) Alternative "X" Piles for Lateral Load

90-1.02G(6) Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H Concrete in Direct Contact with a Corrosive Environment

CIDH Piles

49-1.01D(3) Load Test Piles
49-1.01D(5) Test Borings to Verify Top of Rock Elevation if the Top of Rock Elevations Vary or are Uncertain – for Steel Piling to be Embedded into Rock
49-1.03 Expected Difficult Pile Installation
49-3.02A(1) and 49-3.02C(4) CIDH Piles Designed as End Bearing
49-3.02B(6)(d) Allow the Use of Water Slurry for CIDH Piles with Full-Depth Casing
49-3.02B(7) Allow Slurry Cement Backfill in the Annular Space
49-3.02C(1) Drilling Sequence for CIDH Piling Center-to-Center Spacing less than 3 Pile Diameters
49-3.02C(6) Not-Allowed Installation Methods for Permanent Steel Casing
49-3.02C(7) Permanent Casing for Type II CIDH Piling with Optional Construction Joint Shown
90-1.02G(6) Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H Concrete in Direct Contact with a Corrosive Environment

CISS Piles

49-1.01D(3) Load Test Piles
49-1.01D(4) Dynamic Monitoring of Driven Piles
49-1.01D(5) Test Borings to Verify Top of Rock Elevation if the Top of Rock Elevations Vary or are Uncertain – for Steel Piling to be Embedded into Rock
49-1.03 Expected Difficult Pile Installation
49-2.01A(3)(b) Require Driving System Submittal
49-2.01C(2) Conditions require Revised Specifications for Driving Equipment
49-2.01C(3) Drilling for Driven Piles or Casings is not Allowed or Restricted
49-2.01C(4) Predrilling for Driven Piles is Required
49-3.03C(2) Clean-Out Depth for Open-Ended CISS Piles
90-1.02G(6) Concrete Placed in Soil or Water with a Corrosive Environment
90-1.02H Concrete in Direct Contact with a Corrosive Environment

Appendix 2 – Compilation of Geotechnical Related SSPs

Section 2-1.06B. Use if supplemental project information is available.

List the available supplemental project information.

If railroad relations and insurance requirements are available, insert *Railroad Relations and Insurance Requirements* in the 1st row.

If water use is restricted and the project requires 100,000 gal or more of water for nonlandscaping work, insert *Water source information* in the 1st row.

If water allowance calculations for landscaping are available, insert *Maximum Applied Water Allowance Calculations for New and Rehabilitated Landscapes* in the 1st row.

If electronic design files (e.g., cross sections, roadway design alignments, smoothness data) are available as described in Project Delivery Directive PD-06, insert the names of the files in the 2nd row.

If bridge as-built drawings are available, insert *Bridge as-built drawings* in the 2nd row.

If logs of test borings are available, insert *Logs of test borings* in the 3rd row.

Insert the telephone number in the 5th row.

For a District 10 project, insert (209) 948-7934.

For districts other than 5, 6, and 10, insert the telephone number to call to schedule a viewing date.

Insert the location and telephone number in the 6th row.

For a District 5 project, insert	**1150 LAUREL LN STE 175** **SAN LUIS OBISPO CA** **(805) 549-3116**
For a District 6 project, insert	**2015 E SHIELDS AVE STE 100** **FRESNO CA** **(559) 230-3115**

If a project has more than 5,000 cu yd of earthwork construction, insert the location and the documents specified in section 5-1.25 in the 7th row. Do not include any electronic design files already listed in the 2nd row.

Add rows as necessary. Delete nonapplicable rows.

Add between the 1st and 2nd paragraphs of section 2-1.06B:

The Department makes the following supplemental project information available:

Supplemental Project Information

Means	Description
Included in the *Information Handout*	
Available as specified in the *Standard Specifications*	
Included with the project plans	
Available for inspection at the Transportation Laboratory	
Available for inspection at the District Office Telephone no.: _____	
Available for inspection at: _____ _____ Telephone no.: _____	
Available for inspection at: http://www.dot.ca.gov/_____	

Section 19-3.01D(2). Use for soil nail walls and ground anchor walls.

1–2. Insert the stations and elevations provided by Geotechnical Services. The entire wall surface must be described by the chart. Add or delete cells as needed.
Insert the location of the wall. Add introductions and tables as needed.

Add to section 19-3.01D(2):

1. Use for all soil nail walls if not shown on the plans.

The wall zones for the soil nail wall at _____ are as shown in the following table:

Wall zone	Beginning station	End station	Upper elevation (ft)	Lower elevation (ft)
1				
2				
3				
4				
5				
6				

2. Use for ground anchor walls only if more than 1 wall zone is listed in the foundation report. For ground anchors under spread footings, check with Design if the wall zone is more than 50 feet in length.

The wall zones for the ground anchor wall at _____ are as shown in the following table:

Wall zone	Beginning station	End station	Upper elevation (ft)	Lower elevation (ft)
1				
2				
3				
4				
5				
6				

Section 19-3.03A. Use for bridge and retaining wall earthwork where removal of unstable material below the bottom of the footing is required.

Replace *Reserved* in section 19-3.03A with:

1. If Class 2 AB is not used in the roadwork, edit to specify a stable, cohesive material similar to AB. If possible, specify a material already on the Bid Item List.

Where shown, remove material below the bottom of bridge footings. Replace the material with Class 2 AB and compact it as specified for structure backfill in section 19-3.03E.

2. Edit to be consistent with District plans. If Class 2 AB is not used in the roadwork, edit to specify a stable, cohesive material similar to AB. If possible, specify a material already on the Bid Item List.

Where shown, remove material below the bottom of retaining wall footings. Replace the material with Class 2 AB and compact it as specified for structure backfill in section 19-3.03E. The relative compaction must be at least 95 percent.

3. Use if the remaining base material must be compacted. Complete the table.

A relative compaction of at least 95 percent must be attained to at least 0.5 foot below the bottom of excavated unstable material at the locations shown in the following table:

Bridge name and number	Abutment number	Bent number

Section 19-3.03B(1). Use if structure excavation (Type D) or (Type DH) is shown and no seal course is shown. Insert the type.

Add to the beginning of section 19-3.03B(1):

For footings at locations with structure excavation (Type __), ground or surface water is expected to be encountered but seal course concrete is not needed.

Section 19-3.03B(2). Use for soldier pile retaining walls if the walls are not built completely from the top down. Ensure limits of earthwork are shown. District provides cross-sections; OSD to confirm with District.

Delete the 1st paragraph of section 19-3.03B(2).

Section 19-3.03E(3). Use for soldier pile retaining walls with ground anchors if the height for the compacted backfill is other than 5 feet. Verify with the designer. Insert the backfill height. The height must be adequate to allow anchor testing.

Replace *5* in the 1st sentence of the 3rd paragraph of section 19-3.03E(3) with:

_____

Section 19-3.04. Use for structure excavation and backfill payment clauses.

Add to section 19-3.04:

1. Use for bridge and retaining walls where unstable material below the bottom of the footing is removed and replaced. Edit if the backfill is not Class 2 aggregate.

Class 2 aggregate base placed below footings is paid for as structure backfill.

2. Use when structure excavation (Type D) or (Type DH) is described for sometimes wet, or wet excavation, and no seal course is shown on the project. Insert type (e.g., D, DH). Do not use with par. 3.

Structure excavation for footings at locations not shown as structure excavation (Type __) and where ground or surface water is encountered is paid for as structure excavation (bridge).

3. Use when structure excavation (Type D) or (Type DH) is described for sometimes wet, or wet excavation, and no seal course is shown for that location but a seal course is shown at other locations. Insert type (e.g., D, DH). Do not use with par. 2.

Except at locations where seal course concrete is shown, structure excavation for footings at locations not shown as structure excavation (Type __) and where ground or surface water is encountered is paid for as structure excavation (bridge).

4. Use with SSP 19-3.03B(3) for pier column excavation.

Structure excavation (pier column) is measured from the bottom of the completed foundation excavation to the upper and horizontal limits shown.

5. Use if quantity of pervious backfill material is small. Small is considered to be about 50 cubic yards or less.

Pervious backfill material placed within the limits of payment for bridges is paid for as structure backfill (bridge). Pervious backfill material placed within the limits of payment for retaining walls is paid for as structure backfill (retaining wall).

6. Use if culvert drainage profiles are based on aerial surveys.

Replace item 3 in the list in the 6th paragraph of section 19-3.04 with:

3. Structure excavation more than 1 foot from the depth shown is paid for as a work-character change if you request an adjustment or the Engineer orders an adjustment.

Section 19-6.02B. Use for geosynthetic reinforced embankments.

1. Use if the backfill material must have different requirements from those specified in the Standard Specifications. Insert the percentage passing and plasticity index in the tables.

Replace the 3rd and 4th paragraphs in section 19-6.02B with:

The backfill must comply with the gradation requirements shown in the following table:

Sieve size	Percentage passing
1-1/2"	
3/4"	
No. 4	
No. 40	
No. 200	

The backfill must comply with the quality characteristics shown in the following table:

Quality characteristics	Test method	Requirement
Plasticity index (max)	California Test 204	
pH	California Test 643	

2. Insert the types and LTDS as recommended by the geotechnical designer.

Add to section 19-6.02B:

The LTDS of geosynthetic reinforcement must comply with the requirements shown in the following table:

Geosynthetic reinforcement type	LTDS (lb/ft)

Section 19-6.03B. Use if subsidence of the ground surface is anticipated, and there is a bid item for either embankment, or imported borrow measured by theoretical basis.

Add to the end of section 19-6.03B:

1. Use if the project includes a bid item for embankment. Enter quantity.

A quantity of _____ cubic yards of embankment will be added to the computed embankment quantity for the anticipated effect of subsidence.

2. Use if imported borrow is measured by theoretical basis. Enter quantity.

A quantity of _____ cubic yards of embankment will be added to the computed imported borrow quantity for the anticipated effect of subsidence.

Section 19-6.03D. Use for settlement periods and surcharges.

Add to section 19-6.03D:

1. Use for settlement periods at bridges. For embankments requiring settlement periods but no surcharge; insert 0.0[a] in the surcharge height column. If no such embankments exist, delete the footnote *a* at the bottom of the table.

Settlement periods and surcharges are required for bridge approach embankments as shown in the following table:

Bridge name or number	Abutment number	Bent number	Surcharge height (feet)	Settlement period (days)
x	x	x	x	x
x	x	x	x	x
x	x	x	x	x

[a]At this location, construct embankment by extending the grading plane (GP) in the elevation view of the bridge embankment surcharge detail of standard plan A62B horizontally to the centerline of the abutment.

2. Use for settlement periods at earth retaining structures.

Settlement periods and surcharges are required for roadway embankments at the earth retaining structures as shown in the following table:

Earth retaining structure number	Surcharge height (feet)	Settlement period (days)
x	x	x
x	x	x
x	x	x

Section 19-6.04. Use for embankment construction.

Add to the end of section 19-6.04:

1. Use if the project includes a bid item for embankment and there is anticipated subsidence. Edit to include the quantity.

The payment quantity for imported borrow includes the volume of anticipated subsidence as specified in section 19-6.03B. The payment quantity for embankment includes ____ cu yd for the anticipated effect of subsidence.

Section 19-7.02B. Use for specifying the locations of local borrow sites.

Replace *Reserved* in section 19-7.02B with:

1. Insert locations.

In addition to the locations described for excavation, obtain local borrow from:

1. _____
2. _____

2. Use only if restoration of the local borrow site is required.

After you obtain local borrow, grade the borrow site such that it drains and blends in with the surrounding area.

Section 19-7.02C. Use for specifying imported borrow.

Add to section 19-7.02C:

1. Insert an R-value. Do not specify an R-value greater than that of the native material.

Imported borrow placed within 4 feet of the finished grade must have an R-value of at least _____.

2. Use if gradation requirements are specified.

Process the imported borrow to comply with the grading requirements.

3

Strip materials that adversely affect the imported borrow properties.

4–6. Use if the project has a mandatory source for imported borrow.

4. Insert the location of the mandatory local material source.

Obtain imported borrow from the mandatory local material source at_____.

5. Insert the cost and unit.

The Department has arranged for you to obtain material from the mandatory source for $_____ per _____ for material removed from the site and used in the work. The cost of the material removed is deducted.

6. Use if restoration of the borrow site and haul roads is required. Edit to suit the work.

After obtaining imported borrow, grade the borrow sites and associated haul roads such that sites drain and blend in with the surrounding area. Remove any equipment on the areas before grading.

Section 46-1.01A. Use for ground anchors and soil nails if the designer has a specific work sequencing concern related to ground anchor construction; for example, the timing of ground anchor stressing relative to the construction of other elements. Insert project-specific work sequencing specifications.

Add to section 46-1.01A:

_____.

Section 46-1.03E. Use for ground anchors and soil nails if research investigation is required by the designer. Notify the estimator of the need for supplemental funds. Consult the geotechnical report and edit as necessary to describe the scope of all research activities so the Contractor can estimate the impact on operations.

Replace *Reserved* in section 46-1.03E with:

1. If used for ground anchors, replace "soil nail" with "ground anchor." Edit as necessary if the structure is not a wall.

The Department will conduct research activities within the limits of the soil nail wall.

2. Verify that the locations of the slope indicator casings are shown. Edit as necessary to describe the actual work.

Research activities include installing and monitoring survey markers, slope indicator casings, and other equipment at the locations shown. Survey markers are installed on the face and crest of the wall.

3

Research equipment installation is scheduled in advance. Coordinate construction activities to prevent interference with the equipment installation and monitoring.

4. Edit if the Contractor installs the equipment instead of the Department.

The Department furnishes and installs the research equipment. If the Engineer orders you to assist in installing the equipment, this work is change order work.

Section 46-2.01C. Use for ground anchors if on a short time limit contract or if an alternative number of anchors is allowed.

1. Use for a short time limit contract if strand tendons are shown on the plans.

Add to the 1st paragraph of section 46-2.01C:

You may submit a certificate of compliance for the corrosion-inhibiting grease instead of submitting a test sample and test data.

2. Use if an alternative number of ground anchors is allowed. Confirm with the designer.

Add to section 46-2.01C:

You may submit calculations and details for furnishing an alternative number of ground anchors that provide the same horizontal and vertical components and distribution of the design force as provided by the anchors shown. Include alternative structure details. Alternative design calculations and details must be sealed and signed by an engineer who is registered as a civil engineer in the State.

Section 46-2.01D(2)(b)(i). Use for ground anchors if performance test locations are not shown or if the compressive strength at loading is not 2,880 psi.

1. Use if performance test locations are not shown. Insert the number provided by the designer. For wall anchors, generally a minimum of 3 anchors but not less than 5 percent of anchors should be performance tested. For footing anchors, generally a minimum of 2 anchors but not less than 10 percent of anchors per footing should be performance tested. For footing anchors, add "at each footing" to the end of the 1st sentence.

Add to section 46-2.01D(2)(b)(i):

Performance test a minimum of ___ ground anchors. The Engineer determines which anchors are to be performance tested.

2. Use if the compressive strength at loading is not 2,880 psi or if requested by the designer to delete "or has cured for at least 7 days." Insert the value provided by the designer. Typically, 80 percent of the 28-day compressive strength is specified as the compressive strength at loading.

Replace the 3rd paragraph of section 46-2.01D(2)(b)(i) with:

Do not stress against the concrete until it has attained a compressive strength of at least _____ psi or has cured for at least 7 days.

Section 46-2.03A. Use for ground anchors with project-specific geotechnical conditions or with a grout cover over the corrugated sheathing of other than 1 inch.

1–3. Use for project-specific geotechnical considerations.

Add to section 46-2.03A:

1. Use for expected difficult ground anchor installation.

Insert the location of the difficult installation. Insert the conditions that affect ground anchor installation as described in the foundation report. Insert other conditions that may apply, such as: low overhead clearance, underground utilities, overhead utilities, noise control, vibration monitoring, staged construction, and traffic control.

Verify and obtain concurrence from Geotechnical Services.

Use multiple pars. if foundation reports show differing conditions:

1. Along the ground anchor wall. Insert stations and elevations as provided in the foundation report.

2. For different structures.

Expect difficult ground anchor installation at _____ due to the presence of the following conditions:

1. _____
2. _____
3. _____

2. Use for horizontal ground anchors installed under a spread footing for a structure or if caving of the drilled hole could be detrimental to an existing structure. Insert the location of the ground anchors. Add stations if necessary. Create a table format for multiple locations.

Install the top level of ground anchors at _____ in drilled holes advanced with drill casing. Withdraw the drill casing as the grout is being placed in the drilled hole, keeping the end of the casing immersed in the grout.

3. Use if the ground anchors will pass through slide planes or foundation material with significant voids. Insert the location of the voids. Add stations and elevations if necessary. Create a table format for multiple locations. Confirm with the designer and Geotechnical Services.

At _____ you may encounter voids in the foundation material along the length of the drilled hole that affect drilling and grouting. Use measures such as a grout sock to avoid the excessive loss of grout into the voids encountered.

4. Use if the designer requires a grout cover over the corrugated sheathing of other than 1 inch. Insert the thickness of the cover.

Replace the 9th paragraph of section 46-2.03A with:

The diameter of the drilled hole must be large enough to provide a minimum grout cover of ___ inches over the corrugated sheathing for the full length of the tendon.

Section 46-3.01D(2)(b)(ii)(3). Use for soil nails. Insert 2 percent of the total number of production soil nails.

Add to the 2nd paragraph of section 46-3.01D(2)(b)(ii)(3):

In addition to the proof test soil nails shown, install and test ____ proof test soil nails at locations determined by the Engineer.

Section 46-3.03A. Use for expected difficult soil nail installation.

Insert the location of the difficult installation. Insert the conditions that affect soil nail installation as described in the foundation report. Insert other conditions that may apply, such as: low overhead clearance, underground utilities, overhead utilities, noise control, vibration monitoring, staged construction, and traffic control.

Verify and obtain concurrence from Geotechnical Services.

Use multiple pars. if foundation reports show differing conditions:

1. Along the soil nail wall. Insert stations and elevations as provided in the foundation report.

2. For different structures.

Add to section 46-3.03A:

Expect difficult soil nail installation at _____ due to the presence of the following conditions:

1. _____
2. _____
3. _____

Section 49-1.01D(3). Use for load test piles.

A load test pile is required for driven piles with a diameter greater than 36 inches or if recommended in the foundation report.

Add to the end of section 49-1.01D(3):

1. Modify or delete table as recommended by Geotechnical Services. Table for control zones and load test pile support locations must not be shown on the plans.

The Department performs load tests on the load test pile at each control zone as shown in the following table:

Bridge no.	Control zone	Load test pile support location

2. Use for driven load test piles.

Driven load test piles and anchor piles are monitored for dynamic response during the final 25 feet of driving under section 49-1.01D(4).

Section 49-1.01D(4). Use for dynamic monitoring of driven piles.

Dynamic monitoring is required (1) for driven piles with a diameter 18 inches and greater, (2) if the required nominal driving resistance exceeds 600 kips or (3) if recommended in the foundation report.

Unless otherwise recommended in the foundation report:

1. For piles with a diameter (D) of 18" ≤ D ≤ 36", dynamic monitoring is performed on the actual driven production pile.

2. For piles with a diameter D >36", dynamic monitoring is performed on the driven load test pile and anchor piles.

1. Use if recommended in the foundation report to specify other than 1 day for when the pile will be redriven. Insert number of days.

Replace item 6 in the 8th paragraph of section 49-1.01D(4) with:

6. After ___ days, install the instrument package on the pile and attach the cables and resume driving the pile to the specified tip elevation.

Add to the end of section 49-1.01D(4):

2. Modify or delete table as recommended by Geotechnical Services. Do not use for load test and anchor piles.

The Department performs dynamic monitoring of the first production pile driven for each control zone at the support location shown in the following table:

Bridge no.	Control zone	Dynamic monitoring support location

Section 49-1.01D(5). Use if steel piling is to be embedded into rock and the top of rock elevations vary.

Test borings are used to verify top of rock elevation. It is not the intent of this specification to gather any other geotechnical data. Use only if requested by the Office of Geotechnical Services.

1. Use to specify test boring locations. Delete the table and edit accordingly for only 1 location.

Add to section 49-1.01D(5):

Perform test borings at the locations shown in the following table:

Bridge no.	Location

2. Use to revise the depth of drilling below the specified tip elevation. Insert depth required.

Replace the 5th paragraph of section 49-1.01D(5) with:

Drill test borings by rotary drill methods to a depth of at least __ feet below the specified tip elevation shown. Test borings must be at least 3 inches in diameter.

Section 49-1.03. Use for expected difficult pile installation. Use this SSP for all types of pile installation.

Insert the conditions that affect pile installation as described in the foundation report. Insert other conditions that may apply, such as: low overhead clearance, underground utilities, overhead utilities, noise control, vibration monitoring, staged construction, and traffic control.

Verify and obtain concurrence from Geotechnical Services.

Insert bridge no. and support locations for difficult pile installation. List specific conditions for each location.

Add to section 49-1.03:

Expect difficult pile installation due to the conditions shown in the following table:

Pile location		Conditions
Bridge no.	Support location	

Section 49-2.01A(3). Use for driven piles. Retain Pile and Driving Data Form.

1. Use for all driven piles except if a driving system submittal or dynamic monitoring is requested by Geotechnical Services. Insert bridge number, pile type, and support location (e.g. Abutment 1, Bent 2) or control zone.

Add to section 49-2.01A(3)(a):

Before installing driven piles, submit a Pile and Driving Data Form for each pile type for each of the support locations or control zones shown in the following table:

Bridge no.	Pile type	Support location or control zone

2. Use if a driving system submittal or dynamic monitoring is requested by Geotechnical Services. Insert bridge number, pile type, and support location (e.g. Abutment 1, Bent 2) or control zone.

Add to section 49-2.01A(3)(b):

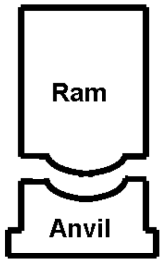
Before installing driven piles, submit a driving system submittal for each pile type for each of the support locations or control zones shown in the following table:

Bridge no.	Pile type	Support location or control zone

CALIFORNIA DEPARTMENT OF TRANSPORTATION
TRANSPORTATION LABORATORY

PILE AND DRIVING DATA FORM

Structure Name :_____ Contract No.:_____
_____ Project:_____
Structure No.:_____ Pile Driving Contractor or
Dist./Co./Rte./Post Mi:_____ Subcontractor_____(Pile Driven By)



Hammer

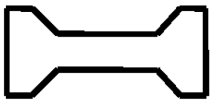
Manufacturer:_____ Model:_____
Type:_____Serial No.:_____
Min Rated Energy: _____ at ___ Length of Stroke ____ Fuel Setting
Max Rated Energy: _____ at ___ Length of Stroke ____ Fuel Setting
Ram Weight:_____kips

Modifications: _____
_____
_____



Capblock (Hammer Cushion)

Material:_____
Thickness:_____in Area:_____in^2
Modulus of Elasticity - E:_____ksi
Coefficient of Restitution - e:_____



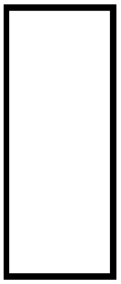
Pile Cap

[Helmet / Bonnet / Anvil Block / Drivehead] Weight:_____ kips



Pile Cushion

Material:_____
Thickness:_____in Area: _____in^2
Modulus of Elasticity - E:_____ksi
Coefficient of Restitution - e:_____



Pile

Pile Type:_____
Length (In Leads):_____ft
Lb/ft.:_____Taper:_____
Wall Thickness:_____in
Cross Sectional Area:_____in^2
Design Pile Capacity:_____kips
Description of Splice:_____
_____
Tip Treatment Description:_____
_____

DISTRIBUTE:

- ☐ Translab, Foundation Testing
- ☐ Translab, Geotechnical Design
- ☐ Resident Engineer

Note: If mandrel or follower is used to drive the pile, attach separate manufacturer's detail sheet(s) including weight and dimensions.

Submitted By:_____
Date:_____Phone No.:_____

Section 49-2.01C(2). Use if revising the specifications for driving equipment.

Add to section 49-2.01C(2):

1. Use only for locations with low overhead clearance, generally less than 20 feet.

You may use internal drop hammers that strike the tip of closed ended piles or impact hammers that do not comply with the minimum energy requirements to advance piles to within 3 feet of the specified tip elevation at the locations shown in the following table:

Bridge name or no.	Abutment no.	Bent no.

2. Use only for sheet piles or for CISS piles that have no geotechnical capacity along the length of the pile or casing. If only 1 location, delete the table and delete *at the locations shown in the following table:* Delete *rotators, or oscillators* if this paragraph is used only for sheet piles.

You may use vibratory hammers, rotators, or oscillators to install sheet piles or steel shells for CISS concrete piles at the locations shown in the following table:

Bridge name or no.	Abutment no.	Bent no.

3. Use for boulders, clay lenses, and other natural obstructions to pile driving, including abandoned culverts, slabs, footings, etc. Depth of excavation at uplift piles should be limited. Use only with steel piles. Edit as necessary.

If you encounter obstructions to driving, provide special driving tips or heavier pile sections, subexcavate below the bottom of footing, or take other measures to prevent damage to the pile during driving.

Section 49-2.01C(3). Use if drilling to assist pile driving is not allowed or is restricted by the foundation report.

Replace the paragraph in section 49-2.01C(3) with:

1. Use if drilling is not allowed for any structures on the project.

Do not use drilling to attain the specified tip elevation shown for driven piles.

2. Use if drilling is not allowed at specific locations and drilling is restricted at other locations. Insert locations.

For _____, do not use drilling to attain the specified tip elevation shown for driven piles.

3. Use if drilling is restricted. Insert locations.

Before driving piles, you may drill holes with a diameter not greater than the least dimension of the pile to attain the specified tip elevation shown for driven piles at the locations and to the bottom of hole elevations shown in the following table:

Bridge no.	Abutment no.	Bent no.	Bottom of hole elevation

Add to section 49-2.01C(3):

4. Use if drilling is not allowed at specific locations and may be allowed at other locations. Insert locations.

For _____, do not use drilling to attain the specified tip elevation shown for driven piles.

5. Use if center-relief drilling may be necessary for open-ended CISS and steel pipe piles. Insert depth not to be drilled. Check the plans and foundation report. If SSP 49-3.03C(2), par. 1 is used, match the depth with the clean out depth of the CISS piles.

Drilling through the center of open-ended steel shells or steel pipe piles to attain the specified tip elevation may be necessary. The diameter of the drilled hole must be less than the inside diameter of the pile. Equipment or methods used for drilling holes must not cause quick soil conditions or cause scouring or caving of the hole. Drilling must not be used within __ feet of the specified tip elevation. Do not drill before driving piles.

Section 49-2.01C(4). Use for driven piles if predrilling is required.

Add to section 49-2.01C(4):

1. Use for all piles in existing embankments and fills in excess of 5 feet or if predrilling is required due to a close proximity to an obstruction.

At existing embankments and fills, drive piles in predrilled holes at the locations and to the bottom of hole elevations shown in the following table:

Bridge name or no.	Abutment no.	Bent no.	Bottom of hole elevation

2. Use if piles must be driven through concrete footings that are to remain structurally functional.

Predrill holes through existing concrete footings for driving of steel piles at the locations shown. Do not damage the existing concrete to remain in place. Drilling methods and equipment must be authorized before starting the drilling.

Section 49-2.01C(5). Use if a pile set period is allowed as an option by Geotechnical Services instead of longer piling.

Add to section 49-2.01C(5):

1. Insert location.

If piles at _____ do not attain the nominal driving resistance at the specified tip elevation shown, you may allow them to stand for a set period without driving. The set period must be at least 12 hours.

2

After the set period has elapsed, redrive 2 piles or 10 percent of the piles in the footing, whichever is greater. The Engineer designates which piles are to be redriven. Redriving consists of operating the driving hammer at full rated energy on the pile and calculating the nominal driving resistance of the pile.

3

If the nominal driving resistance is attained for each pile designated to be redriven, the remaining piles in that footing are considered satisfactory and further driving is not required. If redriving the designated piles demonstrates that the nominal driving resistance has not been attained, redrive all piles in the footing until the nominal driving resistance is attained.

Section 49-3.02A(1). Use if CIDH concrete piles are designed as end bearing.

Add to section 49-3.02A(1) :

The CIDH concrete piles shown in the following table are specified as end bearing:

Bridge name or no.	Abutment no.	Bent no.

Section 49-3.02B(6)(c). Use for CIDH piling at least 24 inches in diameter and 5 feet in length.

Add to section 49-3.02B(6)(c):

The synthetic slurry must be one of the materials shown in the following table:

Material	Manufacturer
SlurryPro CDP	KB INTERNATIONAL LLC 735 BOARD ST STE 209 CHATTANOOGA TN 37402 (423) 266-6964
Super Mud	PDS CO INC 105 W SHARP ST EL DORADO AR 71731 (870) 863-5707
Shore Pac GCV	CETCO CONSTRUCTION DRILLING PRODUCTS 2870 FORBS AVE HOFFMAN ESTATES IL 60192 (800) 527-9948
Terragel or Novagel Polymer	GEO-TECH SERVICES LLC 220 N. ZAPATA HWY STE 11A-449A LAREDO TX 78043 (210) 259-6386
BIG FOOT	MATRIX CONSTRUCTION PRODUCTS 50 S MAIN ST STE 200 NAPERVILLE IL 60540 (877) 591-3137
POLY-BORE	BAROID INDUSTRIAL DRILLING PRODUCTS 3000 N SAM HOUSTON PKWY EAST HOUSTON TX 77032 (877) 379-7412

Use synthetic slurries in compliance with the manufacturer's instructions. Synthetic slurries shown in the above table may not be appropriate for a given job site.

Synthetic slurries must comply with the Department's requirements for synthetic slurries to be included in the above table. The requirements are available from the Offices of Structure Design, P.O. Box 168041, MS# 9-4/11G, Sacramento, CA 95816-8041.

SlurryPro CDP synthetic slurry must comply with the requirements shown in the following table:

SlurryPro CDP

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 ≤ 67.0[a] ≤ 64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 50–120 ≤ 70
pH	Glass electrode pH meter or pH paper	6.0–11.5
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

Super Mud synthetic slurry must comply with the requirements shown in the following table:

Super Mud

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 ≤ 64.0[a] ≤ 64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 32–60 ≤ 60
pH	Glass electrode pH meter or pH paper	8.0–10.0
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

Shore Pac GCV synthetic slurry must comply with the requirements shown in the following table:

Shore Pac GCV

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 ≤ 64.0[a] ≤ 64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 33–74 ≤ 57
pH	Glass electrode pH meter or pH paper	8.0–11.0
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	 ≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

Terragel or Novagel Polymer synthetic slurry must comply with the requirements shown in the following table:

Terragel or Novagel Polymer

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 ≤ 67.0[a] ≤ 64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 45–104 ≤ 104
pH	Glass electrode pH meter or pH paper	6.0–11.5
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	 ≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

BIG-FOOT synthetic slurry must comply with the requirements shown in the following table:

BIG-FOOT

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 ≤ 64.0[a] ≤ 64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 30–125 55-114
pH	Glass electrode pH meter or pH paper	8.5–10.5
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	 ≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

POLY-BORE synthetic slurry must comply with the requirements shown in the following table:

POLY-BORE

Quality characteristic	Test method	Requirement
Density During drilling (pcf) Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1, section 4	 62.8-65.8[a] 62.8-64.0[a]
Viscosity During drilling (sec/qt) Before final cleaning and immediately before placing concrete (sec/qt)	Marsh funnel and cup. API RP 13B-1, section 6.2	 50–80 50-80
pH	Glass electrode pH meter or pH paper	7.0–10.0
Sand content, percent by volume Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	 ≤ 1.0

NOTE: Slurry temperature must be at least 40 °F when tested.
[a]If authorized, you may use slurry in a salt water environment. The allowable density of slurry in a salt water environment may be increased by 2 pcf.

Section 49-3.02B(6)(d). Use for CIDH piling at least 24 inches in diameter and full-depth casing if Geotechnical Services allows the use of water slurry.

Replace *Reserved* in section 49-3.02B(6)(d) with:

You may use water as slurry if a casing is used for the entire length of the drilled hole.

Water slurry must comply with the requirements shown in the following table:

Water Slurry Requirements

Quality characteristic	Test method	Requirement
Density Before final cleaning and immediately before placing concrete (pcf)	Mud weight (density), API RP 13B-1 section 4	63.5[a]
Sand content Before final cleaning and immediately before placing concrete (%)	Sand, API RP 13B-1, section 9	≤ 0.5

[a]If authorized, you may use salt water slurry. The allowable density of the slurry may be increased by 2 pcf.

Section 49-3.02C(1). Use for CIDH piling if (1) piling center-to-center spacing is less than 4 pile diameters, or (2) drilling locations include cobbles, boulders, or hard rock.

1. Use if CIDH piling center-to-center spacing is less than 4 pile diameters. Consult Geotechnical Services to see if a drilling sequence is necessary. If necessary, add the sequence.

Add to section 49-3.02C(1):

If the piling center-to-center spacing is less than 4 pile diameters, do not drill holes or drive casing for an adjacent pile until 24 hours have elapsed after concrete placement in the preceding pile and your prequalification test results for the concrete mix design show that the concrete will attain at least 1800 psi compressive strength at the time of drilling or driving.

2. Use if drilling is anticipated through cobbles, boulders, or hard rock. Consult with Geotechnical Services and Structure Construction.

Drilling equipment must be equipped with instrumentation to measure accurately the actual downward force in pounds. Instrumentation must be visible for reading.

Section 49-3.02C(4). Use if CIDH piles are end bearing. Edit *3 inches* if different from the clearance shown.

Add to section 49-3.02C(4):

If the hole is drilled below the specified tip elevation shown, the reinforcement must extend to within 3 inches of the bottom of the drilled hole for the piles that are specified as end bearing.

Section 49-3.02C(6). Use for permanent steel casings. Delete methods of installation not allowed per the Foundation Report.

Add to section 49-3.02C(6):

Install permanent steel casings by impact or vibratory hammers, oscillators, rotators, or by placing in a drilled hole.

Section 49-3.02C(7). Use for Type II CIDH piling if (1) an optional construction joint is shown and the CIDH piling is less than 5 feet in diameter, (2) Geotechnical Services requires corrugated metal pipe for the casing, (3) Geotechnical Services allows the use of slurry cement backfill to backfill the casing, or (4) Geotechnical Services restricts the method of casing installation.

1. Use if an optional construction joint is shown and the CIDH piling is less than 5 feet in diameter.

Replace the 1st paragraph of section 49-3.02C(7) with:

Section 49-3.02C(7) applies to CIDH concrete piles if an optional construction joint is shown and you choose to construct the optional construction joint.

2–4. Use only one of these paragraphs.

Replace item 5 in the list in the 2nd paragraph of section 49-3.02C(7) with:

2. Use if corrugated metal pipe is required.

5. Be corrugated metal pipe and placed in a drilled hole. Casings placed in a drilled hole must comply with section 49-3.02C(6).

3. Use if slurry cement backfill is allowed. Delete methods not allowed for the installation of the casing.

5. Be installed by impact or vibratory hammers, oscillators, rotators, or by placing in a drilled hole. Casings placed in a drilled hole must comply with section 49-3.02C(6) except slurry cement backfill may be used instead of grout.

4. Use if not all methods listed are allowed for the installation of the casing. Delete methods not allowed.

5. Be installed by impact or vibratory hammers, oscillators, rotators, or by placing in a drilled hole. Casings placed in a drilled hole must comply with section 49-3.02C(6).

Section 49-3.03C(2). Use to revise the clean out depth of open-ended CISS piles.

Insert the number of feet that should not be cleaned out at the bottom of the pile. Eight feet is specified in the Standard Specifications. Generally, use at least 2 times the pipe diameter. Check the plans and foundation report.

Replace item 4 in the list in the 3rd paragraph of section 49-3.03C(2) with:

4. Bottom ___ feet of the pile must not be cleaned out.

Section 49-4.03B. Use for steel soldier piling.

Add to section 49-4.03B:

1. Use if rock foundation material or boulders with soil matrix foundation material is shown on the LOTB or foundation report.

Rock subsurface foundation material is anticipated at the soldier pile retaining wall location. Conventional drilling equipment for drilling in soils may not be suitable for drilling holes for the steel soldier piling.

2. Use if pile substitutions are allowed.

If you substitute piles with a larger diagonal dimension for the piles shown, ream or enlarge the drilled hole to provide a hole diameter at least 4 inches larger than the diagonal dimension of the pile.

Section 49-5. Use for micropiles. Micropiles are also known as pin piles, minipiles, root piles, or needle piles.

For purposes of editing this SSP, micropiles are divided into the following 2 categories:

Category A: Micropiles experiencing primarily axial forces due to axial loading at the top of the micropile. This would typically include bridge foundations and may occasionally include some walls. For Category A, the Contractor determines the length, load testing is performed, and pressure grouting is used.

Category L: Micropiles experiencing primarily shear and bending forces along their length due to lateral loading. This would typically include earth stabilization

structures and most walls. For Category L, the State determines the length, load testing is not performed, and gravity grouting is used.

Confirm the category with the designer. These categories are a guide to editing this SSP based on how Caltrans projects are typically done. However, nonstandard editing may be required for some projects.

For all micropiles, the plans must show:

1. Micropile layout, spacing, and inclination

2. Steel reinforcing element dimensions, yield strength, and any no-splice zones or no-joint zones

3. Grout compressive strength and grout cover

4. Anchorage details and amount of embedment into the concrete structure

5. Corrosion protection details, if required

For Category A micropiles only, the plans must show:

1. Value of FTL and whether it is for compression or tension load testing

2. Verification test micropile locations

3. Length of the pipe, HSS, or casing

4. Limits of the micropile length dimensioned as *Length to be determined by the Contractor*

5. If required, minimum micropile length per par. 98

For Category L micropiles only, the plans must show:

1. Micropile length or tip elevation

For all micropiles, the foundation report must discuss:

1. Any requirements for isolating micropiles from settling embankment

2. Any corrosion protection requirements

For Category A micropiles only, the foundation report must discuss:

1. Location of each verification test micropile and a description of which production micropiles are represented by each verification test micropile

2. Number of proof load tests required per footing or wall zone

3. For walls and earth stabilization structures, the limits of the wall zones

4. Whether verification and proof load testing are in tension, compression, or both

5. Whether creep testing is required for the verification or proof load testing

6. Whether measures need to be taken to provide geotechnical equivalence between the verification test micropile and the production piles it represents per par. 105

7. If required, minimum micropile length per par. 98

For Category L micropiles only, the foundation report must discuss:

1. Micropile length or tip elevation

Replace *Reserved* in section 49-5 with:

49-5.01 GENERAL

49-5.01A Summary

1

Section 49-5 includes specifications for constructing micropiles.

2. Use if the plans show an HS threaded bar with fy=120 ksi as a center bar. Delete *and couplers* if splices are not allowed.

HS threaded bars and couplers must comply with the specifications for HS steel prestressing bars in section 50.

3. Use if the plans show bar reinforcing steel as a center bar.

Bar reinforcing steel must comply with section 52, except you may use deformed bar reinforcing steel that complies with ASTM A615/A615M, Grade 60.

49-5.01B Definitions

4

micropile: Small-diameter, bored, CIP composite pile, in which the applied load is resisted by steel reinforcing elements, grout, and frictional ground-grout bond.

5

steel reinforcing element: Steel element used to strengthen or stiffen a micropile, such as bar reinforcing steel, HS threaded bar, pipe, hollow structural section (HSS), or casing.

49-5.01C Submittals

49-5.01C(1) General

6

Do not order materials nor install micropiles until the experience qualifications, shop drawings and calculations, and installation plan are authorized.

49-5.01C(2) Experience Qualifications

7. For Category L micropiles, replace *Construction details, structural details, and load test results* in item 2 with *Construction details and structural details*.

Submit the following experience qualification information:

1. Summary of the micropile subcontractor's experience that demonstrates compliance with section 49-5.01D(2).
2. Construction details, structural details, and load test results from at least 3 completed micropile installations performed by the micropile subcontractor in the last 5 years. The installations must be from 3 separate projects of similar scope to this Contract. Include a project description and the owner's name and current phone number.
3. List of on-site foremen and drill rig operators who will perform the micropile work and a summary of each individual's experience that demonstrates compliance with section 49-5.01D(2).

8. Edit if a different number of days is required for the review, except delete the paragraph if the number of days is 15.

Allow 10 days for review.

49-5.01C(3) Shop Drawings and Calculations

9

Submit 5 copies of micropile shop drawings and calculations to OSD, Documents Unit. Notify the Engineer of the submittal. Include in the notification the date and contents of the submittal.

10. Edit if a different number of days is required for the review, except delete the 1st sentence if the number of days is 20.

Allow 30 days for review. After the review, submit 6 copies for final authorization and use during construction. Within 20 days after final authorization, submit 1 copy of final shop drawings and calculations.

11

The shop drawings and calculations must be sealed and signed by an engineer who is registered as a civil engineer in the State.

12. Edit to suit the project. Delete item 7 for Category L micropiles. Delete item 3.8 if corrosion protection is not required. Delete item 6 if par. 80 is not used. Insert either *pipe*, *HSS*, or *casing* in the blank in item 6, according to the type of element specified in section 49-5.02B(2). If no splices or joints are allowed in any steel reinforcing elements, modify item 3.5 to read *Steel reinforcing element details, including sizes and lengths*.

The shop drawings and calculations must include:

1. Name, address, and phone number of the micropile subcontractor
2. Plan view, including:
   2.1. Station and offset at the beginning and end of the micropile structure and at any change in the structure's horizontal alignment
   2.2. Identification and location of each exploratory borehole
   2.3. Location of any existing utilities, adjacent existing structures, and other potential interferences
   2.4. Micropile layout and spacing
   2.5. Unique identification number for each micropile
3. Typical sections, including:
   3.1. Micropile inclination
   3.2. Drilled hole diameter
   3.3. Micropile tip elevation
   3.4. Micropile cutoff elevation
   3.5. Steel reinforcing element details, including sizes, lengths, splice or joint types, and splice or joint locations
   3.6. Centralizers and any spacers
   3.7. Micropile anchorage details
   3.8. Corrosion protection details
4. Material properties
5. General notes for constructing the micropiles, including overall construction sequencing
6. Data demonstrating the adequacy of any threaded joints in the _____ for the tension load specified in section 49-5.02B(2)
7. Calculations for the micropile length

13. Use if API N80 casing is shown on the plans. If par. 80 is not used, delete item 3.

If you propose an alternative to the API N80 casing shown, include with the shop drawings and calculations:

1. Calculations demonstrating that the structural capacity of the alternative pipe, HSS, or casing is greater than the structural capacity of the API N80 casing shown
2. Details and calculations for any anchorage changes needed to accommodate the alternative pipe, HSS, or casing
3. Data demonstrating the adequacy of any threaded joints for the value of tension specified in section 49-5.02B(2) if this value is greater than 25 percent of the yield capacity of the alternative pipe, HSS, or casing

49-5.01C(4) Installation Plan

14. Edit to suit the project. Delete items 3 and 4 if they do not apply. Delete item 6 if this requirement is not in the foundation report. Delete item 7 if no splices or joints are allowed in any steel reinforcing elements. For Category L micropiles: (1) delete items 9.6, 11, and 12, (2) delete *including details for post-grouting, if used* in item 9.3, (3) modify item 9.4 to read *Methods and equipment for monitoring and recording grout depth and volume as the grout is placed*, and (4) delete *initial pump pressures or* in item 9.5. Delete item 11.5 if par. 105 is not used.

Submit a micropile installation plan, including:

1. Detailed construction procedures, including personnel, materials, testing, and equipment.
2. Layout drawing showing the micropile installation sequence.
3. Information on headroom and space requirements for installation equipment that verifies that the equipment can perform at the job site.
4. Provisions for constructing micropiles near underground facilities.
5. Drilling or coring methods and equipment, including methods to:
   - 5.1. Provide drilled hole support
   - 5.2. Drill a straight hole
   - 5.3. Advance through boulders and other obstructions
   - 5.4. Prevent detrimental ground movements
6. Provisions for isolating micropiles from settling embankments.
7. Length of steel reinforcing element sections to be used, splice or joint locations, and any splice or joint location restrictions.
8. Methods for placing, positioning, and supporting steel reinforcing elements.
9. Grouting plan, including:
   - 9.1. Grout mix design. Include test results from an authorized laboratory for the compressive strength of the mix at 3, 7, 14, and 28 days and the density of the mix.
   - 9.2. Procedures for monitoring grout quality.
   - 9.3. Placement procedures and equipment, including details for post-grouting, if used.
   - 9.4. Methods and equipment for monitoring and recording grout depth, volume, and pressure as the grout is placed.
   - 9.5. Grouting rate calculations, upon request. Base the calculations on the initial pump pressures or static head on the grout and losses throughout the placing system, including anticipated head of drilling fluid to be displaced, if applicable.
   - 9.6. Minimum cure time and strength requirements for performing load testing.
10. Plan for the control and disposal of surface and groundwater, drill flush, and waste grout.
11. Load testing plan, including drawings and calculations that describe:
    - 11.1. Testing procedures.
    - 11.2. Reaction load system capacity and equipment setup.
    - 11.3. Types and accuracy of the primary and secondary instrumentation equipment to be used for applying and measuring the test loads and top of micropile movements.
    - 11.4. Installation details for the instrumentation to be used for applying and measuring the test loads and measuring the top of micropile movements.
    - 11.5. Provisions for isolating verification test micropiles as specified under section 49-5.03B.
12. Calibration reports and data for each test jack, pressure gauge, load cell, and electronic displacement transducer to be used. The load cell calibration chart must show applied load versus millivolts per volt.

15

The installation plan must be sealed and signed by an engineer who is registered as a civil engineer in the State.

16. Edit if a different number of days is required for the review, except delete the paragraph if the number of days is 15.

Allow 20 days for review.

49-5.01C(5) Mill Test Reports

17. If project special provisions include an exception to the Buy America specifications, consult with Structure Specifications Research and Development.

Submit certified mill test reports for each heat number of each type of steel reinforcing element at least 7 days before using the materials in the work. The certified mill test reports must include ultimate strength, yield strength, elongation, and chemical composition.

49-5.01C(6) Installation Logs

18. For Category L micropiles, delete *and pressures* in item 11. Delete item 13 if no splices or joints are allowed in any reinforcing elements.

Submit each installation log as an informational submittal within 1 business day of the micropile installation. The installation log must include:

1. Micropile identification number and location
2. Names of superintendent, drill rig operator, grout plant operator, and any other personnel involved in the micropile installation
3. Date, time, and duration of drilling, steel reinforcing element installation, and grout placement
4. Drilling or coring method and speed
5. Details of any hole stabilization method used
6. Description of soil or rock encountered
7. Quantity of groundwater encountered
8. Description of any unusual installation behaviors or conditions
9. Drilled hole diameter
10. Micropile tip elevation
11. Log of grout quantities and pressures, including the time and micropile depth
12. Lengths of steel reinforcing elements
13. Details of splicing operations, including locations of the splices or joints

49-5.01C(7) Grout Test Results

19

Submit grout test results for density, efflux time, and compressive strength within 1 business day of testing.

49-5.01C(8) Load Test Data

20–22. For Category L micropiles, delete pars. 20–22 and add *Not Used*.

20

Submit load test data within 1 business day of the completion of a verification or proof load test. Load test data must include:

1. Micropile identification number and location
2. Installation date
3. Load test date
4. Testing personnel
5. Load testing equipment
6. Raw data from the electronic data acquisition system
7. Readings from the secondary load and displacement measurement systems recorded at each load increment
8. Specified curves plotted using data from the electronic data acquisition system
9. Comparison of the load test results and the acceptance criteria

21

Load test data must be sealed and signed by an engineer who is registered as a civil engineer in the State.

22. Edit if a different number of days is required for the review, except delete the paragraph if the number of days is 15.

Allow 10 days for review.

49-5.01D Quality Assurance

49-5.01D(1) General

23

Not Used

49-5.01D(2) Experience Qualifications

24. For Category L micropiles, delete *and load testing* from item 1.

The micropile subcontractor must:

1. Be experienced in micropile construction and load testing
2. Have successfully constructed at least 5 projects in the last 5 years involving a combined total of at least 100 micropiles
3. Have previous micropile drilling and grouting experience in soil or rock similar to the soil or rock for this Contract

25

Each on-site foreman and drill rig operator must have experience installing micropiles on at least 3 projects completed in the last 5 years.

49-5.01D(3) Preconstruction Meeting

26

Schedule and hold a micropile preconstruction meeting at least 5 business days after submitting the micropile shop drawings, calculations, and installation plan and at least 10 days before starting micropile construction. You must provide a meeting facility.

27

The meeting must include the Engineer, your representatives, representatives from the micropile subcontractor, and representatives from any other subcontractor to be involved in the micropile construction.

28. For Category L micropiles, delete item 7.

The Engineer conducts the meeting. Be prepared to discuss:

1. Contractual relationships and delineation of responsibilities among you and the subcontractors
2. Contacts and communication protocol between you and your representatives, the subcontractors, and the Engineer
3. Coordination of the construction schedule and activities
4. Anticipated subsurface conditions
5. Structural, geotechnical, and construction requirements
6. Materials testing
7. Load testing

49-5.01D(4) Quality Control

49-5.01D(4)(a) General

29

Not Used

49-5.01D(4)(b) Grout Testing

30

Before placing grout into each micropile:

1. Test the grout density under API RP 13B-1 using the Baroid mud balance. Take the grout test samples directly from the grout plant.
2. Test the grout efflux time under California Test 541. Take the grout test samples at the point of placement.

31

Test the grout compressive strength under ASTM C109/C109M at an authorized laboratory. Test at least 1 set of three 2-inch grout cubes from each grout plant each day of operation or for every 10 micropiles installed, whichever occurs more frequently. Take the grout test samples directly from the grout plant.

32–59. For Category L micropiles, delete pars. 32–59, including the headings.

49-5.01D(4)(c) Load Testing

49-5.01D(4)(c)(i) General

32

Section 49-1.01D does not apply to micropile load testing.

33. Edit if a lower grout strength is allowed for load testing.

The grout for each load test micropile must attain the compressive strength shown before you perform the load test.

34

Notify the Engineer at least 10 days before you perform each load test.

35

Perform each load test in the Engineer's presence.

36. Use if tension load testing is recommended in the foundation report. If only verification load testing or only proof load testing is performed in tension, edit accordingly.

Perform verification and proof load testing in tension under ASTM D3689, except do not use the loading apparatus described as "Tensile Load Applied by Hydraulic Jack(s) Acting Upward at One End of Test Beam(s)."

37. Use if compression load testing is recommended in the foundation report. If only verification load testing or only proof load testing is performed in compression, edit accordingly.

Perform verification and proof load testing in compression under ASTM D1143/D1143M.

38. Use if both compression and tension verification load testing or both compression and tension proof load testing are recommended in the foundation report. Edit to suit the project. For proof load testing, edit as appropriate or add an additional requirement.

Perform tension and compression verification load tests on the same verification test micropile. Perform the tension load test first.

39. Use if tension load testing is recommended in the foundation report. In item 1, insert the value of FTL in tension from the plans. In item 2, insert the value of the service load in tension provided by the designer. Service load is not the same as design load. Service load is the unfactored demand in LRFD design. If the values of FTL and SL vary for different supports or wall zones, create a table.

For tension load testing:

1. FTL in the load test schedule must be equal to _____ kips
2. SL in the load test schedule must be equal to _____ kips

40. Use if tension load testing is performed on micropiles containing both a center bar and a pipe, HSS, or casing element. If the designer requires that the test load be applied to both the center bar and the pipe, HSS, or casing, change *the center bar only* to *both the center bar and the pipe/HSS/casing*. Choose either *pipe*, *HSS*, or *casing* according to the type of element specified in section 49-5.02B(2).

Apply the test loads for tension load testing to the center bar only.

41. Use if compression load testing is recommended in the foundation report. In item 1, insert the value of FTL in compression from the plans. In item 2, insert the value of the service load in compression provided by the designer. Service load is not the same as design load. Service load is the unfactored demand in LRFD design. If the values of FTL and SL vary for different supports or wall zones, create a table.

For compression load testing:

1. FTL in the load test schedule must be equal to _____ kips
2. SL in the load test schedule must be equal to _____ kips

42. Use if micropiles are near an existing structure.

Do not use an existing structure as part of the reaction system unless authorized.

43

Use a load cell as the primary load measurement system. The load cell must:

1. Be of the bonded electrical resistance strain gauge type.
2. Have a full scale range no greater than 150 percent of the maximum test load.
3. Be moisture resistant.
4. Be temperature compensated. The maximum temperature sensitivity at zero load must be ±0.05 percent of full scale per degree F.
5. Have a resolution within ±0.025 percent of full scale.
6. Have an accuracy within ±0.25 percent of full scale.

44

Use electronic displacement transducers as the primary movement measurement system. Displacement transducers must be capable of measuring to 0.001 inch and have enough travel to allow the load test to be performed without resetting.

45

Apply the test loads using a hydraulic jack. Use the gauge in the jack and pressure gauge assembly as the secondary load measurement system. Jack ram travel must be sufficient to allow the load test to be performed without resetting the equipment. The pressure gauge must be graduated in 100 psi increments or less.

46

The load cell, the electronic displacement transducers, and the jack and gauge assembly must be calibrated by an authorized laboratory accredited for calibration services using equipment traceable to NIST. The jack and gauge assembly must be calibrated as a unit.

47

Use an electronic data acquisition system to simultaneously monitor and record readings from the primary load and displacement measurement systems. The electronic data acquisition system must continuously take readings at regular intervals from the load cell and electronic displacement transducers.

48

The Department may verify the test loads using Department-furnished load cells. Upon request, furnish the resources necessary to install and support the Department's testing equipment at the load testing location and to remove the equipment after the testing is complete.

49-5.01D(4)(c)(ii) Verification Load Testing

49

Perform verification load testing on each verification test micropile installed.

50. Use for bridges. In column 1, insert the bridge number. In column 2, insert the location of each verification test micropile. In column 3, insert the abutment or bent names for the production micropiles represented by the verification test micropile. Add or delete rows as necessary.

The verification test micropile at each listed location represents the production micropiles at the support locations shown in the following table:

Bridge no.	Verification test micropile location	Support locations

51. Use for walls and slope stabilization structures. In column 1, insert the bridge number. In column 2, insert the location of each verification test micropile. In column 3, insert the beginning and end stations of the wall zone for the production micropiles represented by the verification test micropile. Add or delete rows as necessary.

The verification test micropile at each listed location represents the production micropiles in the wall zone shown in the following table:

Bridge no.	Verification test micropile location	Wall zone limits	
		Beginning station	End station

52. Use if creep testing is not recommended during verification load testing. If recommended by the geotechnical designer, the increments between 1.00SL and 1.00FTL can be made smaller by adding load increment rows.

Perform verification load testing as follows:

1. Incrementally load and unload the micropile as shown in the following table:

Verification Load Test Schedule

Load increment	Hold time (minutes)
AL	Until stable
0.25SL	1–2
AL	Until stable
0.25SL	1–2
0.50SL	1–2
AL	Until stable
0.25SL	1–2
0.50SL	1–2

0.75SL	1–2
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
1.00SL	5
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
1.00SL	1–2
0.80SL + 0.20FTL	1–2
0.60SL + 0.40FTL	1–2
0.40SL + 0.60FTL	1–2
0.20SL + 0.80FTL	1–2
1.00FTL[a]	5
0.75FTL	1–2
0.50FTL	1–2
0.25FTL	1–2
AL	Until stable

NOTES:
AL = alignment load, 0.10SL
SL = service load
FTL = factored test load
[a]Maximum test load

2. At each load increment:
   2.1. Apply the load in less than 1 minute.
   2.2. Maintain a constant load for the hold time shown in the load test schedule. Start the hold time as soon as the load increment is fully applied.
   2.3. Measure and record the top of micropile movement at the end of the hold time.
3. Plot the applied test load versus the top of micropile movement at each load increment.

53. Use if creep testing is recommended during verification load testing. If recommended by the geotechnical designer, the increments between 1.00SL and 1.00FTL can be made smaller by adding load increment rows.

Perform verification load testing as follows:

1. Incrementally load and unload the micropile as shown in the following table:

Verification Load Test Schedule

Load increment	Hold time (minutes)
AL	Until stable
0.25SL	1–2
AL	Until stable
0.25SL	1–2
0.50SL	1–2
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
AL	Until stable
0.25SL	1–2

0.50SL	1–2
0.75SL	1–2
1.00SL	10 or 60
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
1.00SL	1–2
0.80SL + 0.20FTL	1–2
0.60SL + 0.40FTL	1–2
0.40SL + 0.60FTL	1–2
0.20SL + 0.80FTL	1–2
1.00FTL[a]	5
0.75FTL	1–2
0.50FTL	1–2
0.25FTL	1–2
AL	Until stable

NOTES:
AL = alignment load, 0.10SL
SL = service load
FTL =factored test load
[a]Maximum test load

2. At each load increment:
   2.1. Apply the load in less than 1 minute.
   2.2. Maintain a constant load for the hold time shown in the load test schedule. Start the hold time as soon as the load increment is fully applied.
   2.3. Measure and record the top of micropile movement at the end of the hold time.
3. At the 1st application of 1.00SL, perform the creep test as follows:
   3.1. Hold the load for 10 minutes.
   3.2. Measure and record the top of micropile movement at 1, 2, 3, 4, 5, 6, and 10 minutes.
   3.3. If the movement measured from 1 to 10 minutes is greater than 0.04 inch, continue the creep test as follows:
      3.3.1. Hold the load for an additional 50 minutes.
      3.3.2. Measure and record the top of micropile movement at 15, 20, 25, 30, 45, and 60 minutes.
      3.3.3. Plot the top of micropile movement as a function of the logarithm of time from 6 to 60 minutes.
4. Plot the applied test load versus the top of micropile movement at each load increment.

49-5.01D(4)(c)(iii) Proof Load Testing

54–55. Use for micropiles at bridge supports. If there is more than 1 micropile footing and the number of proof test micropiles varies among footings, edit to be location specific.

54. Insert the minimum number of proof load tests recommended in the foundation report. Typically, 10 percent of the micropiles or a minimum of 2 micropiles are tested per footing.

Perform proof load tests on _____ micropiles per footing. The Engineer selects each micropile to be proof load tested. The Engineer does not notify you of which micropile is to be proof load tested until after the micropile has been installed.

55. Insert a value for percent of micropiles. A minimum of 25 percent is recommended.

The 1st micropile proof load test at a footing must be performed after at least ____ percent of the micropiles at the footing have been installed.

56–57. Use for micropiles at walls and slope stabilization structures. If there is more than 1 micropile wall or slope stabilization structure and the number of proof test micropiles per wall zone varies among walls and slope stabilization structures, edit to be location specific.

56. Insert the minimum number of proof load tests recommended in the foundation report. Typically, 15 percent of the micropiles or a minimum of 2 micropiles are tested per wall zone.

Perform proof load tests on _____ micropiles per wall zone. The Engineer selects each micropile to be proof load tested. The Engineer does not notify you of which micropile is to be proof load tested until after the micropile has been installed.

57. Insert a value for percent of micropiles. A minimum of 25 percent is recommended.

The 1st micropile proof load test in a wall zone must be performed after at least ____ percent of the micropiles in the wall zone have been installed.

58. Use if creep testing is not recommended during proof load testing. If recommended by the geotechnical designer, the increments between 1.00SL and 0.80FTL can be made smaller by adding load increment rows.

Perform proof load testing as follows:

1. Incrementally load and unload the micropile as shown in the following table:

Proof Load Test Schedule

Load increment	Hold time (minutes)
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
1.00SL	5
0.80SL + 0.20(0.80FTL)	1–2
0.60SL + 0.40(0.80FTL)	1–2
0.40SL + 0.60(0.80FTL)	1–2
0.20SL + 0.80(0.80FTL)	1–2
0.80FTL[a]	5
AL	Until stable

NOTES:
AL = alignment load, 0.10SL
SL = service load
FTL = factored test load
[a]Maximum test load

2. At each load increment:
   2.1. Apply the load in less than 1 minute.
   2.2. Maintain a constant load for the hold time shown in the load test schedule. Start the hold time as soon as the load increment is fully applied.
   2.3. Measure and record the top of micropile movement at the end of the hold time.
3. Plot the applied test load versus the top of micropile movement at each load increment.

59. Use if creep testing is recommended during proof load testing. If recommended by the geotechnical designer, the increments between 1.00SL and 0.80FTL can be made smaller by adding load increment rows.

Perform proof load testing as follows:

1. Incrementally load and unload the micropile as shown in the following table:

Proof Load Test Schedule

Load increment	Hold time (minutes)
AL	Until stable
0.25SL	1–2
0.50SL	1–2
0.75SL	1–2
1.00SL	10 or 60
0.80SL + 0.20(0.80FTL)	1–2
0.60SL + 0.40(0.80FTL)	1–2
0.40SL + 0.60(0.80FTL)	1–2
0.20SL + 0.80(0.80FTL)	1–2
0.80FTL[a]	5
AL	Until stable

NOTES:
AL = alignment load, 0.10SL
SL = service load
FTL = factored test load
[a]Maximum test load

2. At each load increment:
   2.1. Apply the load in less than 1 minute.
   2.2. Maintain a constant load for the hold time shown in the load test schedule. Start the hold time as soon as the load increment is fully applied.
   2.3. Measure and record the top of micropile movement at the end of the hold time.
3. At the 1st application of 1.00SL, perform the creep test as follows:
   3.1. Hold the load for 10 minutes.
   3.2. Measure and record the top of micropile movement at 1, 2, 3, 4, 5, 6, and 10 minutes.
   3.3. If the movement measured from 1 to 10 minutes is greater than 0.04 inch, continue the creep test as follows:
      3.3.1. Hold the load for an additional 50 minutes.
      3.3.2. Measure and record the top of micropile movement at 15, 20, 25, 30, 45, and 60 minutes.
      3.3.3. Plot the top of micropile movement as a function of the logarithm of time from 6 to 60 minutes.
4. Plot the applied test load versus the top of micropile movement at each load increment.

60–66. For Category L micropiles, delete pars. 60–66. Keep the heading *49-5.01D(5) Department Acceptance*, delete the other headings, and add *Not Used*.

49-5.01D(5) Department Acceptance

49-5.01D(5)(a) General

60

Not Used

49-5.01D(5)(b) Verification Load Test

61. Insert in items 1 and 2 the maximum axial movement allowed. If tension verification load testing is not being done, delete item 1, delete *For compression*

testing, the **in item 2, and capitalize *Axial* in item 2. If compression verification load testing is not being done, delete item 2, delete *For tension testing, the* in item 1, and capitalize *Axial* in item 1. Delete item 4 if verification creep testing is not being done.**

Each verification load test must comply with the following acceptance criteria:

1. For tension testing, the axial movement at the top of the micropile measured from the initial alignment load to the 1st application of 1.00SL must not exceed _____ inch at the end of the 1.00SL hold time.
2. For compression testing, the axial movement at the top of the micropile measured from the initial alignment load to the 1st application of 1.00SL must not exceed _____ inch at the end of the 1.00SL hold time.
3. Slope of the applied test load versus the top of micropile movement must not exceed 0.025 inch per kip at the maximum test load.
4. Creep test movement must comply with one of the following:
   4.1. For a 10-minute load hold, the movement measured from 1 to 10 minutes must be less than 0.04 inch.
   4.2. For a 60-minute load hold, the movement measured from 6 to 60 minutes must be less than 0.08 inch and the rate of movement must be linear or decreasing in time logarithmic scale from the 6- to the 60-minute reading.

62

If a verification load test fails to comply with the acceptance criteria, the verification test micropile is rejected. Revise the micropile length, installation methods, or both, and submit revised shop drawings, calculations, and installation plan.

63

After the revised submittals are authorized, install and test a new verification test micropile that incorporates the changes at an authorized location near the rejected verification test micropile. If post-grouting the micropile is the only change, you may post-grout and retest the rejected verification test micropile instead of installing a new verification test micropile.

64

If the new or retested verification test micropile fails to comply with the acceptance criteria, repeat the process specified above until a verification test micropile complies with the acceptance criteria.

49-5.01D(5)(c) Proof Load Test

65. Insert in items 1 and 2 the maximum axial movement allowed. If tension proof load testing is not being done, delete item 1, delete *For compression testing, the* in item 2, and capitalize *Axial* in item 2. If compression proof load testing is not being done, delete item 2, delete *For tension testing, the* in item 1, and capitalize *Axial* in item 1. Delete item 4 if proof creep testing is not being done.

Each proof load test must comply with the following acceptance criteria:

1. For tension testing, the axial movement at the top of the micropile measured from the initial alignment load to the 1st application of 1.00SL must not exceed _____ inch at the end of the 1.00SL hold time.
2. For compression testing, the axial movement at the top of the micropile measured from the initial alignment load to the 1st application of 1.00SL must not exceed _____ inch at the end of the 1.00SL hold time.
3. Slope of the applied test load versus the top of micropile movement must not exceed 0.025 inch per kip at the maximum test load.
4. Creep test movement must comply with one of the following:
   4.1. For a 10-minute load hold, the movement measured from 1 to 10 minutes must be less than 0.04 inch.
   4.2. For a 60-minute load hold, the movement measured from 6 to 60 minutes must be less than 0.08 inch and the rate of movement must be linear or decreasing in time logarithmic scale from the 6- to the 60-minute reading.

66. For walls and slope stabilization structures, replace *footing* with *wall zone* at 2 locations in item 1 and at 1 location in item 2.

If a proof load test fails to comply with the acceptance criteria, the micropile is rejected. Suspend micropile construction and comply with one of the following procedures:

1. Post-grout and retest the rejected micropile. If the post-grouted micropile complies with the acceptance criteria when retested, post-grout all of the micropiles in the footing using identical methods. Any proof load test performed on a micropile before the post-grouting does not count toward the total proof load tests required for the footing.
2. Proof load test all the micropiles in the footing that have been constructed unless otherwise authorized. Submit a plan for replacing rejected micropiles or for installing additional micropiles, including details for any micropile or footing changes required to provide the total micropiling support capacity shown. Suspend micropile construction until the plan is authorized.

49-5.02 MATERIALS

49-5.02A General

67. Delete for Category L micropiles.

Use identical materials and element sizes for a verification test micropile as to be used for the production micropiles it represents.

68

Welding must comply with AWS D1.1.

49-5.02B Steel Reinforcing Elements

49-5.02B(1) General

69. If project special provisions include an exception to the Buy America specifications, consult with Structure Specifications Research and Development.

Mill secondary steel reinforcing elements must not be used.

70. Use if steel reinforcing elements are not allowed to contain any splices or joints. Edit if some types of reinforcing elements are allowed to contain splices or joints and other types are not, and move the requirement to section 49-5.02B(2) or 49-5.02B(3) if appropriate.

Steel reinforcing elements must not contain splices or joints.

49-5.02B(2) Pipe, Hollow Structural Sections, and Casing

71–74. Edit the above heading to show only the type of element that is specified below. Use 1 paragraph and delete the other 3 paragraphs. If the designer chooses to use a steel designation other than the 4 options below, edit accordingly. Delete pars. 71–80 and add *Not Used* if no pipe, HSS, or casing is shown on the plans.

71. Use if the plans show pipe with fy=35 ksi.

Pipe must comply with ASTM A53/A53M, Type E or S, Grade B.

72. Use if the plans show HSS with fy=42 ksi.

HSS must comply with ASTM A500/A500M, Grade B.

73. Use if the plans show HSS with fy=46 ksi.

HSS must comply with ASTM A500/A500M, Grade C.

74. Use if the plans show casing with fy=80 ksi.

Casing must comply with API N80.

75–76. Use if API N80 casing is shown on the plans.

75. Insert the value of the maximum hole diameter provided by the designer. This value must not exceed 13 inches or 1/3 of the micropile center-to-center spacing. Delete item 4 if no grout is shown between the casing and the soil. Delete item 5 if corrosion protection is not recommended in the foundation report or if the corrosion protection does not include a sacrificial thickness.

You may use alternative pipe, round HSS, or casing instead of the API N80 casing shown if:

1. Structural capacity of the alternative pipe, HSS, or casing is greater than or equal to the structural capacity of the API N80 casing shown
2. Alternative pipe, HSS, or casing complies with the requirements for casing specified in section 49-5
3. Drilled hole diameter does not exceed _____ inches
4. Grout cover on the alternative pipe, HSS, or casing is greater than or equal to the grout cover on the API N80 casing shown
5. Thickness of the alternative pipe, HSS, or casing is greater than or equal to the thickness of the API N80 casing shown

76

Alternative pipe, HSS, or casing must comply with one of the following specifications or another authorized specification:

1. ASTM A1085/A1085M
2. API N80
3. API P110
4. API 5L, minimum PSL1

77. Insert either *Pipe*, *HSS*, or *Casing* in the blank, according to the type of element specified in pars 71–74.

_____ to be welded for structural purposes must have a carbon equivalency as defined in AWS D1.1, Annex H5.1, not exceeding 0.47 percent and a sulfur content not exceeding 0.05 percent.

78. Delete *and splices* if no welded splices are allowed in the pipe, HSS, or casing.

Welded seams and splices must be CJP welds.

79. Delete if no welded splices are allowed in the pipe, HSS, or casing.

Circumferential welds must comply with section 49-2.02B(1)(b).

80. Use if the following 3 conditions apply: (1) pipe, HSS, or casing is shown, (2) the pipe, HSS, or casing is designed to provide tension resistance, and (3) the tension resisted by the pipe, HSS, or casing exceeds 25 percent of the yield capacity of the pipe, HSS, or casing. Insert either *pipe*, *HSS*, or *casing* according to the type of element specified above. Insert the value of tension provided by the designer. Delete if no threaded joints are allowed in the pipe, HSS, or casing.

Threaded joints for the _____ must be capable of developing at least ____ kips in tension.

49-5.02B(3) Bar Reinforcing Steel

81–82. Delete if bar reinforcing steel is not shown on the plans, and delete the section heading.

81. Delete if no splices are allowed in the bar reinforcing steel.

Bar reinforcing steel splices must be service splices.

82

For anchorages that require threading nuts and plates onto bar reinforcing, you may cut threads into the bar reinforcing steel if you provide the next larger bar number designation from that shown.

49-5.02C Anchorage Components

83. Use if the plans show stud connectors or concrete anchors. If the plans show concrete anchors, change *Stud connectors* to *Concrete anchors*.

Stud connectors for the micropile anchorage must comply with the specifications for studs in clause 7 of AWS D1.1.

84. Use if the plans show fy=50 ksi for the anchorage plates.

Steel plates for the micropile anchorage must comply with ASTM A709/A709M, Grade 50, or ASTM A572/A572M, Grade 50.

85. Use if the plans show fy=36 ksi for the anchorage plates.

Steel plates for the micropile anchorage must comply with ASTM A709/A709M, Grade 36, or ASTM A36/A36M.

86. Use if the anchorage detail on the plans shows nuts.

Nuts and washers for the micropile anchorage must be capable of holding the bar at a load producing a tensile stress of at least the specified minimum ultimate tensile strength of the bar.

49-5.02D Grout

87

Grout must be a stable, neat grout consisting of cement and water. Cement must comply with section 90-1.02B(2). Water must comply with section 90-1.02D.

88

If authorized, you may use an admixture in the grout. The admixture must comply with sections 90-1.01C(4) and 90-1.02E, except the admixture must not contain chloride ions in excess of 0.25 percent by weight nor be an accelerating admixture.

89

If authorized, you may add fine aggregate to the grout. Fine aggregate must comply with section 90-1.02C(3). Grout with fine aggregate must:

1. Have a slump of at least 7 inches when measured under ASTM C143/C143M
2. Have an air content of no more than 2 percent when measured under California Test 504
3. Not contain air-entraining admixtures

90

Mix the grout as follows:

1. Add the water to the mixer followed by the cement and any admixtures or fine aggregate.
2. Mix the grout with mechanical mixing equipment that produces a uniform and thoroughly mixed grout.
3. Agitate the grout continuously until the grout is pumped.
4. Do not add water after the initial mixing.

91

Grout must comply with the following requirements:

1. Density must be greater than or equal to the density submitted with the authorized mix design.
2. Efflux time must be at least 11 seconds.
3. Compressive strength must be at least that shown at 28 days.

49-5.02E Centralizers and Spacers

92

Centralizers and spacers must be fabricated from plastic, steel, or other material that is not detrimental to the steel reinforcing elements. Do not use wood centralizers or spacers.

93

Centralizers and spacers must be strong enough to support the steel reinforcing elements during construction activities.

49-5.02F Corrosion Protection

94–97. If corrosion protection is not recommended in the foundation report, delete pars. 94–97 and add *Not Used*. Edit accordingly if the corrosion protection is required only at certain locations.

94. Use if encapsulation is recommended for bar reinforcing steel or HS threaded bars. Insert either *Bar reinforcing steel* or *HS threaded bars* in the first blank and insert either *pipe*, *HSS*, or *casing* in the second blank, according to what is shown on the plans.

_____ must be encapsulated. The encapsulation must:

1. Be shop fabricated using HDPE corrugated tubing that complies with ASTM D3350, Type III, with UV stabilizer
2. Have a minimum nominal wall thickness of 60 mils
3. Provide at least a 0.20-inch inside annulus between the bar and the tubing that is fully grouted before installation
4. Be watertight
5. Be capable of:
   5.1. Transferring stresses from the grout surrounding the bar to the grout bonding to the ground
   5.2. Withstanding abrasion, impact, and bending during handling and installation
   5.3. Resisting internal grouting pressures
   5.4. Resisting chemical attack from aggressive environments and grout
6. Be cleaned of oil, grease, dirt, and other extraneous substances and have any damage repaired or replaced before installation
7. Extend at least 5 feet into the _____

95. Use if epoxy coating is recommended for bar reinforcing steel or HS threaded bars. Insert either *Bar reinforcing steel* or *HS threaded bars* in the first blank and insert either *pipe*, *HSS*, or *casing* in the second blank, according to what is shown on the plans.

_____ must be epoxy coated. The epoxy coating must:

1. Comply with section 52-2.03, except the bend test requirements are waived and the epoxy thickness must be from 10 to 12 mils
2. Extend at least 5 feet into the _____

96. Use if galvanization is recommended for bar reinforcing steel.

Bar reinforcing steel must be galvanized under section 52-3.

97. Delete for Category L micropiles. Insert the type of corrosion protection required above for production micropiles: Encapsulation, Epoxy coating, or Galvanizing.

_____ is not required for steel reinforcing elements used in verification test micropiles.

49-5.03 CONSTRUCTION

49-5.03A General

98–100. Delete for Category L micropiles.

98. Tip elevations for tension and compression must not be shown on the plans. Delete the 2nd sentence unless required by the designer, and use it only for a load case other than tension or compression. Insert the load case (e.g., lateral load, group effect, etc.).

Determine the micropile length and installation methods necessary to comply with the micropile load test acceptance criteria. Your proposed micropile tip elevation must not be higher than that shown in the Pile Data Table for _____.

99

You may perform additional geotechnical investigation for the purpose of determining the micropile length and installation methods.

100

Do not construct any production micropiles until the test results are authorized for the verification test micropile that represents the production micropiles.

101. The values for 5 feet and 12 hours may be edited if other values are recommended in the foundation report. For Category L micropiles, delete *, pressure grout, or post-grout*.

Do not drill, pressure grout, or post-grout a micropile that is within a center-to-center spacing of 5 feet from an open micropile hole or a micropile in which the initial grout has set for less than 12 hours.

49-5.03B Verification Test Micropiles

102–107. For Category L micropiles, delete pars. 102–107 and add *Not Used* under each heading.

102

Install a verification test micropile at each location shown. Notify the Engineer at least 7 days before installing a verification test micropile.

103. Edit accordingly if the test site is not supposed to be level.

Excavate the verification load test site as necessary to provide a level work area. Keep the test site free of water throughout the testing.

104

Construct each verification test micropile in the Engineer's presence. Use identical drilling and grouting methods, inclination, tip elevation, and dimensions as to be used for the production micropiles it represents.

105. Use if (1) the foundation report recommends taking measures to provide geotechnical equivalence between the verification test micropile and the production micropiles it represents and (2) if these measures are not shown on the plans. Insert the location of the verification test micropile, or delete *at* _____ if there is only 1 verification test micropile on the project. Edit if this paragraph applies to more than 1 verification test micropile. Insert the elevation to which isolation is required, or edit the paragraph if measures other than isolation are recommended.

Isolate the verification test micropile at _____ down to an elevation of _____ such that the soil above this elevation is not engaged.

106

After the verification load test results are authorized, remove the verification test micropile and any anchor piles as specified for removing portions of bridges in section 60-2.02.

49-5.03C Proof Test Micropiles

107. Edit accordingly if the test site is not supposed to be level.

Throughout proof load testing, maintain the test site in a condition that is level and free of water.

49-5.03D Drilling

108

Select drilling equipment and methods that are suitable for drilling through the conditions to be encountered without causing damage to any overlying or adjacent structure or service and without causing detrimental ground movements.

109

Use temporary casing or another authorized drilled hole support method in caving or unstable ground.

110. Use if micropiles are constructed near settlement-sensitive structures.

Do not use vibratory pile-driving hammers to advance casing.

111. Use if coring through concrete (e.g., adding micropiles to an existing footing).

Core through concrete structures using methods that do not shatter or damage the concrete adjacent to the hole.

112

Each drilled hole must comply with the following tolerances:

1. Centerline of the drilled hole must not deviate from the micropile location shown by more than 3 inches.
2. Center-to-center spacing of the drilled holes must not deviate from the micropile spacing shown by more than 3 inches.
3. Axis of the drilled hole must not deviate from the alignment shown by more than 1-1/2 inches per 10 feet of length.

113

Remove any material dislodged or drawn into the hole during micropile construction. The drilled hole must be open along its full length to the hole diameter shown before placing grout or any steel reinforcing elements not used to case the drilled hole.

114

Dispose of drill cuttings under section 19-2.03B.

49-5.03E Placing and Splicing Steel Reinforcing Elements

115

Place the steel reinforcing elements before withdrawing any temporary casing.

116. Delete if no splices or joints are allowed in any reinforcing elements, and also delete *and Splicing* in the heading above.

Splice the steel reinforcing elements such that the axes of the 2 spliced lengths are aligned.

117. Use if both a center bar and a pipe, HSS, or casing are shown on the plans. Delete if bar splices are not allowed or if pipe, HSS, or casing threaded joints are

not allowed. Insert either *pipe*, *HSS*, or *casing* according to the type of element specified in section 49-5.02B(2).

If the _____ has threaded joints, locate any bar splices at least 2 _____ diameters from any threaded joint.

118. Use if both a center bar and a pipe, HSS, or casing are shown on the plans.

Use spacers to separate steel reinforcing elements. Place the spacers at 10-foot maximum intervals.

119. Confirm that the plans show centralizers that comply with these requirements. Edit *10-foot* if the designer requests a smaller maximum interval.

For steel reinforcing elements not used to case the drilled hole, use centralizers to support the element in the center of the hole and to provide at least the specified grout cover. Place the centralizers at 10-foot maximum intervals, with the uppermost centralizer a maximum of 5 feet from the top of the micropile and the lowermost centralizer from 2 to 5 feet from the bottom of the micropile.

120. Delete both occurrences of *and spacers* if par. 118 is deleted.

Attach centralizers and spacers to the steel reinforcing elements such that the centralizers and spacers (1) are secure enough to withstand installation stresses and (2) allow the free flow of grout without misalignment of the steel reinforcing elements.

121

Before you insert each steel reinforcing element into a drilled hole, clean the surface of the element of deleterious substances, such as soil, mud, grease, and oil.

122

If you cannot insert a steel reinforcing element into the drilled hole to the required depth without difficulty, remove the reinforcing element, clean any grout from the surface of the reinforcing element, clean or redrill the hole, and reinsert the reinforcing element. Do not force or drive a reinforcing element into a drilled hole. Micropiles with partially inserted steel reinforcing elements are rejected.

49-5.03F Grouting

123

Grout each micropile the same day the hole is drilled.

124

You may place the grout before or after placing the steel reinforcing elements.

125

Place the grout within 1 hour of mixing.

126

Inject the grout at the lowest point of the drilled hole. Continue the injection until uncontaminated grout flows from the top of the micropile.

127

Grout each micropile in 1 continuous operation. Use grouting procedures that ensure complete continuity of the grout column.

128

If temporary casing is used, extract the casing in stages. After you remove each length of casing, bring the grout level back up to ground level before removing the next length of casing. Maintain the grout at a level above the bottom of the temporary casing adequate to prevent displacement of the grout by material from outside the casing. The tremie pipe or casing must extend at least 10 feet below the grout level in the drilled hole at all times during grout placement.

129. Delete for Category L micropiles.

If grout is placed under pressure:

1. Measure and record the grout quantity and pumping pressure
2. Use a grout pump equipped with a pressure gauge
3. Place a 2nd pressure gauge at the point of injection into the top of the micropile
4. Use pressure gauges capable of measuring pressures of at least 150 psi or twice the actual grout pressure used, whichever is greater
5. Do not use compressed air to directly pressurize the fluid grout

130

Grout tubes may remain in the hole after the completion of grouting but must be filled with grout before pile acceptance.

131

Maintain the grout level at or above the micropile cutoff elevation until the grout has set.

132

Provide a positive means of support for maintaining the position of the steel reinforcing elements until the grout has set.

133. Delete for Category L micropiles.

Load test micropiles must remain undisturbed until the grout is strong enough to provide anchorage during load testing.

134

Dispose of material resulting from grouting.

49-5.03G Ground Heave and Subsidence

135. Delete for Category L micropiles.

Control the grout pressures and grout takes to prevent heave and fracturing of soil or rock formations.

136

If you observe signs of ground heave or subsidence, immediately notify the Engineer and suspend the drilling and grouting operations. If the Engineer determines that the movements require corrective action, take the actions necessary to stop the movement and perform repairs.

49-5.03H Installation Logs

137

Prepare a separate installation log for each micropile.

49-5.04 PAYMENT

138

Section 49-1.04 does not apply.

139. Delete for Category L micropiles.

Verification test micropiles are paid for as micropiles.

Section 58-2.03A. Use for sound walls on piles or trench footings that reference Standard Plan B15-1, B15-5, B15-8, or B15-15.

Insert the sound wall location and the angle of internal friction. Check with Geotechnical Services/Office of Structure Design for the value of ϕ. Add a table of locations if necessary.

Add to section 58-2.03A:

The angle of internal friction (ϕ) to be used with the plans for the soil at sound wall _____ is _____.

Section 61-2.01D(3). Use for field leakage testing of culvert and drainage pipes (1) where settlement is expected to be in excess of 3 inches, (2) for areas having fine cohesionless soils, or (3) whenever infiltration or exfiltration water quality issues exist (e.g., water permits, local agency regulations, etc.).

1. Use if settlement of a pipeline is expected to be in excess of 3 inches. Modify the number of days as necessary to ensure adequate settlement of embankment.

Replace the 10th paragraph of section 61-2.01D(3)(a) with:

Perform final leakage testing of culverts and drainage pipes from 30 to 45 days after backfilling around the pipe unless a different time period is authorized.

2. Use for fine cohesionless soils.

Replace 1000 in the 1st sentence of item 2 of the 1st paragraph of section 61-2.01D(3)(b) with:

600.

3. Use if infiltration or exfiltration water quality issues exist, (e.g., water permits, local agency regulations, etc.)

Replace 1000 in the 1st sentence of item 2 of the 1st paragraph of section 61-2.01D(3)(b) with:

200.

Section 65-2. Miscellaneous RCP requirements for freeze-thaw and corrosive environments, difficult excavation, and timber bulkheads.

1–2. Use when reinforced concrete pipe is allowed on projects constructed at elevations above 3,000 feet. Otherwise delete pars. 1 and 2.

1

Add to section 65-2.01C:

Submit a certificate of compliance for wet-cast pipe and test reports for measured air entrainment.

2

Add to section 65-2.02A:

Wet-cast pipe must be made from concrete placed and consolidated by conventional equipment using concrete with a slump of 2 inches or more. Wet-cast pipe must contain 5.5 ± 1.5 percent air by volume determined under ASTM C231.

3. Use if the designer indicates that results from AltPipe software require adjustment to concrete mix design and/or minimum cover over the reinforcement due to corrosive conditions. Edit the table by adding cementitious material content values as provided by AltPipe. If corrosive conditions are not indicated, delete par. 3.

Replace the 2nd paragraph in section 65-2.02A with:

For cementitious material content, use one of the options shown in the following table. You may use SCM.

Minimum Cementitious Material Content in Pounds per CY

Minimum Concrete Cover	Minimum cementitious material content based on maximum water to cementitious material ratio	
	0.35	0.40
1.00 inch	----lb/cu yd	----lb/cu yd
1.25 inches	----lb/cu yd	----lb/cu yd
1.50 inches	----lb/cu yd	----lb/cu yd

4–5. Use if (1) it is anticipated that rock or other unyielding material will be encountered in the excavation below the bottom of the bedding shown on Standard Plan A62DA; otherwise delete pars. 4 and 5.

Include supplemental funds in the Engineer's Estimate for excavation work.

Add to section 65-2.03B:

4

If you encounter solid rock or other unyielding material at the planned elevation of the bottom of the bedding shown, remove the material below the bottom of the bedding to a depth of 1/50 of the height of the embankment over the top of the culvert but not less than 6 inches or more than 12 inches. Backfill the resulting trench below the bottom of the bedding with structure backfill material under section 19-3.03E. Do not compact the outer bedding before pipe placement.

5

The excavation and backfill below the planned elevation of the bottom of the bedding shown is change order work.

6. Use if timber bulkheads are shown; otherwise delete. If used, show timber bulkheads on the project plans.

Add to section 65-2.03B:

Construct and place timber bulkheads across the ends of unconnected reinforced concrete. Wood for timber bulkheads must be construction heart grade redwood at least 1 inch thick.

Section 68-2.02F(1). Use to specify Class 2 or Class 3 permeable material for underdrains.

1. Insert the Class 2 or Class 3 as recommended by the Office of Roadway Geotechnical Engineering. Edit as directed by the Office of Roadway Geotechnical Engineering.

Replace paragraph 3 of section 68-2.02F(1) with:

Use ____ permeable material for underdrains.

Section 68-5. Use if permeable material blanket is described.

Replace *Reserved* in section 68-5 with:

68-5.01 GENERAL

1

Section 68-5 includes specifications for installing permeable material blankets.

68-5.02 MATERIALS

2. Insert the class of permeable material.

Permeable material for permeable material blanket must be Class _____ and must comply with section 68-2 except for payment.

3

Filter fabric must comply with section 96-1.02B.

68-5.03 CONSTRUCTION

4

Place filter fabric as follows:

1. Ensure the subgrade complies with the compaction and elevation tolerance specified for the material involved before placing the filter fabric on the subgrade.
2. Handle and place filter fabric under the manufacturer's instructions.
3. Align and place the fabric without wrinkles.
4. Overlap or stitch adjacent borders of the fabric from 12 to 18 inches. The preceding roll must overlap the following roll in the direction the permeable material is being spread or must be stitched. If the fabric is joined by stitching, the fabric must be stitched with yarn of a contrasting color. The size and composition of the yarn must be as recommended by the fabric's manufacturer. There must be 5 to 7 stitches per inch of seam.
5. Cover the fabric with the planned thickness of permeable material or aggregate subbase material as shown within 24 hours after the filter fabric has been placed.
6. Maintain at least 6 inches of the material between the fabric and your equipment during spreading and compaction of the permeable material and aggregate subbase. Where embankment material is to be placed on the filter fabric, maintain at least 18 inches of embankment material between the fabric and your equipment. Do not operate or drive equipment or vehicles directly on the filter fabric.

68-5.04 PAYMENT

Not Used

Section 69-2. Use for geomembrane and cushion fabric.

When not using cushion fabric, delete *AND CUSHION FABRIC* in the title.

Replace section 69-2 with:

69-2 GEOMEMBRANE AND CUSHION FABRIC

69-2.01 GENERAL

69-2.01A Summary

1

Section 69-2 includes specifications for placing geomembrane for overside drains.

69-2.01B Definitions

2

Not Used

69-2.01C Submittals

3. Edit when not using cushion fabric and when not seaming.

Submit before installation:

1. Shop drawings, which must include:
   1.1. Proposed panel layout identifying the seams and details
   1.2. Construction equipment and method
2. Quality control plan for:
   2.1. Material handling
   2.2. Storage
   2.3. Installation
   2.4. Geomembrane and seam tests
   2.5. Repair
3. Test seam samples

69-2.01D Quality Assurance

4. ASTM D4437 provides options for nondestructive testing, which includes ASTM D5641, ASTM D5820, ASTM D6365, and ASTM D7006. If any specific tests are preferred, edit as appropriate.

Perform nondestructive testing of geomembrane seams over the full length of seamed tracks under ASTM D4437.

5

Perform destructive testing of geomembrane seams under GSI GRI Test Method GM14 and GM19 and ASTM D7747 and ASTM D7749.

6

When tested under the ASTM D7747 and ASTM D7749, the seam strength and related properties of geomembrane must have the values of the geomembrane with the same thickness and formulation as shown in GSI GRI Test Method GM19.

7

Mark, number, measure, and report each location that fails nondestructive testing.

69-2.02 MATERIALS

8. Specify the application and the class of geomembrane based on the required degree of survivability. For multiple applications, copy par. 11 and edit.

Use Class A for condition that requires very high survivability as instructed in the Geotechnical Design Report.

Use Class B for condition that requires high survivability as instructed in the Geotechnical Design Report.

Use Class C for condition that requires medium survivability as instructed in the Geotechnical Design Report.

Geomembrane for ____ must be Class __.

9. Insert *smooth* or *textured* as instructed in the Geotechnical Design Report.

Geomembrane must be _____ on both sides.

10. Edit as necessary.

Geomembrane must be:

1. Water and gasoline resistant
2. Free from holes, bubbles, blisters, and contamination by foreign matter
3. Be able to conform to subgrade irregularities by its own weight without heating, bending, or overburden placed on it

11. Edit as instructed in the Geotechnical Design Report.

Cushion fabric must be Class __.

69-2.03 CONSTRUCTION

12

Remove loose or extraneous material, large rocks, and sharp objects from the subgrade that may come in contact with the geomembrane and cushion fabric. Compact subgrade to receive geomembrane under section 19-5.03B.

13. Edit for cushion fabric.

Place geomembrane and cushion fabric as shown.

14. Delete if not using cushion fabric.

Cushion fabric must be placed below and above geomembrane.

15. Edit for cushion fabric.

Geomembrane and cushion fabric must not be installed:

1. In standing water
2. While raining
3. During strong winds
4. When the temperature is below 32 degrees F

16

The geomembrane must rest in intimate contact with the subgrade.

17. When not using cushion fabric, delete *or cushion fabric*.

Do not operate equipment or vehicles directly on geomembrane or cushion fabric.

18. Delete when not using cushion fabric.

Cushion fabric pieces must be shingled. Overlaps must be at least 12 inches. Patch or replace cushion fabric damaged during construction. The patch material must extend at least 18 inches beyond damaged area.

19

Place geomembrane in a manner that minimizes handling and seaming.

20. Use when welding of geomembrane sheets is not required as instructed in the Geotechnical Design Report.

Geomembrane sheets must be shingled with the upstream piece on top of the downstream piece. Overlaps must be at least 18 inches. Geomembrane sheets must be seamed by welding or solvent-bonding at subgrade with slopes 5 percent or less.

21. Use when on-site welding of geomembrane sheets is required as instructed in the Geotechnical Design Report.

Geomembrane sheets must be welded or solvent-bonded. Overlap must be at least 6 inches. The seaming overlap must be smooth and free of wrinkles. Seam must be at least 1 inch wide. Seams must follow the direction of the slope. Butt seams or roll-end seams are not allowed.

22

Patch or replace geomembrane damaged during construction. The patch material must have rounded corners and extend at least 6 inches on all sides of damaged area. Patched and damaged geomembrane must be field seamed by welding or solvent-bonding.

23. Delete when anchor trenches are not shown. Edit for cushion fabric.

Anchor the transverse and longitudinal edges of the geomembrane and cushion fabric in trenches at least 8 inches deep.

24. Delete when there are no culverts, transitional devices, or headwalls shown.

At the joint between geomembrane and culvert, culvert transitional device, or headwall:

1. No cushion fabric is required
2. Overlaps must be:
    2.1. At least 6 inches
    2.2. Closely fitted
    2.3. Secured
    2.4. Sealed watertight

25. Delete when not using rocks over the geomembrane.

Place rocks such that the rock drop will not damage the geomembrane and cushion fabric. Place rocks from downstream and up slope. Minimize spreading of rocks to avoid damaging geomembrane and slippage of cushion fabric.

69-2.04 PAYMENT

26

The payment quantity for geomembrane does not include the additional quantity used for overlaps.

Section 71-3.03. Use for injecting grout from within the culvert to fill voids between culvert outer wall and the surrounding ground. The culvert must be at least 60 inches in diameter and human entry must be possible.

If the only work on the project is culvert contact grouting, include a bid item for mobilization.

Replace *Reserved* in section 71-3.03 with:

71-3.03A General

1

Section 71-3.03 includes specifications for drilling, probing voids, installing grout ports, pumping grout from within the culvert, and filling voids between the culvert outer wall and the surrounding ground.

2

Submit a grout plan under section 71-3.01A(3)(d) and include the tabulated probe results. Obtain authorization before starting grouting.

71-3.03B Materials

3

Not Used

71-3.03C Construction

4

Before starting contact grouting of the culvert, assemble equipment and materials at the job site.

5

Grouting activities must comply with section 13. Furnish pumps, if necessary, to remove drill cuttings, wastewater, and excess grout.

6. Edit to suit the work. Grout port locations must be shown on the plans, and grout port location information may be included in the geotechnical recommendations. Delete third sentence if not applicable.

Install grout ports at the 2, 4, 8, and 10 o'clock circumferential positions every 6 feet along the culvert as shown. Install valves or removable plugs at grout ports to verify grout coverage and control grout flow. Extend grout ports through the invert paving using steel pipes or suitable packers. Grout ports must be watertight. If authorized, you may screw grout ports in place or attach them by other methods. Do not weld grout ports to galvanized surfaces.

7

Probe at each grout port location. The probe must be at least 4 feet long, fit through the grout ports, and be rigid enough to sense probe refusal.

8. If this work is anticipated, include supplemental funds

If ordered, hammer soundings and additional probing for voids is change order work.

9

Prevent deformation of the culvert and culvert lining during grouting.

10. If grouting around a concrete culvert or a metal culvert with a concrete lining add "or if cracking of concrete occurs" to the 2nd sentence.

Pump grout into voids until it appears that all water and air has been ejected. Plug grout ports or close port valves as soon as you stop pumping the grout.

11

The maximum injection pressure at the nozzle must not exceed 5 psi for fluid, unsanded grout mix.

12

Monitor the culvert for deformation and cracks. If cracking occurs in a concrete culvert or lining, reduce the grout injection pressure. If deformation of the existing structure exceeds 1/2 inch at any location, reduce the grout injection pressure.

13

Repair any permanent deformations or cracks resulting from your grouting work. The Department does not pay for these repairs.

71-3.03D Payment

14

Record the quantity of grout that is installed and submit this quantity. The Department does not pay for grout that leaks through to the inside of the culvert. The Department does not pay for grout material that is wasted, disposed of, or remaining on hand after completion of the work.

Section 72-2.02C. Use for nonstandard fabric for RSP.

1. Edit to suit the work.

Replace the paragraph in section 72-2.02C with:

Fabric must comply with the specifications for _____.

Section 96-1.02B. Use for projects with filter fabric.

Add to section 96-1.02B:

1. Specify the application and the class of filter fabric (based on testing of soil on the job site). For multiple applications, copy paragraph 1 and edit. If the soil is not tested, specify Class A as the default.

Use Class A if less than 15 percent passes a no. 200 sieve.

Use Class B if 15 to 50 percent passes a no. 200 sieve or the plasticity index is more than 7.

Use Class C if more than 50 percent passes a no. 200 sieve.

Filter fabric for ____ must be Class __.

Geotechnical Investigations

This module addresses geotechnical investigations. The information presented is general in that it applies to most geotechnical investigations regardless of the purpose or type(s) of proposed improvement(s). Specifics on the appropriate types and extent of explorations are provided in the individual design modules.

The three references cited at the end of this module contain relevant information that is not provided here and it is recommended that the Geoprofessional (GP) become familiar with their contents as they may serve as valuable references for investigation work.

This module documents the standards and procedures for both planning (K and 0 phase) investigations and design (1 phase) investigations. Geotechnical investigations are conducted in the office and at the project site, generally in the following order:

- Office work (As-built/Literature review)
- Site investigation (Planning Phase)
- Site investigation (Design Phase)

Office investigations consist largely of literature reviews and discussions with other staff. Site investigations consist of activities performed at the project site to acquire information such as surface mapping, or subsurface investigations using drilling, geophysics, or the Cone Penetration Test (CPT).

In some cases, more than one GP from more than one design branch may be performing investigations on the same project. If more than one GP is working on the project, communicate and coordinate the investigative work with all parties to maximize efficiency.

Investigation Process

The geotechnical investigation for a project typically begins with a request for a District or Structure Preliminary Geotechnical Report (DPGR or SPGR). The information presented in these planning-phase reports is typically based on the results of a literature review, conversations with knowledgeable staff such as other GPs or Maintenance personnel, and a limited site investigation. Reports prepared during the 0-phase and 1-phase, such as the Preliminary Geotechnical Design Report (PGDR), Geotechnical Design Report (GDR), Preliminary Foundation Report (PFR) and Foundation Report (FR), are based on the findings of the planning-phase work and the site investigation, which may include drilling, sampling, testing, geophysical testing, or the CPT.

Office Work

The GP should become familiar with the proposed project, and specific geotechnical work requested, by reviewing the request and plans, and meeting or speaking with the client to review the work request. Location and size of structures, embankments, and cuts should be understood. Discuss the amount of flexibility in the location of structures or other works, and determine the approximate loads.

Consult with other GPs who have worked in the area or perhaps even on that project. Contact the local Maintenance supervisor to inquire about their experience with the site and invite them to the site visit. They may have information that will be useful in planning the geotechnical investigation and/or that will be documented in the report, e.g. landslide or rockfall history.

The GP should perform a thorough literature search to become familiar with all pertinent information. Review the relevant design module(s) to identify the types of information needed and the reporting requirements. The following resources should be reviewed during the literature search and prior to the site investigation.

Publicly available resources:

- Geology and topography of the area and site:
  - California Geological Survey
  - US Geological Survey
  - Google Maps
  - Topozone
  - HistoricAerials
- Groundwater data are available from the:
  - California Department of Water Resources
  - Los Angeles County Wells
- Soil surveys are available from the Natural Resources Conservation Service.
- Liquefaction maps are available from the Association of Bay Area Governments.
- California fault maps (e.g., Alquist-Priolo Earthquake Fault Zone Maps) from the California Geological Survey.
- Environmental, soil, and water information is available at GeoTracker.

Caltrans Internal Information

- GeoDOG for geotechnical reports, lab tests, and Log of Test Borings (LOTB).
- Pathweb for a virtual drive-by.
- California Log of Bridges on State Highways
- BIRIS for bridge As-built plans (including LOTB) and maintenance records.
- Document Retrieval System (DRS) for District roadway and maintenance records.
- Caltrans ARS Online for seismic information.
- Aerial photographs are available from DHIPP or the Office of Photogrammetry.

The Caltrans Library at 1120 N Street Sacramento, as well as the California Geological Survey Library (801 K St. 14th Floor, Sacramento) have many reports and references, and can facilitate interlibrary loans.

Site Investigation for K Phase

Subsurface investigations utilizing drilling, CPT, or geophysical testing are typically not performed in the K phase, or for non-programmed work. However, if project needs justify these or other non-resourced work, the GP should propose the work to the

Project Engineer (PE) and Project Manager and communicate resource needs through DES Program Project Management to the District. An example would be a proposed bridge site with no available subsurface information but where liquefaction is suspected and would impact foundation and structure types.

The purpose of the planning-phase site investigation is to gather existing site information, evaluate if the proposed work is appropriate, and to support preliminary recommendations. The site investigation is typically limited to a site visit, observations, mapping, and hand measurements such as lengths, heights, and slope angles. If there are anticipated challenges relating to the design-phase site work, invite the appropriate support representatives along to plan future work. The Project Engineer and the local Maintenance supervisor should be invited.

Specific observations or measurements made will depend on the types of improvements proposed. Read the relevant design and reporting modules to identify the information needed. During the planning-phase site investigation:

- Field-check preliminary plans for accuracy, ideally with the Project Engineer.
- Assess the performance of existing structures and improvements (e.g., scour, slope stability, erosion, and rock fall) as they relate to the proposed work.
- Decide whether the proposed work is constructible as planned. Identify suitable foundation types, and needs for ground improvement or other items that could have significant impact on the scope of work.
- Walk the site with Maintenance. Ask them to identify any problems in the area and their availability to assist with future site work such as traffic control or access road construction.
- Select potential boring locations and related access.
- Determine geophysical testing needs.
- With the PE and Caltrans personnel involved in obtaining permits, determine the need for permits and when to start the permit process.
- Determine potential task order needs and discuss with the PE and Project Manager (PM).
- Photograph or video relevant site features and probable exploration locations.
- If drilling is to be performed during the planning phase, refer to the following sections on planning a subsurface investigation for the design phase.

Site Investigation for 0 or 1 Phase

The 0 or 1 phase geotechnical investigation begins with a request for a Preliminary Geotechnical Design Report (DPGR), Geotechnical Design Report (GDR), Preliminary Foundation Report (PFR), or Foundation Report (FR). The purpose of the 0 or 1-phase site investigation program is to obtain the engineering properties of the soil or rock and to determine the areal extent, depth, and thickness of each identifiable soil/rock unit that could affect the design of the project, while minimizing exploration costs.

The geotechnical investigation for design should adequately define the subsurface conditions for design purposes, and be consistent with the standards of practice identified in this manual. The extent of the exploration and testing, and the number of measurements of each critical design property in each unit/stratum must give a reasonable degree of confidence in the property measured. Specifics of information needed for design are presented in the applicable design module(s) of the Geotechnical Manual and their references.

The GP may or may not have been involved in the K-phase investigation for the project, so the amount of literature review necessary at this time will vary.

Preparing for Site Visit

To prepare for the site visit:

- Review work request for completeness. See the applicable design module, Bridge Memo to Designers (MTD) 1-35 for bridge foundations, or MTD 5-19 for earth retaining structures for the required content. The request must provide adequate information to allow for effective planning of the investigation.
- Speak with the requestor to establish a working relationship and to review the work request. Invite the client to the site visit.
- Review literature and all K- and 0-phase reports. Speak with the report authors, if available.
- Submit a *Site Assessment Questionnaire* to the District Environmental Coordinator. Hazardous subsurface materials could affect the investigation and therefore need to be identified as early as possible. The specifics of hazardous materials present at the site will determine whether or not GS staff can perform the drilling, CPT, or other investigative work, and determine special precautions needed. If GS cannot perform the site work then the GP must arrange for a Task Order to have a consultant perform the site work.
- Speak to and invite the local Maintenance supervisor to the site visit.
- If permits are anticipated invite the appropriate district personnel (Right of Way, Environmental, etc.)
- Contact District Surveys to obtain a benchmark close to the project.

Site Visit

The objective of the site visit is to prepare for a successful site investigation. In many cases multiple site visits may be necessary to prepare for the subsurface investigation work. The GP should:

- Meet with personnel who will support the site investigation, such as the project engineer, maintenance supervisor, and environmental coordinator.
- Review the site and plans with the project engineer. Confirm that the planned work is appropriate. Use of a handheld GPS to locate proposed improvements is helpful.
- Field-check plans, layouts, and cross sections for accuracy with the project engineer.

- Assess the performance of existing structures and improvements (scour, slope stability, erosion, rock fall, etc.) as it relates to the current work. Do not limit observations to just the footprint of the structure, wall, or other improvements being considered. Make note of relevant geologic features within a reasonable distance of the site. Ask the maintenance supervisor for their insight and experience with the site.
- Estimate the type and variability of subsurface conditions. What's expected will help determine the appropriate investigation tools to be used, the number and location of borings, and the sampling needs.
- Identify site investigation methods to be used (drilling, CPT, geophysics) and the preferred order (or staging) of the work. See *Selection of the Appropriate Investigation Method* and *Choosing Boring Locations and Depth* below.
- Look for conditions that will affect the location of borings such as:
    - Wetland or surface water
    - Environmentally sensitive areas
    - Underground or overhead utilities
    - Active rockfall areas
- Evaluate site access, the need for permits and timing of work. Talk with designer about potential for changes to alignments, support locations, wall lengths, etc, and consider these when applying for permits and determining access routes (see the Offices of Geotechnical Design – Quality Management Plan, *Permitting* section).
- Working with Maintenance and Drilling Services, check access for exploration equipment and determine the best suited equipment type for the site. If site preparation is necessary, determine the type of equipment, such as a bulldozer, that may be needed for drilling equipment access. Note exploration locations and access details on the Layout Plan Sheet and submit this with the *Drilling Request*. District Environmental will need to be involved in any plans to do grading work for site access.
- For projects adjacent to rivers or lakes inquire with the District Environmental if water can be pumped and used for drilling. This will eliminate idle time associated with water resupply.
- Determine the need for traffic control to accomplish the site exploration program (see *Choosing Boring Locations and Depth*).
- Locate the benchmark(s) to be used (see *Borehole Location Module*). Create a temporary benchmark closer to the planned exploration locations to make the borehole location task easier on the day of drilling, especially for night drilling.
- Identify geophysical testing needs.
- Mark for utility clearance (see the Offices of Geotechnical Design – Quality Management Plan, *Utility Clearance* section) and ask Maintenance to mark irrigation and electrical lines at all proposed boring locations.

Follow-up work to the site visit

This may include the following, depending on the nature of the planned site investigation work:

- Submit Drilling Request, CPT Request, and/or Geophysics Request per the Offices of Geotechnical Design – Quality Management Plan.
- Work with Maintenance to finalize site access and traffic control needs.
- Submit pertinent information for permitting to the District.
- Obtain utility clearance. (see the Offices of Geotechnical Design – Quality Management Plan, *Utility Clearance* section).

Selection of the Appropriate Investigation Method

The purpose of the investigation is to acquire the information needed to design the project. The GP should evaluate As-built information and utilize it for planning the investigation. This may reduce or eliminate site work. It is often possible to obtain information in several ways. The specific method used can vary depending on equipment availability, access, timing, site conditions, and the planned work. For example, a foundation investigation for a bridge using driven standard plan piles could use mud rotary drilling, augering, CPT, or the dynamic penetrometer. Sampling could range from none in CPT and penetrometer holes, to visual descriptions and on-site testing of samples using pocket penetrometer, torvane and vane shear, to detailed laboratory testing.

Consider the cost of the investigation including time and dollars, safety, and effects on traffic and maintenance operations. Rather than planning many conventional borings with Standard Penetration Test (SPT) borings, consider using many CPT holes in conjunction with a few conventional borings with SPT. If piles are to be founded on rock or dense soils, consider dynamic penetrometer holes or geophysical testing to delineate the bearing surface. For the cost of one SPT boring it is possible to push about 5 CPT soundings or 20 dynamic penetrometer holes. Weigh the effort versus the cost and desired outcome. Consider Wacker-driven 1-inch soil tubes instead of truck mounted drilling, if clearing new access roads are required for the latter. Instead of drilling in lane closures or at night, consider drilling on city streets, even if it requires an extra boring or two.

Caltrans has the ability to drill borings using wireline/conventional rotary drilling, hollow stem augers, dynamic penetrometer, hand-drilled soil tubes, and CPT. For more details, refer to the Drilling Services webpage or contact the appropriate Drilling Services liaison. In some situations hand or backhoe trenches, geophysical measurements, and surface mapping can replace drilling. Exposure of workers to traffic can be minimized by choosing the CPT and/or dynamic penetrometer (DP) in high traffic areas and correlating results with conventional drilling performed in safer areas. CPT is considered safer because the operator is protected by the truck, and dynamic penetrometer may be safer because holes are completed in minutes rather than hours.

When developing a subsurface investigation, consider the various drill methods and sampling needed to achieve the project goals. Augering is cheap for relatively shallow holes and provides instant groundwater information, but it is not suitable for SPT work below the water table or when borehole geophysical measurements are planned. Wireline (self-casing) drilling makes it easy for the drillers to maintain stability of the

holes and is ideal for borehole geophysics, but their use may mask hole instability. If CIDH piles are being contemplated for the design, then consider at least one hole not using a self-casing drilling method in order to determine if caving is a problem. All methods have limitations. CPT cannot go through very hard or dense layers, or retrieve samples. Augers cannot penetrate hard rock. The dynamic penetrometer provides only relative data. SPT holes require truck or trailer mounted equipment which may require construction of access roads. But each method has its use.

Testing should also be appropriate for the foundations and design methods being considered (refer to the relevant design module). In situ and site testing are often quicker and cheaper than lab testing. Where appropriate, use vane shear, pocket penetrometer, and torvane to determine cohesive shear strength. The pressure meter or dilatometer can determine rock strength at the site.

Consult with the Geophysics and Geology Branch to determine what they can do to reduce the effort (time, cost) and uncertainties. Geophysics cannot provide a description of the soil or rock; however, it can be used to evaluate variability of subsurface soil and rock, and can provide in situ material properties such as bulk density, shear modulus, and porosity. Shear wave velocity can be obtained directly from a down-hole instrument placed in a borehole at the end of drilling operations, or indirectly from surface wave measurements. Geophysics can be cost-effective for small projects where specific targets are needed, e.g. landslide imaging, borehole geophysics for in situ properties, void detection, subsurface utilities, and rock rippability. For large projects, it may be useful to perform geophysical investigations to determine the uniformity of the subsurface conditions. Based on those results, the number or location of boreholes may be adjusted.

CPT can provide shear wave velocity measurements more cheaply than PS Suspension logging. Consider using the seismic cone even if measurements cannot be made to a depth of 100 feet. Surface wave velocity measurements can also be used in lieu of drilling and CPT in areas with restricted access or rocky conditions.

If a required investigative method is not available in-house, the GP should work with the appropriate support branch to acquire consultant services.

Choosing Boring Locations and Depth

For many investigations the exact location of boreholes may not be critical. Choosing the location of boreholes is a function of the project and the geology. For example, if the project is a landslide investigation where the slide is occurring on a highly localized geologic feature, such as a thin clay layer, then it may be imperative that the boring be located exactly. If the project is a retaining wall on piles in an area of uniform geology, it may be sufficient to drill a hole up to several hundred feet from the wall layout line. Whenever possible, the borehole locations should be chosen so that the drilling does not require lane closures or expose personnel to traffic. Maintenance can remove barrier rails to allow access to potential boring locations. Drilling on city streets and frontage roads is safer than drilling on highways and freeways. Consider drilling two holes slightly off-site rather than one exactly on-site to lessen exposure to traffic.

Depth of drilling is a function of the project and the geology. If the project foundations are well established, e.g., it is certain that Class 90 Alternative "X" piles will be used, then it makes sense to drill 10 to 20 feet deeper than the estimated pile tip. On the other hand, if there is a reasonable possibility that the proposed pile group foundation may be replaced with single CIDH or CISS piles, then at least some of the exploration borings should be deep enough for design of the CISS or CIDH piles. Consideration of the geology is also important. Where the geology is well exposed and uniform, drilling may not be necessary. For example, even if V_{S30} is required for a seismic analysis and the site is homogeneous granite, it may not be necessary to drill a 100-foot deep hole to establish the shear wave velocity. The design modules provide general guidance for drilling depths. However, consideration of the geology and project parameters must take precedence.

Minimize encroachment on environmentally sensitive areas (ESA) to save the time and extra work needed to address the special needs of the area. Locations of ESAs are typically shown on the project layout plans. The PE, or the project's environmental staff, should be able to locate the ESAs during the site visit. The Categorical Exemption/Categorical Exclusion (CE) obtained from District Environmental prior to drilling will identify the ESAs and special requirements needed during the drilling process.

Drilling through a bridge deck may be more efficient than using a barge. Drilling through a bridge deck requires the approval of the Bridge Maintenance Engineer to assure that critical structural elements are not damaged by the coring operations.

Refer to AASHTO LRFD BDS Section 10.4.2, *Subsurface Exploration*, for more information about the number, locations, and depths of exploratory borings.

References

1. "*Subsurface Investigations – Geotechnical Site Characterization*", FHWA NHI-01-031, May 2002
2. Geotechnical Engineering Circular No. 5, Geotechnical Site Characterization, FHWA NHI-16-072, April 2017
3. "*Manual on Subsurface Investigations*", AASHTO 1988
4. AASHTO LRFD BDS Section 10.4.2, *Subsurface Exploration*, 8th Edition, 2017

Soil and Rock Logging

Soil and rock logging must be performed in accordance with the current version of the *Soil and Rock Logging, Classification, and Presentation Manual.*

Borehole Location

The Soil and Rock Logging, Classification and Presentation Manual requires that each borehole be located by Station and Offset from the reference line, and that the benchmark used to determine the borehole elevation be identified on the plan sheet. This section provides additional information and direction relating to these requirements.

The purpose of locating the borehole is to present its location relative to the planned work and to document the benchmark used to determine top of borehole elevation.

Benchmark

Prior to drilling, the geoprofessional should locate and verify a nearby benchmark, which may be an actual USGS or Caltrans monument or a fixed object of known elevation, such as a centerline begin bridge or end bridge. Usable benchmarks may be shown on the project plans, topographic maps, or can be obtained from Caltrans Surveys or the US Geological Survey.

Verify whether the benchmark is located close enough to the planned boreholes to allow convenient determination of the borehole elevation. If not, the geoprofessional should consider setting a temporary benchmark close to the planned work so to allow quick and easy determination of the borehole elevation before, during, or after drilling operations. Such a temporary benchmark could be a paint mark made during an initial site visit or while marking the site for USA.

Location Methods

Borings can be located using traditional tools, GPS devices, topographic mapping, and District surveys.

Traditional Tools

The accuracy associated with traditional tools used by the Geoprofessional (measuring tapes and wheels, laser range finder, level, etc.) are acceptable for the log of test borings sheet, boring record, and geotechnical reporting.

To determine the station and offset, use convenient features shown on the plans. Foundation plans and layout plans typically present the alignment line, centerlines, and fixed objects such as drainage inlets, curbs, trees, structures and other features which may be used as a reference to locate a borehole. The station and offset of the feature can be scaled directly from the plan sheet. Alternatively, horizontal locations of reference points

can be determined by using the measuring tools within the MicroStation program if desired.

In situations where it is not possible to use an existing feature as a reference (e.g. a new alignment) contact the District Project Engineer or Surveys to inquire about existing benchmarks, or to request the local Caltrans surveyors set benchmarks, stake the centerline, or stake the begin bridge (BB) and end bridge (EB) locations.

Using the existing feature for horizontal control and benchmarks for vertical control the Geoprofessional determines the location and elevation of the boreholes. It is expected that the foundation drillers will assist with this process, as requested by the Geoprofessional, during or shortly after drilling.

If more than one leveling equipment setup is used to locate one or more boreholes, it is recommended that the loop be closed to assure accurate measurements were obtained.

GPS

The accuracy of GPS tools varies widely. Use tools that can achieve accuracy that meets the reporting requirements. In general most GPS devices cannot achieve the needed accuracy for elevation.

Topographic Mapping

Topographic mapping may be used to determine borehole elevations if the map accuracy meets the reporting requirement.

District Surveys

Using District Surveys to determine borehole locations or elevations is recommended for those occasions when it is not possible, safe, or cost effective for the geoprofessional to do so in the field. Typical examples are:

- New alignment where prior staking is not available
- Site is located in an area of high relief where the location cannot be determined by physical means (tape measure, measuring wheel) or where the benchmark and borehole elevations are significantly different.
- Site is otherwise not conducive to direct measurement (e.g. in a wooded area)
- Site is heavily vegetated and the elevations cannot be determined without establishing a number of temporary benchmarks and turning points.
- Site is along a corridor in a metropolitan area with numerous boreholes.

Reporting

Report station and offset to the nearest foot. For structures report elevation to the nearest 0.1 foot. For earthwork report elevation to the nearest foot unless special needs (e.g. slope inclinometer for landslide monitoring) or geology concerns (e.g. potential for encountering hard rock) require reporting the elevation to the nearest 0.1 foot.

The datum used for borehole location should be the same datum used by the Project Engineer.

Soil Correlations

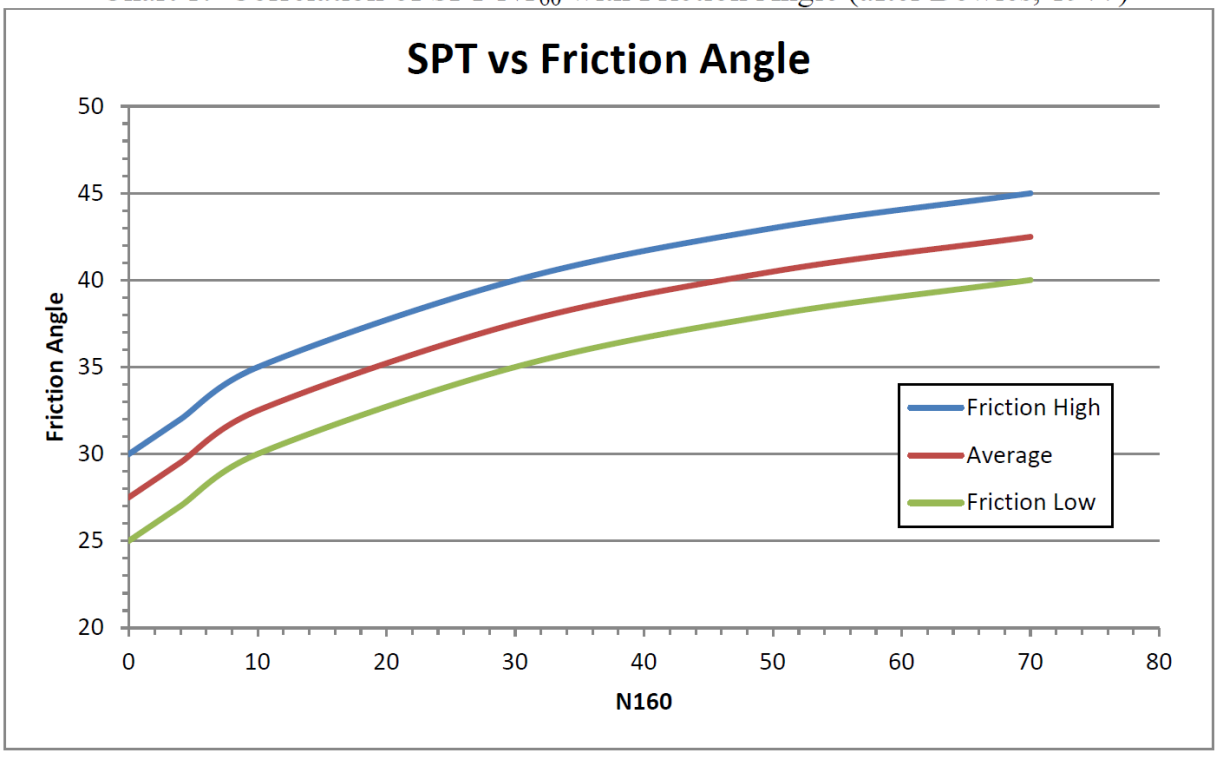
This section of the Geotechnical Manual presents the SPT correlations to be used for friction angle (phi angle) and unit weight. The correlations use Standard Penetration Test (N) values corrected for overburden and hammer efficiency ($N1_{60}$). Usage of correlations for geotechnical design is addressed in the various design sections of the Geotechnical Manual. Other correlations, e.g. CPT correlations and shear wave velocity correlations are found elsewhere in the Geotechnical Manual.

Cohesionless Soil: Friction Angle

Correlations of SPT blow counts to cohesionless soil friction angle and unit weight follow Bowles (1977) and are consistent with many of the NHI manuals used by the department. The correlations use Standard Penetration Test (N) values corrected for overburden and hammer efficiency ($N1_{60}$).

Use Chart 1 to correlate $N1_{60}$ to the friction (phi) angle.

Chart 1: Correlation of SPT $N1_{60}$ with Friction Angle (after Bowles, 1977)



Choose the friction angle (expressed to the nearest degree) based upon the soil type, particle size(s), and rounding or angularity. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant (about 30+ %) silt-sized material will fall in the lower portion of the range. Coarser materials

with less than 5% fines will fall in the upper portion of the range. The extreme range of phi angles for any $N1_{60}$ is five degrees, so the adjustment factors for particle size and roundness should be only a degree or two. The following bullets provide help in determining which value to select for a given $N1_{60}$ and soil type:

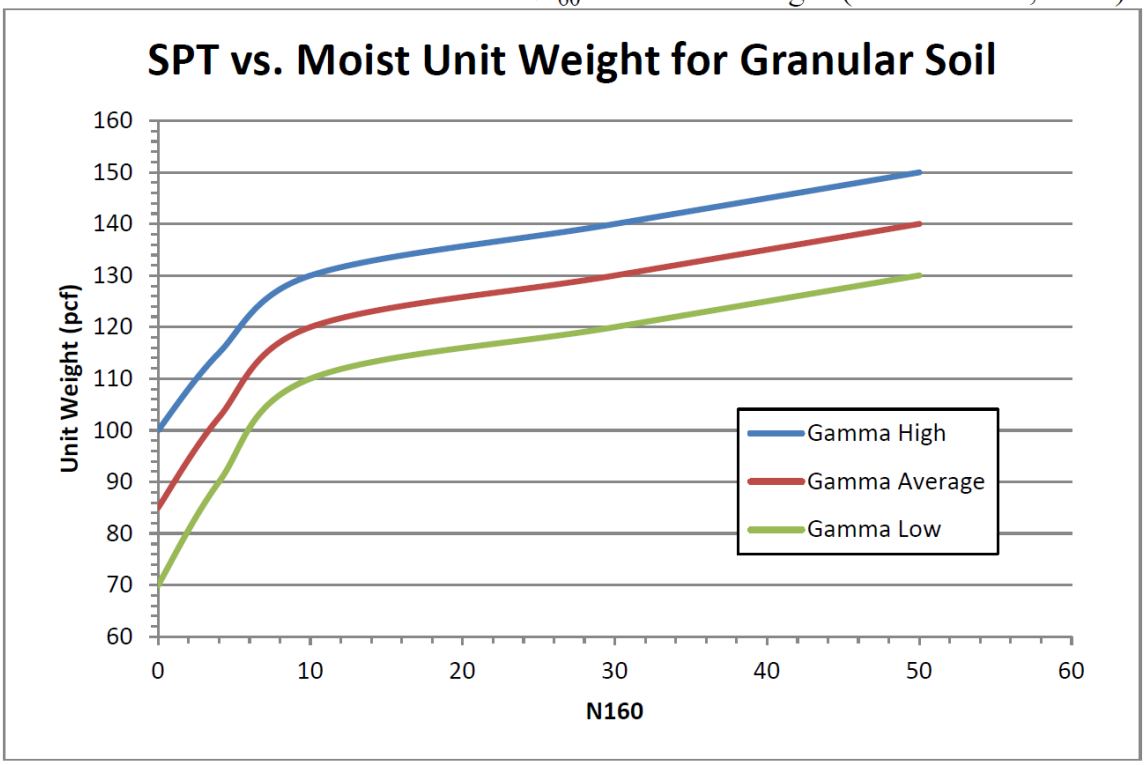
- Use the maximum value for GW
- Use the average for GM and SP
- Use the minimum for SC
- Use the minimum + 0.5 for ML
- Use the average +1 for SW
- Use the average -1 for GC
- Use the Maximum -1 for GP

Values may also be increased with increasing grain size and/or particle angularity and decreased with decreasing grain size and/or increasing roundness. For example, an SP with $N1_{60}$ = 30 could be assigned phi angles of 37, 38 or 39 degrees for fine, medium and coarse grain sizes respectively.

Cohesionless Soil: Unit Weight

Use Chart 2 to correlate $N1_{60}$ to the moist unit weight for cohesionless (Granular) soil.

Chart 2: Correlation of SPT $N1_{60}$ with Unit Weight (after Bowles, 1977).



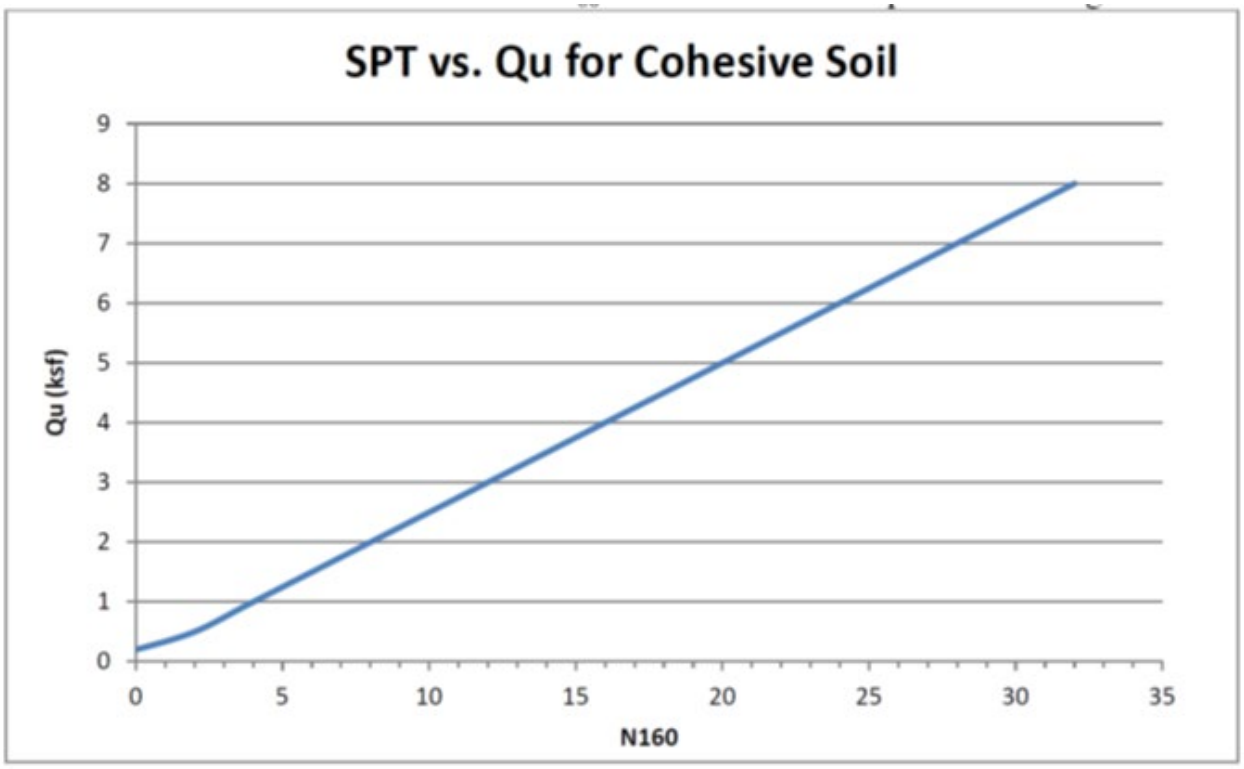
Choose the unit weight expressed to the nearest five pcf for the soil type based on the following guidelines:

- Use the higher values for well-graded sands and gravels and average values for poorly-graded sands and gravels.
- Use lower values for elastic silt, and clayey or silty sands and gravel.
- Deduct up to 20% for dry soils.

Cohesive Soil: Unconfined Compressive Strength (Qu)
Undrained Shear Strength (Su)

The standard practice is to determine shear strength of cohesive soils in the field based on measurements with torvane, pocket penetrometer, or vane shear. It is not acceptable to use SPT correlations to determine shear strength or to assign consistency values. For preliminary studies, use Chart 3 to assign shear strength values when only SPT values are available. Usually this is applicable when data are available from old as-built LOTBs where field or laboratory strength tests are not available.

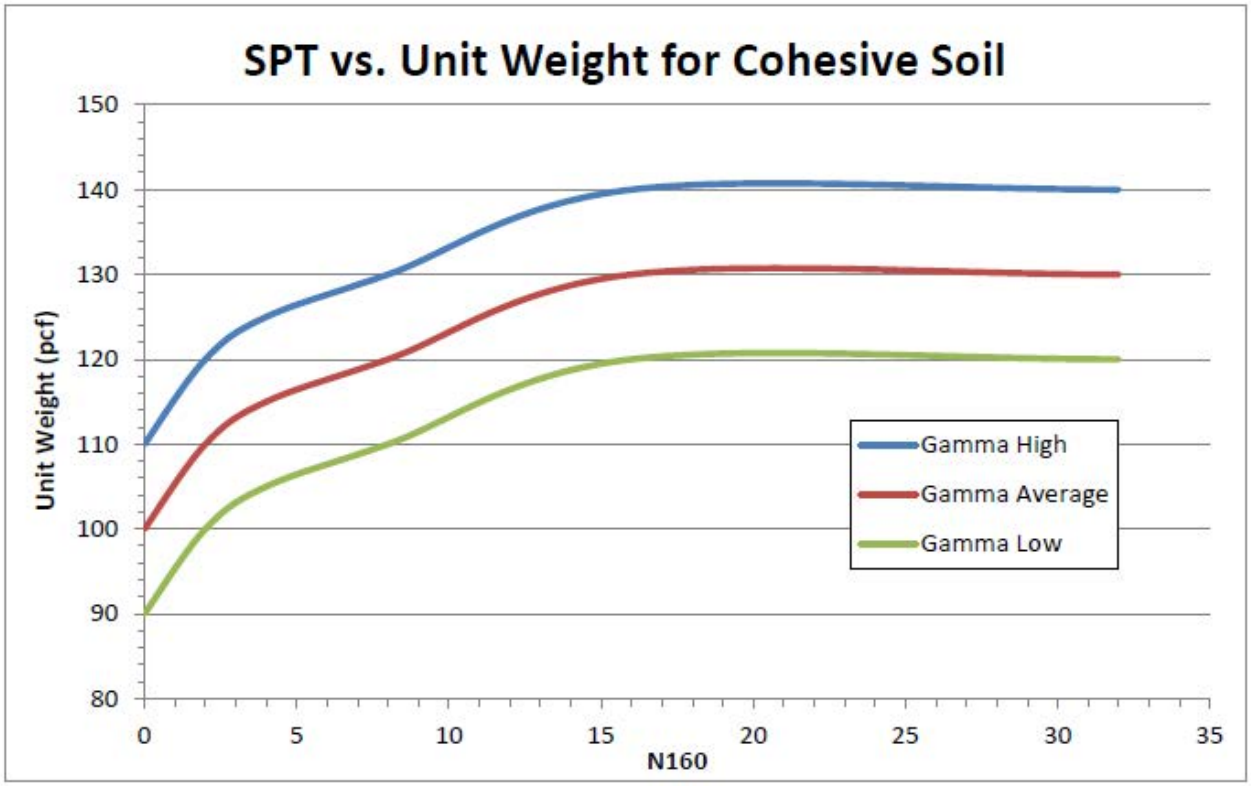
Chart 3: Correlation of SPT $N1_{60}$ to Unconfined Compressive Strength (after Bowles, 1977)



Cohesive Soil: Unit Weight

Use Chart 4 to correlate $N1_{60}$ with the Unit Weight of cohesive soil.

Chart 4: Correlation of SPT $N1_{60}$ with Unit Weight (after Bowles, 1977).



Comparing field pocket penetrometer and/or torvane readings to Chart 4 is a good way of determining whether high or low values should be used. For example, if the pocket penetrometer reading for a clay with $N1_{60}$ = 10 is about 2.5 ksf (the same as the value shown in Chart 3) the unit weight should correspond to the average value. If the pocket penetrometer reading is higher, the unit weight should be increased from the average, and if the pocket penetrometer reading is lower, the unit weight should be decreased from the average.

In the absence of SPT data, unit weights can be estimated using Charts 3 and 4 and the strength data (e.g., pocket penetrometer reading). For example, from Chart 3, a pocket penetrometer value of 5 ksf corresponds to an SPT $N1_{60}$ value of 20. Chart 4 shows the average unit weight of a cohesive soil with SPT $N1_{60}$ = 20 is 130 pcf.

Sampler Size Conversions to SPT N-value

When sampler sizes vary from that of a SPT sampler (per ASTM 1586), conversions are needed to modify the field SPT N-values. For conversions to SPT sampler from other sampler sizes where weight (140 lb.) and height (18 inches) do not vary from ASTM 1586, the following table provides conversion values for common alternate sampler sizes.

Table 1: Conversion Values for Sampler Sizes

Sampler Type	Outside Diameter (inches)	Inside Diameter (inches)	Conversion to SPT N-value	
			Clay	Sand
1.41-inch	2.0	1.375	1.00	1.00
2-inch	2.5	1.95	0.85	0.63
2.4-inch	3.0	2.4	0.65	0.41
2.5-inch	3.875	2.5	0.24	0.13

1-inch Diameter Driven Sampler

Some As-built LOTBs include 1” diameter sampler data and use soil descriptors that differ from current practice. The following table lists these descriptors, 1" soil tube blow counts, and associated SPT blow counts. The 1” soil tube blow counts are obtained by driving a 1” sampler one foot with a 25-pound hammer dropped one foot.

Table 2: Density/Consistency descriptor conversions for 1" Soil Tube (Closed Tip)
(Ref: Handbook of Engineering Geology, State of CA Division of Highways, 1958)

Density/Consistency		1" Soil Tube -- Blows Per Foot			Standard Penetration Blows Per Foot
Granular	Cohesive	Sand and Gravel	Silt	Clay	
Very Loose	Very Soft	0-50	0-50	0-60	0-5
Loose	Soft	50-100	50-180	60-250	5-10
Slightly Compact	Stiff	100-350	180-1000	250-1000	10-20
Compact	Very Stiff	350-525	1000-2000	1000-4000	20-35
Dense	Hard	525-1500	2000-5000	4000-5000	35-70
Very Dense	Very Hard	1500+	5000+	5000+	70+

References

- Bowles, J. E., 1977, *Foundation Analysis and Design*, McGraw-Hill, Inc., New York

Geotechnical Design using Standard Plan and Bridge Standard Detail Sheets

The *Standard Plans* and *Bridge Standard Detail Sheets* contain design and construction details for commonly used structures. These pre-engineered designs use assumed design parameters and site conditions and eliminate the need for special designs at many sites. Examples of standardized designs commonly used by GS include sound walls, earth retaining systems, overhead sign foundations, standard class piles, mechanically stabilized embankments and culverts.

The intent of standardized designs is to minimize engineering time involved with repetitive designs while standardizing field explorations, material fabrication, and construction efforts. In many cases, projects could be designed for more efficient use of materials, but this would be offset by the additional costs for exploration, engineering, custom fabrication of forms and materials, construction, increased quality control and inspection.

Standard designs that involve soil-structure interaction use assumed soil strength parameters and site geometry. For example, standard sound wall designs use three assumed soil strengths (ϕ = 25°, 30° or 35°) and two assumed ground conditions (flat or sloping). If the site parameters meet or exceed those of the standard plan then the standard design is acceptable. If the site conditions do not meet or exceed these minimum soil strength design values or ground conditions, then the standardized design may not be used and a special design will likely be required. Likewise, the sound wall site investigation procedures in this manual would not apply as those are tailored to standard site conditions.

In many cases it may be obvious that the minimum design requirements are exceeded, and detailed investigations and calculations are not necessary. For example, where the proposed location for a Standard Plan Type 1 retaining wall places the footing on rock or intermediate geo-materials, the footing bearing stress will usually not exceed the factored bearing resistance regardless of the extent of fracturing, weathering, hardness, or strength. In this case the site investigation may be focused more on constructability issues, such as ease of rock excavation, than geotechnical design.

Modified Standard Plan Design

When the required minimum design and geometric conditions are not met for a standard design, it may be possible to adapt an alternative standard design by selecting a more conservative option. For example, if the factored gross nominal bearing resistance at a proposed wall site is not sufficient to support a proposed 10-foot Type 1 retaining wall on a spread footing, then the geoprofessional should evaluate a 12-foot Type 1 retaining wall constructed to a height of 10 feet. The taller wall will have a larger footing width, which will produce a larger factored gross nominal bearing resistance and a reduced footing gross uniform bearing stress. In most cases, recommending construction of a larger wall footing will be more economical than constructing the wall on deep foundations. Sometimes it is feasible to modify the site conditions to meet the

standard design requirements, such as slope flattening or ground improvement to increase bearing resistance, and the feasibility of those options should be evaluated prior to selecting a special design.

Usage

It is expected that the geoprofessional make every effort to recommend the standard or modified standard plan designs that are available in the Standard Plans or Bridge Standard Detail Sheets in their projects. Special designs should be used only when all standard designs or modified standard plan designs are deemed infeasible.

The procedures of the following standard plan sections are to be used when investigating, designing and recommending standard plan structures. If site conditions do not allow construction of a standard plan or modified standard plan structure, then the procedures of that section no longer apply and the geoprofessional should modify the site investigation as appropriate to support the planned special design.

Sound Walls

Sound wall foundation design consists of choosing standard designs from the current Standard Plans (2018 Standard Plan B15-1 to B15-15). The following foundation types may support standard sound walls:

- Pile Cap
- Trench Footing
- Spread Footing

Sound wall foundation types and sizes shown on the *Standard Plan Sheets* were designed using assumed soil strength parameters. The objective of a sound wall foundation investigation is to determine if the soil strength at the proposed wall location meets or exceeds the assumed soil strength used to design the sound wall.

Sound walls are not designed to retain soil. In cases where soil retention is necessary the sound wall may be constructed on a Type 736S/SV Barrier, which allows retention of up to four feet of soil, or on a retaining wall. Standard Type 1, 5 and 7 retaining walls have been redesigned to accommodate sound walls and are available as Bridge Standard Details Sheets.

Typically, the most cost-effective sound wall foundation is cast-in-drilled-hole (CIDH) piles. This is different from most structures where the least expensive foundation is usually spread footings. Trench footings may be considered where groundwater would require wet pile installation but the trench footing is dry. Maximum trench footing depth is less than 12 feet.

Investigations

Exploration for standard plan sound wall foundation design seeks to identify and describe the subsurface material, determine its strength, locate the water table (if within the depth of the anticipated sound wall foundations), and to identify relevant conditions that might affect the foundation construction, such as caving conditions, the presence of cobbles and boulders, or shallow rock.

Borings should be spaced no closer than 500 feet apart unless warranted by anticipated variations in soil strength parameters, constructability conditions and/or foundation types.

Auger borings are preferred since they mimic the typical construction method for sound wall CIDH pile foundations. They provide information on hole stability, water table, and presence of cobbles, or shallow rock, which might render CIDH piles infeasible. Use the CPT to supplement auger borings where feasible.

Corrosion testing is not required for standard plan sound walls.

It is not necessary to conduct a drilling program for sound walls that will be founded entirely in fill that has yet to be placed, such as elevated roadway embankments or sliver widenings, as fills are constructed to a minimum of 90% relative compaction.

CIDH Pile and Trench Footing (Cohesionless Soil)

As sound wall foundation design is based on constructability and three standard soil strength cases, laboratory testing should not be performed for the purposes of determining soil strength or *Soil Classification* in cohesionless soil. The SPT is sufficient for correlating to soil strength. Presenting the *Soil Identification* as determined by the visual/manual method provides sufficient information for designing and constructing sound wall foundations.

CIDH Pile and Trench Footing (Cohesive Soil)

In non-saturated cohesive soil, sample and perform laboratory testing to determine the drained shear strength.

Sound wall design in saturated cohesive soil require a special design. Lengthen borings to 50 feet, sample and perform field and laboratory testing to determine the undrained shear strength.

Design Procedures: Sound Walls on CIDH Piles and Trench Footings

The design of sound walls supported on CIDH piles and trench footing foundations in cohesionless or non-saturated cohesive soil are based on two parameters:

1. Ground Line

    Case 1: Level ground (± 10%) on both sides of the wall
    Case 2: Level ground (± 10%) on the traffic side of the wall, sloping ground no steeper than 2:1 on the opposite side

2. Soil Friction Angle (ϕ)

    Phi = 25°
    Phi = 30°
    Phi = 35°

For cohesionless soil, determine the soil friction angle (ϕ) in accordance with field SPT measurements and the *Soil Correlations* module. Then select the appropriate pile spacing and length from the Standard Plan given the wall height. For non-saturated cohesive soil, determine the drained shear strength via laboratory testing. Then select the appropriate pile spacing and length from the Standard Plan given the wall height.

For saturated cohesive soil, a special design is required. Work with the Structure Designer to determine what data needs to be provided by GS in order for Structure Design to complete the design.

Engineered fills compacted to 95% relative compaction (e.g., approach fills within 150 feet of a bridge abutments) should be assumed to satisfy the requirements for phi = 35°. Engineered fills compacted to 90% relative compaction (most fills other than abutment approach fills) should be assumed to satisfy the requirements for phi = 30°.

If the field investigation reveals that the CIDH piles would extend below the ground water surface, then CIDH piles should be abandoned as the preferred foundation type in favor of a trench or spread footing. If neither the trench nor spread footing alternatives are feasible, then CIDH piles should be recommended provided they are 24-inches in diameter to satisfy the requirement for non-destructive testing (gamma-gamma logging) of the piles. Recommending 24-inch CIDH piles for support of a sound wall will require a special design by Structures Design, so this alternative should be used only after the three standard foundation types are proven infeasible. The only standard plan sound wall design requiring 24-inch CIDH piles for support is for a wall spanning over buried utilities as shown on Standard plan sheet B15-15.

Design Procedures: Sound Walls on Spread Footings

The design of standard plan sound walls supported on spread footing foundations is based on Working Stress Design and Allowable Soil Bearing pressures. The Standard Plans show two ground line cases as follows:

Case 1: Level ground ($\pm$ 10%) on both sides of the wall
Case 2: Level ground ($\pm$ 10%) on the traffic side of the wall, sloping ground no steeper than 2:1 on the opposite side

Table 3 presents the footing width (B), the effective footing width (B'), the equivalent footing contact pressures ($q_{contact}$) and the required ultimate soil bearing capacity (q_{ult}) for the various sound wall heights. When recommending a sound wall to be supported on a spread footing, verify that the soil conditions at the wall location meet or exceed the required Ultimate Soil Bearing Capacity, q_{ult}.

Table 1: Spread Footings

Wall Height (H) (feet)	Footing Width (B) (feet)	Effective Footing Width (B') (feet)	Equivalent Uniform Footing Contact Pressure, ($q_{contact}$) (tsf)	Required Ultimate Soil Bearing Capacity, (q_{ult}) (tsf)
6	3	1.1	0.5	1.5
8	4	1.7	0.3	0.9
10	5	2.7	0.2	0.6
12	5.75	2.6	0.2	0.6
14	6.5	2.9	0.2	0.6
16	7.5	2.5	0.2	0.6

Design Procedures: Sound Walls on Retaining Walls on Spread Footings

For sound walls supported on retaining walls supported on spread footings, refer to the appropriate Bridge Standard Details "XS" sheets for the required ultimate soil bearing pressure, q_{ult}.

Helpful hints

- Standard sound wall pile is 16 inches in diameter. Maximum particle size for easy construction of a 16" CIDH pile is about 5". Soil conditions with cobbles larger than 5" may not be suitable for CIDH pile foundations.
- The maximum required ultimate soil bearing capacity of a standard plan sound wall spread footing is about 1.5 tsf. This is typically satisfied by an average $(N_1)_{60}$ value of 5 or larger within the depth B below the bottom of the proposed footing.
- If liquefaction is probable, the Geoprofessional should contact the client and discuss whether or not to include recommendations for mitigation in the report.
- Standard trench width for sound wall foundations is 12" (Case 1) or 15" (Case 2). Maximum particle size for construction of trench footings is about 6".
- Trench footings are not suitable foundation types in caving soils or below the water table.

Related Standards:

- Standard Plan Numbers B15-1 through B15-15
- Bridge Standard Details Sheets XS14-210-1 through XS15-130-2.

Reporting

Standard plan sound wall foundation recommendations must be conveyed to the client using the Geotechnical Design Report (GDR) format. The report should be brief and include a table presenting the following information:

- The beginning and ending stationing
- The wall height, H
- Recommended foundation type (e.g., CIDH piles, trench footing, or spread footing)
- Ground line Case condition, (Case 1 or Case 2)
- Soil strength, Phi value to be used for design, $\phi = 25°$, $\phi = 30°$ or $\phi = 35°$

A Log of Test Borings (LOTB) must be included with the design recommendations. A note should be added to the report directing the designer to include the LOTB in the contract plans, along with a list of LOTB sheet titles.

Table 2 presents example sound wall foundation recommendations. The following conditions exist at the site, which caused the designer to deviate from the preferred CIDH foundation type.

- From Station 16+27 SW LOL to Sta 20+37 SW LOL, shallow ground water exists.
- From Station 20+37 SW LOL to Sta 22+ 89 SW LOL shallow ground water and caving conditions exist.

Table 2: Sound Wall Foundation Recommendations

Wall Location	Wall Height (H)	Foundation Type	Ground line	Friction Angle (ϕ)
Sta 08+59 SW LOL to Sta 11+ 47 SW LOL	H = 12	CIDH Piles	Case 1	35°
Sta 11+47 SW LOL to Sta 12+89 SW LOL	H = 10	CIDH Piles	Case 2	35°
Sta 15+17 SW LOL to Sta 16+ 27 SW LOL	H = 10	CIDH Piles	Case 2	35°
Sta 16+27 SW LOL to Sta 20+37 SW LOL	H = 10	Trench Footing	Case 1	30°
Sta 20+37 SW LOL to Sta 22+ 89 SW LOL	H = 12	Spread Footing	Case 1	N/A

Standard Plan Overhead and Changeable Message Signs

This module presents the Department's standard of practice for the investigation, design, and reporting of foundation recommendations for standard plan Overhead (OH) and Changeable Message Signs (CMS), hereafter referred to as sign(s).

Sign foundation design consists of choosing standard designs from the current *Standard Plans and/or Revised Standard Plans* (RSP). Sign types, related foundation types, and pertinent Standard Plan Sheets are presented in Table 1.

Table 1: Standard Plan OH and CMS and Associated Foundation Types

Sign Type	Foundation Type	2018 Standard Plan
OH Signs-Truss, Single Post	CIDH, Spread Footing	RSP S1, RSP S2, S8
OH Signs-Truss, Two Post	CIDH, Spread Footing	S9, S15
OH Signs-Tubular	CIDH	S30, S36, S37
OH Signs-Lightweight	CIDH, Spread Footing	S49
CMS Model 500	CIDH	S101, S105, S109, S116

Sign foundation types and sizes shown on the Standard Plans were designed using assumed soil strength parameters. The objective of a standard plan sign foundation investigation is to determine if the soil strength at the proposed sign location meets or exceeds the assumed soil strength used to design the sign, and to evaluate constructability.

Sites that do not meet the minimum design requirements will require a special foundation design. Notify the design engineer when the minimum requirements are not met to determine whether the sign can be relocated or if a special design sign is required. Special designs for sign foundations must follow the applicable foundation design module.

Investigations

Exploration for a sign foundation design seeks to identify and describe the subsurface material, determine its strength or verify that the strength exceeds the design strength and unit weight, locate the water table (if within the depth of the anticipated sign foundation), and to identify conditions that might affect the foundation construction, such as caving, the presence of cobbles and boulders, or shallow rock.

Borings for standard overhead signs should extend no deeper than 45 feet below the finished grade at the overhead sign. The deepest foundation required for a standard plan overhead sign is 39 feet and there is little reason to consider drilling deeper.

Auger borings are preferred since they mimic the typical construction method for sign CIDH pile foundations. They provide information on hole stability, water table, and

presence of cobbles and boulders or shallow rock, which might render CIDH piles difficult to construct. Borings into rock are performed for constructability evaluation only since the minimum strength requirements are met.

Sampling and testing should be limited to Standard Penetration Testing (SPT) in granular soil or undisturbed sampling with field pocket penetrometer or torvane testing in cohesive soils. Soil corrosion testing should be performed as specified in Section 7.1 of the *Corrosion Guidelines*.

As sign foundation design is based on constructability and standard soil strength cases, laboratory testing should not be performed for the purposes of determining soil strength or Soil Classification. SPT, pocket penetrometer and torvane measurements are sufficient for correlating to soil strength. Use the *Soil Correlations* module to determine soil strength and unit weight. Presenting the Soil Identification as determined by the visual/manual method provides sufficient information for designing and constructing sign foundations. Consider performing laboratory strength tests only when the field test results are near to but less than the minimum strength requirements.

It is not necessary to conduct a drilling program for signs that will be founded entirely in fill that has yet to be placed, such as elevated roadway embankments or sliver widenings, as fills constructed to a minimum of 90% relative compaction exceed the minimum required soil strength.

Design Procedures

CIDH pile foundations require a minimum soil angle of internal friction of 30 degrees and a unit weight of 120 pcf (or submerged unit weight of 58 pcf), or a shear strength of 1.5 ksf for cohesive soils. Engineered fills compacted to 95% relative compaction (e.g., approach fills within 150 ft of a bridge abutment) should be assumed to satisfy the requirements for phi = 35°. Engineered fills compacted to 90% relative compaction (most fills other than abutment approach fills) should be assumed to satisfy the requirements for phi = 30°.

It is generally not economical to try to shorten piles. Do not perform lateral or axial resistance analyses if the minimum soil strength is available.

The Standard Plans currently show a spread footing option for OH Signs-Truss, Single Post and Two Post, and OH Signs-Lightweight. In the rare case where a spread footing design is considered, contact the standard plan owner to obtain the loads and eccentricities, then calculate the bearing capacity, and check it against the design criteria to verify the geotechnical design.

The maximum required ultimate soil bearing capacity of a standard plan overhead sign spread footing is about 7.5 ksf. This is typically satisfied by an average $(N_1)_{60}$ value of 10 or larger within the depth B below the bottom of the proposed footing.

Seismic investigations are not necessary except to evaluate liquefaction potential. If liquefaction is probable, contact the client and discuss whether or not to include recommendations for mitigation in the report.

Signs are sometimes added to a project by the District late in the PS&E process (after field investigations are completed and/or geotechnical reports have been completed). If soil data are not available, check with the project engineer to determine if a risk-based design, using the most conservative (longest) pile lengths, is appropriate. Such design should only be done if the groundwater elevation is known.

Truss Single Post Types II through IX

Single post truss signs may be supported by either spread footings or CIDH piles. Spread footings use the footing dimensions as specified in the revised Standard Plan RSP S2. CIDH pile foundations use Standard Plan S8 for pile dimensions.

Truss Two Post Types I-S through VII-S

Two post truss signs may be supported by either spread footings or CIDH piles. Spread footings use the footing dimensions as specified in the Standard Plan S9. CIDH piles use Standard Plan S15 for pile lengths.

Overhead Signs - Tubular

Single post and two post tubular signs are supported by CIDH piles. See Standard Plan S36 and S37 for pile dimensions.

Overhead Signs - Lightweight Extinguishable Message Sign and Flashing Beacons

Lightweight Extinguishable Message Sign and Flashing Beacons may be supported by either spread footings or CIDH piles. Spread footings use the footing dimensions as specified in Standard Plan S49. CIDH pile foundations use Standard Plan S49 for pile dimensions.

Overhead Signs-Truss, Single Post, Unbalanced/Balanced/Full Cantilever Butterfly CMS Model 500

Model 500 CMS are supported by CIDH piles. See Standard Plan S116 for pile details.

Reporting

Sign foundation recommendations are conveyed to the client in a Geotechnical Design Report (GDR). Reporting for a sign should be brief and include only the following information:

- Location of the overhead sign
- Sign type
- Summary of investigation performed
- Soil description, strength, and unit weight
- Corrosion test results
- Discussion of liquefaction potential
- Recommended foundation type(s) and dimensions;
    - Pile length and diameter for CIDH piles including the adjustment for sloping ground if applicable.
    - Spread footing dimensions and bottom of footing elevation or depth of burial.
- Depth to groundwater and recommendation for wet CIDH methods including gamma-gamma inspection pipes if the pile excavation extends below the groundwater level.

A Log of Test Borings (LOTB) or Boring Record must be included with the design recommendations if a boring was drilled or if an as-built boring was used. A note should be added to the report directing the designer to include the LOTB in the contract plans, along with a list of LOTB sheet titles.

Design Acceleration Response Spectrum

This module presents procedures for developing the design Acceleration Response Spectrum (ARS) for Caltrans bridges in accordance with the requirements of the Seismic Design Criteria (Caltrans, 2019), Version 2.0, with October 2019 Interim Revisions (SDC v2.0).

This module does not address the development of the design ARS for project sites that require a site-specific dynamic ground response analysis (DGRA) per Appendix B of the SDC v2.0.

Unless specified otherwise in a Project-Specific Seismic Design Criteria, Caltrans current practice is to use the Safety Evaluation Earthquake (SEE) design ARS developed per SDC v2.0 to characterize design ground motions for earth retaining structures, embankments, slopes, sign structures and other appurtenant highway facilities.

Design Ground Motion

Per SDC v2.0, the design ground motion at a bridge site due to earthquakes is characterized by the Design Spectrum in the form of the design ARS for 5% damping.

Unless otherwise requested by Structure Design, Geotechnical Services provides the SEE design ARS that characterizes the site horizontal ground motion at or near the ground surface with a 5% probability of exceedance in 50 years (i.e., 975 years return period). The SEE design ARS is evaluated based on the United States Geological Survey's (USGS) 2014 National Seismic Hazard Map (2014 NSHM) for 975 years return period (USGS, 2014, Peterson et al., 2014 and 2015). Modifications due to the basin-effects and/or near-fault effects are applied where necessary, per Appendix B of SDC v2.0.

In some cases, Structure Design may request the Functional Evaluation Earthquake (FEE) design ARS that characterizes the site horizontal ground motion with a 20% probability of exceedance in 50 years (i.e., 225 years return period). Except the return period, all other development requirements specified in Appendix B of the SDC v2.0 for the SEE design ARS are also applicable to the development of the FEE design ARS.

For bridge sites with low shear wave velocity, as specified in Appendix B of the SDC v2.0, a site-specific DGRA (i.e., numerical analysis of vertically propagating seismic shear waves from bedrock through the overlying subsurface soils to at or near the ground surface) is required to develop the final design ARS.

Unified Hazard Tool and ARS Online V3.0 Tool

The 2014 NSHM is the basis for developing the design ARS for Caltrans bridge sites. The USGS' Unified Hazard Tool (UHT) is used to obtain the 2014 NSHM ARS, and other required design ground motion and related seismic source parameters. Attachment 1

presents additional information regarding the 2014 NSHM and the UHT. It also briefly discusses the orientation-independent, rotated spectral acceleration definition used in the 2014 NSHM ARS, and the directionality of the component of the design ground motion acting on the horizontal plane as represented by the design ARS developed per SDC v2.0

Caltrans has developed the ARS Online v3.0 web tool (Caltrans, 2020) to conveniently determine the SEE design ARS and other related design ground motion parameters. For FEE design ARS, a procedure that utilizes both the UHT and ARS Online is discussed later in this document. In the future, the ARS Online will be updated to include FEE design ARS.

Design Ground Motion Parameters

The following information related to design ground motion is required in the foundation reports.

- Site coordinates (latitude and longitude in decimal degrees)
- Time-averaged shear wave velocity VS30 in feet /sec for the upper 100 feet (30 meters) of the design subsurface soil/rock profile(s)
- ARS
- Horizontal Peak Ground Acceleration (HPGA or PGA), in units of g, where g is the acceleration of gravity.
- Deaggregated mean earthquake moment magnitude (M or Mw) for the design PGA, and the
- Deaggregated mean site-to-source distance (R or Rrup, in km) for the 1.0 second period design spectral acceleration.

Site Coordinates

Site coordinates can be determined using a variety of tools, including Caltrans' Postmile Services, USGS' web-based Interactive Fault Map, and Google Earth.

Time-Averaged Shear Wave Velocity

The 2014 NSHM provides ARS for a site location based on time-averaged shear wave velocity (V_{S30}) for the upper 30 meters (100 feet) of the subsurface soil/rock profile.

To determine V_{S30}, the upper 100 feet (30 meters) of the subsurface soil/rock profile is divided into N number of layers based on the average layer shear wave velocity V_{Si} (i=1 to N). The equation used for calculating V_{S30} in feet/sec is:

$$V_{S30}\ \left(\frac{\text{feet}}{\text{sec}}\right) = \frac{100\ \text{feet}}{\left(\frac{D_1}{V_{S1}}\right) + \left(\frac{D_2}{V_{S2}}\right) + \cdots + \left(\frac{D_i}{V_{Si}}\right) + \ldots.. + \left(\frac{D_N}{V_{SN}}\right)}$$

Where D_i and V_{Si} are the thickness in feet and average shear wave velocity in feet/sec, respectively, for the i^{th} layer (i=1 to N), and $\sum_{i=1}^{N} D_i = 100\ feet$. Attention must be paid to the unit used for the soil/rock shear wave velocity since both feet/sec and meters/sec are widely used. For example, V_{S30} evaluated based on the above equation is in feet/sec, which must be converted to meters/sec unit for use with the ARS Online v3.0 or the UHT.

Design V_{S30} Profile(s)

For many bridge sites, the spatial variations in the subsurface soil/rock conditions within the upper 100 feet are not substantial to cause significantly different ground shaking at different support locations. In such cases, the design horizontal ground motion at all support locations can be represented by a single ARS evaluated based on a representative soil/rock profile or V_{S30} value for the entire site.

For a bridge site, where V_{S30} values differ drastically between support locations, more than one design ARS should be provided. In general, different design ARS should be provided if the “Soil Profile Type”, as defined in Figure B.10 of the SDC v2.0, differs between supports by one or more letters (e.g., Soil Profile Type C at abutments and Soil Profile Type D or E at intermediate supports). This does not consider any effects due to wave scattering, and thus may not apply to long bridges (e.g. >1000 feet). For such bridges the variations in the ground motions between supports due to wave-passage effects can be significant.

Field Testing for In-situ Shear Wave Velocity

Soil/rock shear wave velocity is best obtained by direct field or in-situ measurement methods such as those listed in Table 1.

Table 1. Common Test Methods for In-situ Measurements of Shear Wave Velocity

Test Methods	Brief Description, Advantages and Disadvantages
Seismic Cone Penetration Test (SCPT)	The SCPT uses a fixed source at the surface to generate seismic shear waves in the soils. Receivers (seismometers) are placed on the SCPT rod at locations near and above the cone tip. Soil shear wave velocities are determined at discrete depth intervals. No predrilled borehole is necessary. Simultaneously obtains soil stratigraphic information and other useful data for geotechnical interpretation and evaluation of the subsurface conditions and design analysis. SCPT can be used to reduce the numbers of boreholes to be drilled, or to verify and correlate with an adjacent borehole drilled as part of a regular geotechnical site investigation. SCPT is fast and suitable for developing multiple shear wave velocity profiles at project sites, where necessary. Data resolution reduces with depth.
Rayleigh Wave Inversion	Measurements are made at the surface. Several variations of this method are available, including Refraction Microtremor (ReMi), Multichannel Analysis of Surface Waves (MASW) and Spectral Analysis of Surface Waves (SASW). Non-destructive method requiring no boreholes. Measures shear wave velocities of larger volumes of soil/rock. Suitable for V_{S30} determination. Inversion analysis does not result in a unique solution.
P- and S-Wave (PS) Suspension Logging	Shear wave measurements are made in an uncased or thermoplastic-cased borehole. Source and receivers have a fixed separation and are both within the borehole. Localized shear wave velocities are measured at discrete depth intervals. Good quality boreholes and coupling between the casing, when used, and the surrounding grout and borehole wall soils are required to obtain reliable soil or rock shear wave velocity measurements. Requires coordination with drill crew at borehole completion. Data resolution remains the same with depth.
Downhole Seismic Logging	The seismic wave source is fixed at the surface, and shear wave measurements are made with the receiver placed in the uncased or open borehole at discrete depth intervals. Uses the same basic seismic principles as the SCPT with data resolution decreasing with depths. Good quality borehole is required to obtain reliable soil or rock shear wave velocities. Requires coordination with drill crew at borehole completion.

Depending on the types of field measurements and methods of data interpretation, soil/rock shear wave velocities may be presented either in terms of the average shear wave velocity (V_s) for each interpreted uniform soil/rock layer or simply as a near continuous function of depth (d). In the latter case, when appropriate, it may be convenient to divide the upper 30 m of the soil/rock profile into a fewer number of relatively uniform soil/rock layers and their average shear velocities (V_{Si}) calculated from the measured shear wave velocities. The total number of layers (N) and the thicknesses D_i (i=1,N) of the individual layers should be determined based on the type (e.g., sand, silt, clay etc.) and density or consistency of the soil/rock, and the variations in the measured

shear wave velocity with depth. The greater the variations in the measured shear wave velocity with depth the smaller should be the layer thickness D_i.

Geophysical tests have their own limitations, and each test type has its own advantages and disadvantages compared with the others. Reliability of the shear wave velocities measured is dependent on many factors, including the equipment operator, test method or type used; the quality of the boreholes, when used, and the field measurements; the assumptions and the tools used for data analysis and interpretation, and the complexities of the actual subsurface conditions (Coe et al., 2018; NCHRP, 2006).

In conjunction with the typical geotechnical subsurface investigation, and where appropriate for use, SCPT is the preferred method for measuring soil shear wave velocities in the field for both V_{S30} determination and site-specific DGRA. It is also preferred when measurements at more than one locations are necessary at a project site due to spatial variability in the subsurface conditions.

Surface-based geophysical field tests (e.g., MASW) are preferred for V_{S30} determination at: (a) soil sites underlain by boulders, cobbles and/or significant fraction of coarse gravels or other obstructions, where SCPT data may not be reliable and/or refusal is anticipated at depths <30m, and (b) shallow rock sites, when shear wave velocity measurement is considered necessary.

PS Suspension Logging or Downhole Seismic methods may also be used for obtaining soil/rock shear wave velocities for both V_{S30} determination and site-specific DGRA. PS Suspension Logging should be used when shear wave velocity measurements need to extend significantly deeper than 30m. Table 1 provides additional information for each of the above field test methods.

Field measured shear wave velocity data is generally required for performing a site-specific DGRA. For these sites, the depth of measurements may need to extend to depths significantly greater than 100 feet (30 m) to reach bedrock whenever feasible. For deep soil deposits where it may not be feasible to reach bedrock, field measurements need to extend at least to the depth of very dense/hard soil (shear wave velocity > 1200 ft/sec or 360 m/sec). Field measurements should extend at least 10 to 20 feet into the very dense/hard soils to ensure that soil shear wave velocity is increasing with depth or at least not decreasing. For assistance in evaluating these situations, refer to an experienced seismic specialist.

Established empirical correlations for shear wave velocity, such as those included in Attachment 2, with other directly measured soil/rock parameters or properties, where available, may be used to estimate V_{S30} for preliminary evaluations. Use of empirical correlations for final design may be acceptable when:

- A site-specific DGRA is not required, and
- Additional subsurface field investigation is not considered necessary or planned for the preparation of the foundation report, or

- Subsurface field investigation at the project site will be performed but the V_{S30} is estimated to be at least 1000 ft/sec (300 m/sec) based on reliable information, and field measurement of shear wave velocity is considered not feasible. For such cases, an experienced seismic specialist may be contacted for further assistance.

When using empirical correlations, for clay soils those with directly measured undrained shear strength are preferred over CPT tip resistance. For sand and silts, the empirical correlations with CPT tip resistance are preferred. In general, for all soils, CPT tip based empirical correlations are preferred over those based on SPT blow counts.

ARS Online v3.0 Tool

The ARS Online v3.0 tool develops the SEE design ARS by running the UHT tool in the background to obtain the 2014 NSHM ARS data for 975 years return period, and by applying, where required, the modifications factors due to near-fault and/or basin effects.

The following site input parameters are required to run the ARS Online v3.0 tool. The latest available version of ARS Online v3.0.x tool shall be used, here "x" indicates the latest updated version.

- Site coordinates (latitude and longitude in decimal degrees), and the
- V_{S30} (meters/sec) for the design soil/rock profile.

A screen shot of the ARS Online v3.0 tool input page is shown in Figure 1.

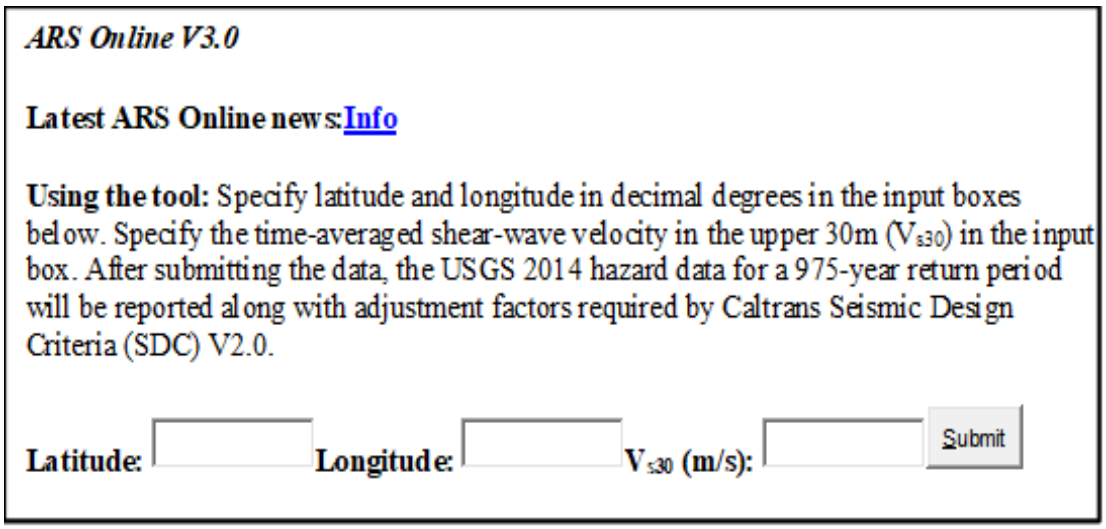


Figure 1: Input Page of the ARS Online v3.0 Web Tool

The 2014 NSHM provides hazards and ARS data for sites with $V_{S30} \geq 180$ meters/sec (590 feet/sec). For Caltrans project sites, the maximum value of the V_{S30} is limited to 1150 meters/sec. Therefore, when using the UHT for Caltrans project sites, the option for V_{S30},

that is the UHT "Site Class" input parameter, is limited to the following specific values only: 180, 259, 360, 537, 760 and 1150 meters/sec (590, 850, 1180, 1760, 2500, and 3775 feet/sec). These V_{S30} values are referred to hereafter in this document as the "Site Class" V_{S30}.

Additional input parameters, as discussed in Attachment 1, are necessary to run the UHT. For a site or support location with a design V_{S30} equal to one of the "Site Class" V_{S30}, the ARS Online v3.0 automatically enters the appropriate values for the additional input parameters required to run the UHT for 975 years return in the background.

For a site with V_{S30} not equal to one of the "Site Class" V_{S30} value, the ARS Online v3.0 determines the design ARS based on the interpolation of the two design ARS obtained for the two nearest bounding "Site Class" V_{S30} values, one higher and the other lower.

A screen shot of the ARS Online v3.0 result page for a site is shown in Figure 2.

ARS Online V3.0

Using the tool: Specify latitude and longitude in decimal degrees in the input boxes below. Alternatively, **Google Maps** can be used to find the site location. Specify the time-averaged shear-wave velocity in the upper 30m (Vs30) in the input box. After submitting the data, the USGS 2014 hazard data for a 975-year return period will be reported along with adjustment factors required by Caltrans Seismic Design Criteria (SDC) V2.0.

Latitude: 33.764645 **Longitude:** -118.2210 **Vs30 (m/s):** 180 Submit

Caltrans Design Spectrum (5% damping)

Period(s)	Sa_{2008}(g)	Sa_{2014}(g)	$Basin_{2008}$	$Basin_{2014}$	Near Fault Amp	Design Sa_{2008}(g)	Design Sa_{2014}(g)
PGA	0.49	0.51	1	1	1	0.49	0.51
0.10	0.76	0.82	1	1	1	0.76	0.82
0.20	1	1.12	1	1	1	1	1.12
0.30	1.06	1.31	1	1	1	1.06	1.31
0.50	1	1.33	1	1	1	1	1.33
0.75	0.86	1.13	1	1	1.09	0.93	1.22
1.0	0.74	0.99	1.02	1	1.17	0.88	1.16
2.0	0.46	0.61	1.05	1	1.17	0.56	0.72
3.0	0.29	0.4	1.06	1	1.17	0.36	0.47
4.0	0.21	0.27	1.06	1	1.17	0.26	0.32
5.0	0.17	0.2	1.06	1	1.17	0.21	0.23

Copy Table

Deaggregation (based on 2014 hazard)

mean magnitude (for PGA) 6.71

mean site-source distance (km, for Sa at 1s) 16.5

Option: recalculate Near Fault amplification with user specified distance

Site-source distance (km): 16.5 Update

Figure 2: ARS Online v3.0 Result Page

Figure 2 shows the specified site input parameters followed by a tabular presentation of the structure periods (in secs) in the 1st column and the corresponding SEE design spectral accelerations as "Design S_{a2014}(g)" in the last column. The table also includes the 2014 NSHM base ARS spectral accelerations values as "S_{a2014} (g)", and the corresponding basin amplification factors as "$Basin_{2014}$" and near-fault amplifications factors as "Near-Fault Amp" in the 3rd, 5th and 6th columns, respectively. The table also includes, for information and visual comparison only, ground motion parameters corresponding to the USGS' 2008 NSHM for the same return period.

The required source parameters from hazard deaggregation, i.e., the mean M and R values for the PGA and 1.0 second period spectral acceleration, respectively, are presented in the ARS data table in Figure 2.

A "**Copy Table"** button is included to copy all the data in the table into the computer's clipboard and paste in a spreadsheet for further processing and plotting. An additional option to calculate the near-fault amplification factors for any site based on a user specified R value is also included near the bottom of the result page.

Procedure for Developing SEE Design ARS

ARS Online v3.0 tool is used to determine the SEE design ARS. The recommended steps are:

STEP 1. Determine the site coordinates in terms of latitude and longitude (in decimal degrees)

STEP 2: Determine V_{S30} value(s) in meters/sec for the representative subsurface soil/rock profile(s) at the bridge support location(s). Evaluate if more than one design ARS is necessary at the site due to variations in the V_{S30}.

STEP 3: Determine if a site-specific DGRA is required to develop the final design ARS per Appendix B of the SDC v2.0. If not, continue with Step 4. Otherwise, contact an experienced seismic specialist for assistance.

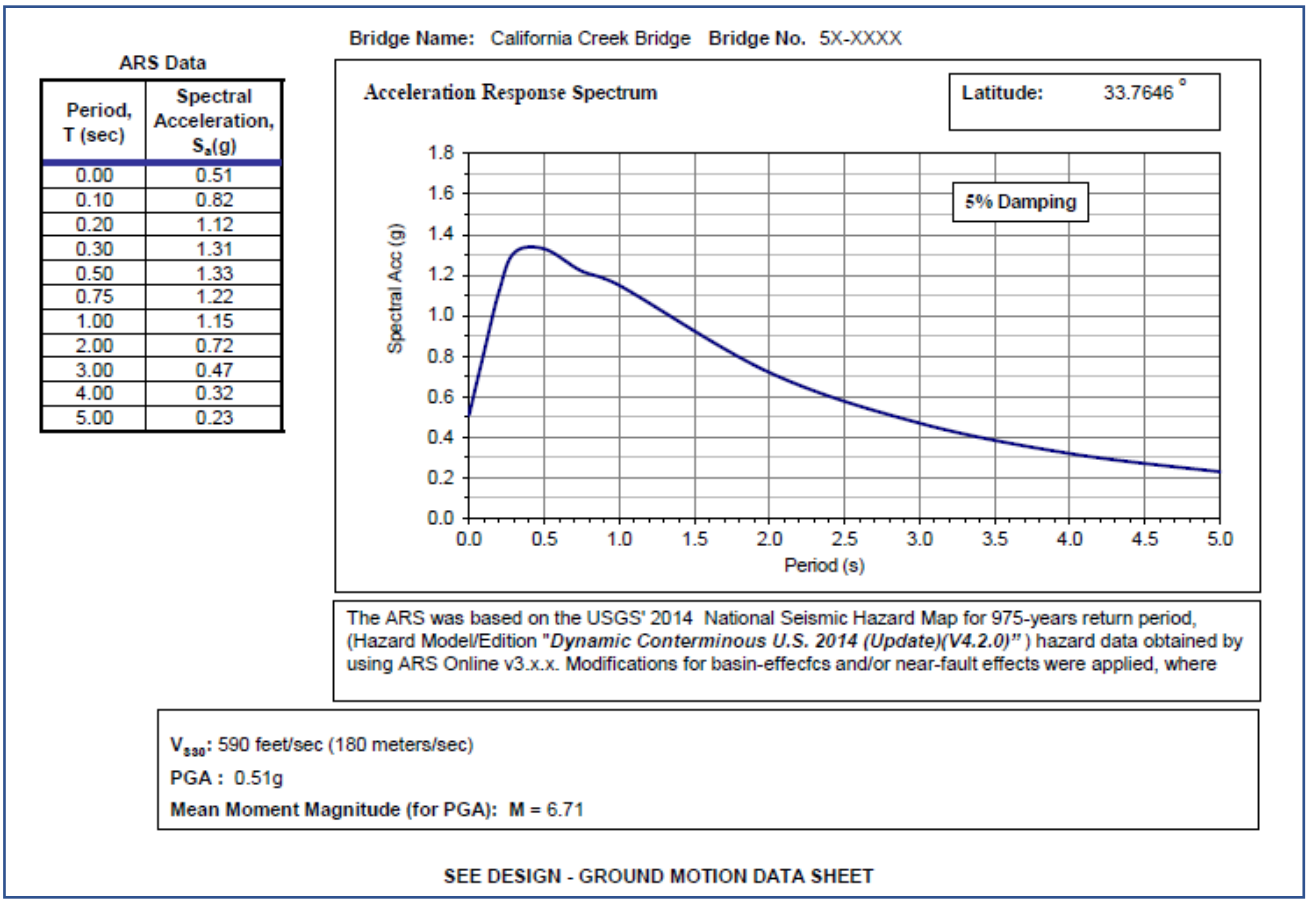
STEP 4: Open ARS Online v3.0 web tool available at https://arsonline.dot.ca.gov/. Enter the site latitude and longitude in decimal degrees and the design V_{S30} in meters/sec for the subsurface soil profile.

STEP 5: Click on the "Submit" button to run the tool. It may take a minute for the result page to show up.

STEP 6: Copy the ARS data table by clicking on the "Copy Table" button".

STEP 7: Paste the copied ARS data table into the Ground Motion Data Sheet (or equivalent spreadsheet). Change the "PGA" in the "Period(s)" column to 0.0 sec.

STEP 8: Plot and present the SEE design ARS data points as shown in Figure 3. Present all information shown on the Ground Motion Data Sheet for the project.



Bridge Name: California Creek Bridge Bridge No. 5X-XXXX

ARS Data

Period, T (sec)	Spectral Acceleration, S_a(g)
0.00	0.51
0.10	0.82
0.20	1.12
0.30	1.31
0.50	1.33
0.75	1.22
1.00	1.15
2.00	0.72
3.00	0.47
4.00	0.32
5.00	0.23

The ARS was based on the USGS' 2014 National Seismic Hazard Map for 975-years return period, (Hazard Model/Edition "*Dynamic Conterminous U.S. 2014 (Update)(V4.2.0)*") hazard data obtained by using ARS Online v3.x.x. Modifications for basin-effecfcs and/or near-fault effects were applied, where

V_{s30}: 590 feet/sec (180 meters/sec)
PGA : 0.51g
Mean Moment Magnitude (for PGA): M = 6.71

SEE DESIGN - GROUND MOTION DATA SHEET

Figure 3: SEE Design - Ground Motion Data Sheet

Step by Step Procedure for Developing FEE Design ARS

The FEE design ARS is provided only if requested by the Structure Designer. The same site- and design ground motion parameters are required as for the SEE design ARS, except for 225 years return period. Until included in a future version of the ARS Online web tool, the following step by step procedure is recommended to develop the FEE design ARS:

STEP 1: Determine the site latitude and longitude (in decimal degrees).

STEP 2: Determine V_{S30} value(s) in meters/sec for the representative subsurface soil/rock profile(s) at the support location(s). Evaluate if more than one design ARS is necessary for the site due to variations in the V_{S30}.

STEP 3: Determine if a site-specific DGRA is required to develop the final design ARS per Appendix B of the SDC v2.0. If not, continue with Step 4. Otherwise, contact an experienced seismic specialist for assistance.

STEP 4: If the design V_{S30} is not equal to one of the “Site Class” V_{S30}, determine the

nearest "Site Class" V_{S30}. The "Site Class" V_{S30} values are those that can be selected from the UHT "Site Class" drop-down menu. The "Site Class" V_{S30} values for Caltrans' project sites are 180, 259, 360, 537, 760 and 1150 meters/sec (Limit V_{S30} to 1150 meters/sec if site V_{S30}>1150 meters/sec).

As an example, the V_{S30} for a bridge project site was determined to be 205 meters/sec in Step 2. This V_{S30} is not equal to one of the "Site Class" V_{S30}. The nearest "Site Class" V_{S30} values are 180 meters/sec and 259 meters/sec. The average of these two nearest "Site Class" V_{S30} values 219.5 meters/sec. Since the site V_{S30} (205 meters/sec) ≤ 219.5 meters/sec, the nearest "Site Class" V_{S30} for this site is 180 meters/sec. For a different site, the design V_{S30} was determined to be 220 meters/sec in Step 2. For this site, since V_{S30} (220 meters/sec) >219.5 meters/sec, the nearest "Site Class" V_{S30} is 259 meters/sec.

STEP 5: Open the UHT: (https://earthquake.usgs.gov/hazards/interactive/).

STEP 6: Enter the input parameter options/values from Table 2.

STEP 7: Run hazard/ARS analysis by clicking on the "Compute Hazard" button that appears below the input parameter section. It may take a minute for the UHT to complete calculations and display results on the screen.

STEP 8: Once the results are shown, determine and record the ARS data points in a spreadsheet by clicking on the data symbols on the ARS plot. This ARS data set or (T, S_a) points correspond to the 2014 NSHM base ARS for 225 years return period. It is advantageous to keep open the UHT web page and the spreadsheet until all the steps are completed.

Table 2: UHT Input Options

Input Parameter	Option/Value
Edition	*Dynamic Conterminous U.S. 2014 (Update)(V4.2.0)*
Latitude	Site latitude (in decimal degrees) from Step 1
Longitude	Site longitude (decimal degrees) from Step 1
Site Class	"Nearest Site Class V_{S30}" from in Step 4
Spectral Period	Any one value of the available input options will work
Time Horizon (Return Period, yrs.)	225

STEP 9: Return to open UHT web page and select "Peak Ground Acceleration" (i.e., PGA) as the input for the "Spectral Period" parameter. Check and confirm that all other UHT input parameter options/values remained the same as those in Table 2.

STEP 10: Run hazard deaggregation for the PGA by clicking on the "Compute Deaggregation" button that appears below the hazard curves/ARS plot. UHT may take up to one minute to complete the calculations.

STEP 11: From the PGA hazard deaggregation results, find the value of the parameter "m" (magnitude) reported in the section under the "Mean (over all sources)" heading. This is the deaggregated mean earthquake moment magnitude M for PGA.

STEP 12: Return to the open UHT web page and select "1.0 Second Spectral Acceleration" option for the "Spectral Period" parameter. Check and confirm that all other UHT input parameter options/values remained the same as those in Table 2.

STEP 13: Run hazard deaggregation for the "1.0 Second Spectral Acceleration" by clicking on the "Compute Deaggregation" button.

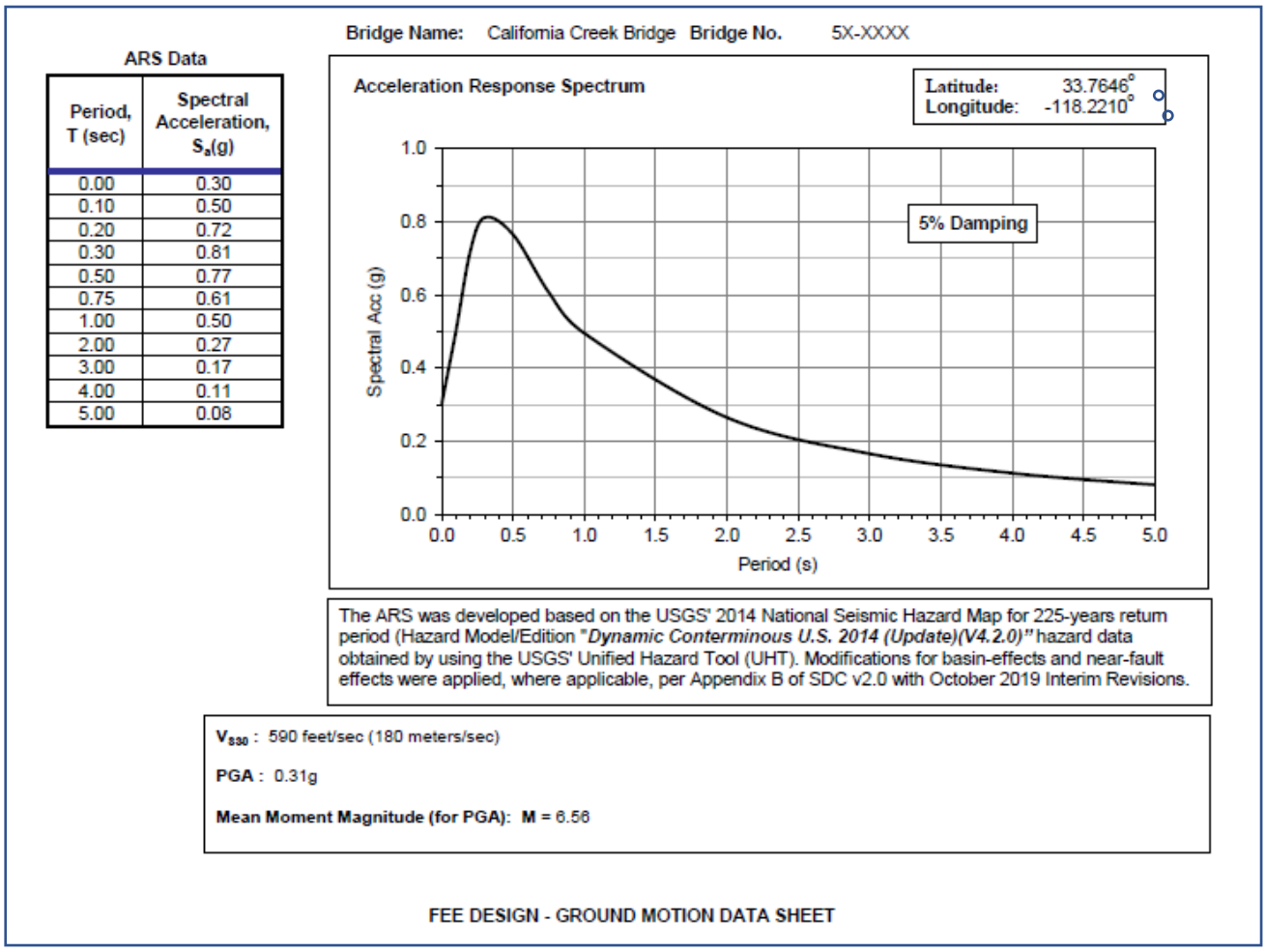
STEP 14: From the results of "1.0 Second Spectral Acceleration" hazard deaggregation analysis, find the value of the parameter "r" (distance, km) reported in the section under the "Mean (over all sources)" heading. This is the deaggregated mean site-to-source distance R (or R_{rup}) for 1.0 second period design spectral acceleration.

STEP 15: To determine the amplification factors for basin-effects, run ARS Online v3.0 for the site location (latitude and longitude from Step 1) and the nearest "Site Class" V_{S30} from Step 4. Copy the "$Basin_{2014}$" data column from the ARS data table (5^{th} column) and paste this data in the spreadsheet next to the 2014 basic ARS data. Keep the ARS Online v3.0 result page open.

STEP 16: To determine the near-fault factors, return to the open ARS Online v3.0 result page and replace the "site-source distance (km)" input value equal with the mean site-to-source distance R (for 1.0 sec S_a) determined in Step 14. Click on the "update" button. Once the updated results are available, copy the "Near-Fault Amp" data column from the ARS data table (6^{th} column) and paste in the spreadsheet next to the "Basin2014" data.

STEP 17: Determine the FEE ARS by multiplying the 2014 base ARS spectral acceleration values by the corresponding "$Basin_{2014}$" and Near-Fault Amp" factors.

STEP 18: Plot and present the FEE design ARS data points as shown in Figure 4. Include all data/information shown on the Ground Motion Data Sheet for the project site.



Bridge Name: California Creek Bridge Bridge No. 5X-XXXX

ARS Data

Period, T (sec)	Spectral Acceleration, S_a(g)
0.00	0.30
0.10	0.50
0.20	0.72
0.30	0.81
0.50	0.77
0.75	0.61
1.00	0.50
2.00	0.27
3.00	0.17
4.00	0.11
5.00	0.08

The ARS was developed based on the USGS' 2014 National Seismic Hazard Map for 225-years return period (Hazard Model/Edition "***Dynamic Conterminous U.S. 2014 (Update)(V4.2.0)***" hazard data obtained by using the USGS' Unified Hazard Tool (UHT). Modifications for basin-effects and near-fault effects were applied, where applicable, per Appendix B of SDC v2.0 with October 2019 Interim Revisions.

V_{S30} : 590 feet/sec (180 meters/sec)

PGA : 0.31g

Mean Moment Magnitude (for PGA): M = 6.56

FEE DESIGN - GROUND MOTION DATA SHEET

Figure 4: FEE Design - Ground Motion Data Sheet

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2014 National Seismic Hazard Map

The United Stated Geological Survey (USGS) developed the 2014 National Seismic Hazard Map (2014 NSHM) based on its 2014 long-term time-independent seismic hazard model (Peterson et al., 2015). This hazard model was developed utilizing the latest component models, data and methods available at the time for assessing earthquake-induced ground motion hazards.

The California portion of the 2014 NSHM is used as the basis to determine the design ARS for Caltrans project sites as specified in SDC v2.0. Detail information on the California portion of the 2014 NSHM, including the seismic source models, the fault source map, and the ground motion models (GMMs) or the ground motion prediction equations (GMPEs) used and other related documentation or references, can be found at https://earthquake.usgs.gov/hazards/hazmaps/conterminous/index.php#2014).

Orientation-Independent, Rotated Spectral Acceleration

With regard to the engineering applications, an important aspect of the 2014 NSHM is that the ground motion models used in its development no longer predict the spectral intensity of the horizontal ground motions in terms of the geometric-mean spectral acceleration, $(S_a)_{GM}$. The $(S_a)_{GM}$ is a scalar parameter and determined based on the spectral accelerations (S_a) of the two orthogonal components (field ground motion records are usually oriented at arbitrary azimuthal directions) of the horizontal ground motion.

The GMMs used predict the spectral intensity of the horizontal ground motions in terms of the recording instrument orientation independent, rotated single horizontal component spectral acceleration $(S_a)_{Rotxx}$, as defined by Boore (2010). Here, symbol “Rot” indicates that this spectral acceleration is calculated for a single horizontal ground motion component obtained by rotating the “as recorded”, two arbitrarily oriented orthogonal components of a horizontal ground motion event through all possible non-redundant rotation angles (0 to 180^o). The symbol “xx” represent the percentile of the all the rotated spectral accelerations calculated for a horizontal ground motion record (orthogonal pairs).

These GMMs provide predictions of the mean $(S_a)_{Rotxx}$, designated as $(S_a)_{RotD50}$ and the associated standard deviation necessary for the development of the hazard curves. Here, the symbol “D” indicates that the rotation angle for the special acceleration is dependent on the structure period, and “50” indicates the 50th percentile of the spectral accelerations $(S_a)_{Rotxx}$ obtained for all non-redundant rotation angles.

Based on the above spectral intensity measure definition and the calculation procedure used, the $(S_a)_{Rotxx}$ for a horizontal ground motion ranges from a minimum of $(S_a)_{RotD00}$ to a maximum of $(S_a)_{RotD100}$, $(S_a)_{RotD50}$ being the median value (See Boore, 2010). The GMMs used in the development of the 2014 NSHM predict the median spectral acceleration $(S_a)_{RotD50}$ and the associated standard deviation.

Seismic Source Models for California

The seismic hazard model for the California portion of the 2014 NSHM is based on the Time-Independent Model (Field et al, 2014) component of the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3). The UCERF3 model consists of three new and updated component models: fault source model, seismic deformation (slip rates) model and long-term earthquake rate model. The UCERF3 used updated seismic source models consisting of fault-based (system) and gridded seismicity-based (background) sources to estimate future earthquake hazards from known active fault sections and those from unknown or unidentified sources, respectively. It also considers rupture scenarios involving multiple fault segments.

Earthquakes from gridded seismicity-based (background) sources are represented as points or planar fault sources at the centers of evenly spaced grid cells that make up the UCERF3 forecast region. The UCERF3 forecast region cover the part of the State California where shallow crustal earthquakes occur. This includes most of the State, except the northwestern coastal region where the Cascadia subduction interface and deep subduction intra-slab earthquake events dominate the hazard. The gridded seismicity-based sources are used to predict future, distributed earthquake occurrences to account for the fact that many significant earthquakes do not occur on known, mapped faults.

Detail information on the UCERF3, including the fault-section database and the main report titled "Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3)-The Time-Independent Model (USGS OFR 2013-1165)", and the many appendices and supplemental information can be found at https://pubs.usgs.gov/of/2013/1165/.

The UCERF3 provides updated (relative to the 2008 USGS NSHM) estimates of earthquake magnitudes, locations and rates of earthquakes of moment magnitudes $M \geq 5.0$ from shallow crustal sources in California. In addition to UCERF3, the 2014 USGS NSHM also includes an updated source model for the Cascadia subduction interface and deep subduction intra-slab earthquake events for the northwest part of California.

Ground Motion Models for California

The California portion of the 2014 NSHM uses three categories of GMMs for predicting ground motion intensity depending on the tectonic settings.

For shallow crustal earthquake events from the UCERF3 source model, all five of the following NGA (Next Generation Attenuation) -West 2 GMMs are used. These include Bozorgnia et al (2013, 2014), Abrahamson et al (2013, 2014), Boore et al (2013, 2014, Campbell and Bozorgnia (2013, 2014), Chiou and Young (2103, 2014); and Idriss (2013, 2014) for the reference firm site conditions only (i.e., V_{S30}=760 meters/sec).

The GMMs used for the Cascadia subduction interface earthquake events include Atkinson and Boore (2003) global model, Atkinson and Macias (2009), Addo et al (2012), Zhao et al (2006), and BC Hydro (Addo et al, 2012)

The GMMs used for the Cascadia deep intra-slab earthquake events include Atkinson and Boore (2003) global model, Atkinson and Boore (2003) Cascadia model, Zhao et al (2006), and BC Hydro (Addo et al, 2012).

The use of the updated as well as new source models and GMMs, with the new definition of the spectral acceleration, resulted in differences between the probabilistic ground motions in the 2008 and 2014 NSHMs, which were used as the basis for the development of the design ARS per the SDC v1.7 and the SDC v2.0, respectively.

Unified Hazard Tool (UHT)

The web-based UHT is developed and made available by USGS for determining the ARS data and related information, including hazard deaggregation for selected periods, for various versions of the USGS' NSHMs. The UHT can be accessed at https://earthquake.usgs.gov/hazards/interactive/.

A screen shot of the UHT front page is shown in Figure A-1. The available options under each panel can be accessed by expanding it with a click on the down-arrow next to the panel title. The UHT provides 2014 NSHM ARS data or spectral accelerations for periods, T= 0.0 (PGA), 0.1, 0.2, 0.3, 0.5, 0.75, 1.0, 2.0, 3.0, 4.0 and 5.0 seconds.

As seen in the Figure A-1, the UHT input parameters include:

- "Edition" for the hazard model. For developing design ARS per SDC v2.0, the appropriate hazard model and edition is identified as *"Dynamic Conterminous U.S. 2014(Update)(V4.2.0*)" in the drop-down menu.
- Latitude" and "Longitude" (in decimal degrees) for the site location co-ordinates. The site latitude and longitude can be entered directly or by utilizing the "Choose location using a map" option.
- Site Class" for the site V_{S30} (meters/sec). The "Site Class" can be selected from one of the options available in the drop-down menu. For Caltrans project sites, the available option for the V_{S30} value includes 180, 259, 360, 537, 760, and 1150 meters/sec.

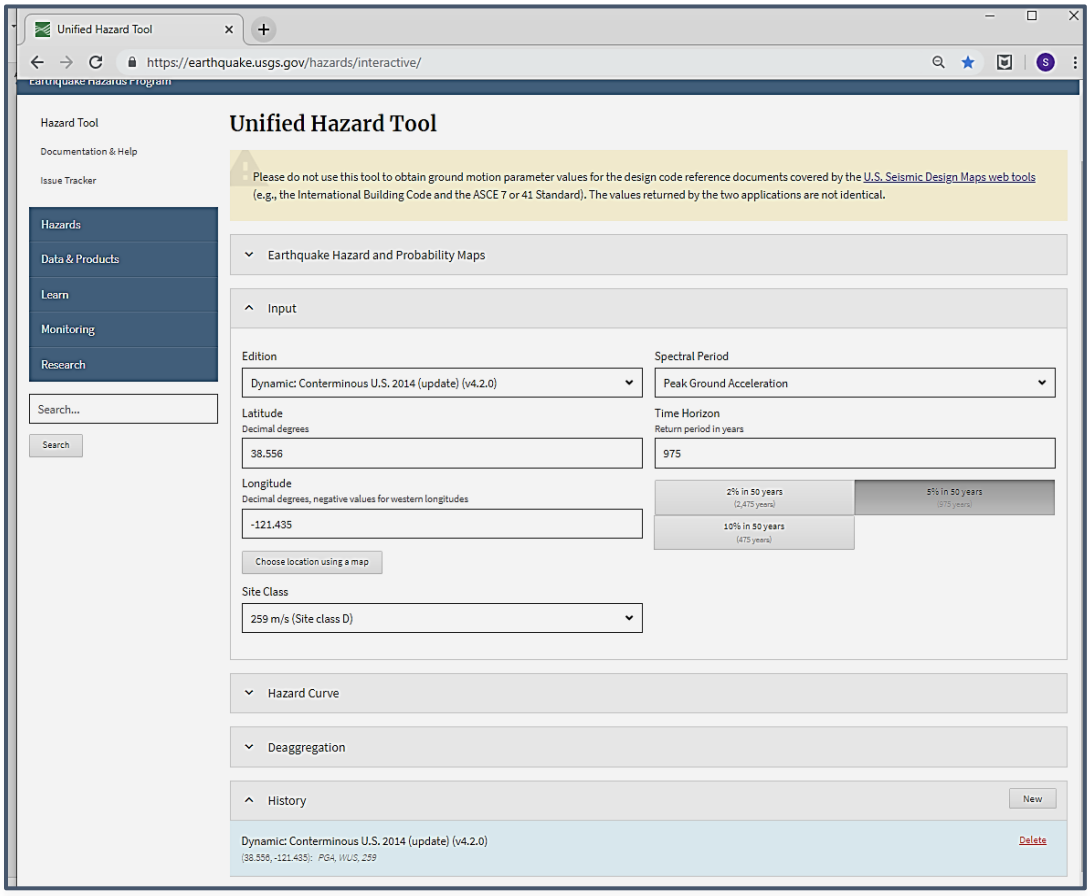


Figure A-1: UHT Front Page (Accessed on January 27, 2020 at https://earthquake.usgs.gov/hazards/interactive/)

- "Spectral Period" for a spectral acceleration corresponding to one of the period value from the following list: 0.0 (PGA), 0.1, 0.2, 0.3, 0.5, 0.75, 1.0, 2.0, 3.0, 4.0 and 5.0 seconds. For the hazard (or ARS) analysis only, any of the available "Spectral Period" option can be selected from the drop-down menu. To run the deaggregation analysis for a particular period-specific spectral acceleration, it must be the option entered by selecting from the drop-down menu.
- "Time Horizon (return period in years)" for the return period can be entered directly or, if available, by clicking on the corresponding button. To obtain the 2014 USGS NSHM ARS data for use in the development of the SEE design ARS, the "Time Horizon/Return period in years" must be 975 years. To determine 2014 NSHM ARS data for use in the development of the FEE design ARS, the "Time Horizon/Return period in years" must be 225 years.
- Once all the input parameter values are entered, the hazard analysis for the input return period is performed by clicking on the "Compute Hazard Curve" button that appears in the expanded "Hazard Curve" panel. Once the calculations are

completed, the results of the analysis are present on the screen as plots of the hazard curves for the site and the ARS curve for the input return period.

The exact values of the plotted ARS data points can be obtained by clicking once on the data point symbols. These ARS data points corresponds to the 2014 NSHM ARS for the specified input return period. Modifications to these ARS data points are applied, where necessary, for basin-effects and/or near-fault effects, as specified in Appendix B of the SDC v2.0.

Hazard deaggregation analysis for a period-specific spectral acceleration is performed by selecting it from the “Spectral Period” drop-down menu, making sure all other input parameters remained unchanged and then clicking on the “Compute Deaggregation” button available in the “Deaggregation” analysis panel below the ‘Input” panel. The results of the analysis are presented on the screen. The analysis must be repeated for each period-specific spectral acceleration for which the results of hazard deaggregation analysis are required.

Ground Motion Directionality

The azimuth direction of the design horizontal ground motion component represented by the USGS’ 2014 NSHM basic ARS at a site is random (i.e., equal probability of occurrence in any direction on the horizontal plane).

The design ARS determined per Caltrans SDC v2.0, which includes only the basin effects, where necessary, also represents the randomly oriented mean (or average) component of the design horizontal ground motion at the site. The design ARS obtained by applying near-fault effects represents the Fault-Normal (FN) component for near-fault sites.

Empirical Correlations for Estimating Shear Wave Velocity

In the absence of field measurements, the shear wave velocity Vs for most soil and soft sedimentary rock formation may be estimated based on established empirical correlations with other field and/or laboratory measured site and layer-specific soil/rock properties or index parameters, if available.

The geo-professionals should be aware of the limitations of the empirical correlations for Vs, including the soil/rock properties or parameters uses. For example, penetration of the SPT sampler in earth material may be limited or affected by the presence of large particles (e.g. gravel, cobbles, boulders or rock fragments). Correlations, in particular using SPT data, should only be used with test data and other related information that are reliable and representative of the actual site conditions. If established empirical correlations are not applicable (e.g. SPT correlation used in a thick, coarse gravel deposit) or not available, then in-situ measured shear wave velocities should be used.

For cohesive soil layers, empirical correlations with laboratory measured undrained shear strength (S_u) are preferred. For cohesionless soil layers, correlations with CPT (ASTM D 5778) tip resistance are preferred. In the absence of CPT tip resistance, SPT (Standard Penetration Test – ASTM D1586) blow count, N_{60} (blow counts corrected for hammer efficiency but not for overburden pressure) may be used to estimate V_S for cohesionless soil layers.

For young sedimentary rock deposits that display soil-like properties, established correlations with SPT blow counts may be used to estimate layer V_S values. For stronger rock, V_{S30} may be estimated based on the field measured V_S for nearby sites underlain by similar deposits; or based on published measured V_S or V_{S30} data or surface geology based V_{S30} maps available for, the same geographically and geologically-defined rock unit or formation and with similar intact rock and rock-mass characteristics. These characteristics include, but not limited to uniaxial compressive strength, small-strain modulus, Poisson's ratio, RQD, hardness, degree of weathering, fracture density and conditions, ultrasonic seismic shear and/or compression wave velocities (Mayne et al., 2001).

Recommended empirical correlations are presented below. Other well-documented and established empirical V_S correlations specific to the earth material under consideration at the project site or the general area with similar earth material, may be used by experienced geo-professionals provided adequate justifications of their use and the pertinent references are included in the report.

Recommended Empirical Correlations

Wair et al (2012) compiled published correlations between layer shear wave velocity and common in-situ geotechnical test parameters and presented recommended correlations for various soil types. The empirical correlations recommended below are based on a review of this and other noted references.

Soil Layers

For the SPT based empirical corrections presented below, the parameter N_{60} is obtained by correcting the field measured SPT blow count (N_m) for variations from the ASTM Standard D1586, including the hammer energy, borehole diameter, rod length and sampler without liner (see *Liquefaction Evaluation* Module or Youd et al. 2001). No correction due to the current effective overburden stress (σ'_{vo}) is applied, and these correlations are valid for $3 \leq N_{60} \leq 100$.

Note that the recommended empirical correlation presented below provide estimates of the seismic shear wave velocity V_S in meter/sec at the depth of measurement of the correlated parameter (e.g., S_u or CPT tip resistance). Unless stated otherwise, the unit of (σ'_{vo}) is kPa. Attention must also be paid to the specific units or definitions used for the other correlated parameters.

Cohesive Soil Layers

The following correlation by Dickenson (1994) based on the undrained shear strength (S_u) is recommended for cohesive soil layers:

$$V_s = 203 \left(\frac{S_u}{p_a}\right)^{0.475}$$

Where, p_a is the atmospheric pressure (same unit as S_u). The undrained shear strength (S_u) should be measured as a function of depth either in-situ by field vane shear test or in the laboratory by unconsolidated undrained (UU) tests, or as a function of the effective overburden stress (σ'_v) by consolidated undrained (CU) tests, on undisturbed samples.

In the absence of measured undrained shear strength data, the shear wave velocity for cohesive soils may be estimated by using the following empirical correlation with CPT tip resistance developed by Mayne and Rix (1995):

$$V_s = 1.75 \, (q_t)^{0.627}$$

Where, q_t is the measured CPT tip resistance (kPa) corrected for water pressure effects due to unequal end area (See Robertson, 2016).

If S_u or CPT tip resistance (q_t) data are not available, the following correlation with the corrected SPT blow count (N_{60}) developed by Wair et al (2012) may be used to determine V_S for cohesive soil layers:

$$\mathrm{Vs} = 26 \, (ASF) \, (N_{60})^{0.17} \, (\sigma'_{vo})^{0.32}$$

Here, ASF = Dimensionless Age Scaling Factor for geologic age of the soil deposit (=1.0 for Quaternary or 0.88 for Holocene, and 1.12 for Pleistocene, if known).

Cohesionless Soil Layers

The following correlation developed by Baldi et al (1989) is recommended to calculate V_S for uncemented cohesionless soil layers when using CPT data:

$$V_S = 277(q_t)^{0.13} (\sigma'_{vo})^{0.27}$$

Here, both q_t and σ'_{vo} are in MPa

In the absence of CPT data, the following correlations developed by Wair et al (2012) are recommended for cohesionless soil layers when using SPT data:

Silts

The SPT N_{60} correlation recommended above for cohesive soil layers is also recommended for silt layers.

Sands

$$Vs = 30\ (ASF)\ (N_{60})^{0.23}(\sigma'_{vo})^{0.23}$$

Where, ASF =1.0 for Quaternary or 0.9 for Holocene and 1.17 for Pleistocene.

Gravels

$$Vs = 53(N_{60})^{0.19}(\sigma'_{vo})^{0.18}$$ for Holocence

$$Vs = 115(N_{60})^{0.17}(\sigma'_{vo})^{0.12}$$ for Pleistocene

Young Sedimentary Rock Formations

Imai and Tonouchi (1982) reviewed over a hundred SPTs with corresponding field measured V_S in young sedimentary rocks (Tertiary deposits) and developed the following correlation for “Tertiary Sand/Clay”.

$$V_S = 109\ (N_{60})^{0.319}$$

If site-specific measured shear wave velocity data are not available, this empirical correlation may be used for young sedimentary rock deposits in California. Limit the maximum value of V_S obtained from this correlation to 560 m/sec.

Other Rock Formations

There are relatively few studies available that correlate seismic shear wave velocity to physical properties of other rock masses. Two notable studies that may be useful to geo-professionals in estimating approximate seismic shear wave velocities in rock, based on physical properties of rock masses are: (1) Fumal (1978), which correlated shear wave velocity to weathering, hardness, fracture spacing, and lithology based on data from 27 sites in the upland areas of the San Francisco (Bay Area) region, and (2) Fumal and Tinsley (1985). which extended the 1978 study to include 84 sites in the Los Angeles region. Some physical properties of the rock were more important than others depending on lithology, texture and hardness, but fracture spacing was found to be the most important factor affecting shear wave velocity. A thorough review of these studies will significantly aid geo-professionals in their estimations of the shear velocities in rock formations.

For Caltrans projects, the V_{S30} for rock sites should be limited to 760 meters/sec when in-situ measured $V_s(d)$ data are not available.

Estimating V_{s30} for Sites with Depth of Exploration 10 to 29m

The V_{S30} estimated based on the below extrapolation scheme should not be used to develop the final design ARS when the assessed site Soil Profile Type is D. This extrapolation scheme should not be used for project sites: (1) underlain by unusual soil profiles not used in its development, or (2) for which a site-specific DGRA may be required per Appendix B of the SDC v2.0.

For sites for which the subsurface information is available only for the upper 10 to 29 m, V_{S30} in meters/sec for assessment of the site “Soil Profile Type” as defined in Table B.10 of the SDC v2.0, may be estimated by extrapolation using the following empirical correlation developed based on Boore (2004):

$$V_{S30} = (1.45\text{-}0.015*d)V_{sd}$$

Where, V_{sd} (meters/sec) is the time-averaged shear wave velocity for the upper “d” meters of soil/rock for which subsurface information is available ($10 \leq d \leq 29$ meters). V_{sd} is calculated using the same equation as V_{S30}, except that the total depth of calculation is limited to “d” meters. This equation was developed based on the assumption that that shear wave velocity increases with depth and no significant abrupt changes in the subsurface conditions, particularly in the small-strain stiffness of the soil/rock, occur between the depths of 10 and 29 m.

Liquefaction Evaluation

Soil liquefaction may substantially increase the cost of bridge and highway projects. If liquefaction hazard is not reported in a timely manner, there may be an inaccurate allocation of funds and resources. For instance, if the soil is incorrectly characterized to liquefy, the project engineer may develop an expensive preliminary design with large diameter piles, deep foundations, or unnecessary soil mitigation measures. Conversely, if the project engineer incorrectly assumes that liquefaction will not occur then the project may be significantly delayed because the project cost estimate did not account for liquefaction design.

Liquefaction investigations during the planning phase (K or 0) are usually limited to the evaluation of existing information for a site (see Geotechnical Investigations). If liquefaction hazard potential is *unknown or cannot be reliably determined based on the available information* prior to Type Selection, an unsuitable foundation type may be selected. If the potential consequences of liquefaction appear to be substantial, the Geoprofessional should discuss with the Project Development Team (PDT) the option of performing site-specific subsurface investigations prior to Type Selection to better determine the liquefaction hazard. Refer to Memos to Designers (MTD) 20-14, "*Quantifying the Impacts of Soil Liquefaction and Lateral Spreading on Project Delivery*."

This module presents:

- The criteria for preliminary screening or assessment of liquefaction potential at a site based on available information (e.g., published liquefaction hazard maps, As-Built Log of Test Boring, etc.) The results of this (generally qualitative) preliminary screening are reported in the DPGR, SPGR, or PFR.
- The guidance for site investigations in potentially liquefiable soils.
- The methodology for performing quantitative analysis of liquefaction potential to determine if liquefaction will occur based on site-specific field exploration and laboratory testing data in accordance with Youd et al (2001) and Boulanger and Idriss (2006). This information will be presented in an FR and/or a GDR.
- Appendix A: Cone Penetration Test (CPT) Example (Youd et al., 2001)
- Appendix B: Standard Penetration Test (SPT) Example (Youd et al., 2001)

This module does not address the effects of liquefaction on a project, mitigation of the liquefaction hazard, or geotechnical design in areas of liquefaction.

Preliminary Screening of Potentially Liquefiable Soils

Preliminary screening involves the evaluation of a site for soil liquefaction potential using existing information such as liquefaction hazard maps such as State "Seismic Hazard and Zones of Required Investigation Maps" and USGS "Liquefaction Hazards Maps", groundwater information, reports, as-built plans/subsurface data, and in some cases existing sampling and testing. The purpose of the preliminary screening is to alert the PDT of the potential for soil liquefaction to occur at the site.

Use the following five (5) criteria for preliminary screening evaluation of soil liquefaction potential at a project site.

1. Groundwater Table (historic, current, or anticipated future level)
2. Age of Soil
3. Soil Type
4. In-situ Soil Density/Initial State
5. Design Peak Ground Acceleration and Earthquake Moment Magnitude

Use all five (5) criteria to make a preliminary screening assessment. If the preliminary screening indicates no liquefaction potential, the site investigation may be planned as if liquefaction is not expected. If there is a potential for liquefaction, or the potential is unknown, the Geoprofessional must report as discussed below, and the field investigation planned to include appropriate (CPT) soundings, drilling method and SPT blow count measurements and sampling of the potentially liquefiable soils. All field and laboratory works must be performed in accordance with the applicable ASTM standards.

1. Groundwater Table

Sites with potentially liquefiable soils and groundwater table within 50 feet of the ground surface are generally considered to be most susceptible to liquefaction. If the groundwater table is anticipated to be shallower than 50 feet, the Geoprofessional should anticipate the need for and plan to collect samples and perform necessary field tests for liquefaction potential, unless the potential for liquefaction hazard can be confidently ruled out based on other criteria (items 2-5 above). If the groundwater is known to be below 70 feet, the site may be considered non-liquefiable for preliminary screening purposes

Sources of groundwater data include "As-Built" boring logs, County well log data, water wells (Department of Water Resources), and historic ground water levels (USGS) and ground water plates published in the State Seismic Hazard Zone map reports.

2. Age of Soil

Holocene deposits (younger than 11,000 years) and man-made fills ranging from very loose to medium dense are susceptible to liquefaction. Geologic deposits older than Holocene age (> 11,000 years) are considered to have low liquefaction susceptibility. A source of soil age data is the CGS California Geologic Map for the site.

3. Soil Type

Soils types that are susceptible to liquefaction are sand, silty sand, low plasticity (PI<7) silt and, in unusual cases, gravel. Rock and most clay soils are not liquefiable.

4. In-situ Soil Density/Initial State

Granular/cohesionless soils with an initial state represented by a normalized, clean-sand equivalent CPT resistance $(q_{c1N})_{cs}$ >160, or SPT blow count $(N_1)_{60cs}$ >30 are considered not susceptible to liquefaction irrespective of the other criteria or conditions.

5. Peak Ground Acceleration and Earthquake Magnitude

Liquefaction potential increases with increasing Peak Ground Acceleration (PGA) and earthquake magnitude (M). The site design PGA and M correspond to a return period of 975-years and are obtained from ARS Online webtool.

Field Investigation

If preliminary screening indicates liquefaction potential exists or is unknown, the field investigation should gather information for liquefaction assessment including the soil characteristics (classification/type, grain size distribution, density, Atterberg Limits) and spatial distribution of the potentially liquefiable soil, and the groundwater level. Typical field investigations use rotary wash or auger SPT sample borings and laboratory test samples, and/or CPT soundings as per applicable ASTM Standards. The depth of the exploration must be 70 feet below ground surface or 20 feet below the pile design tip elevation, whichever is greater.

Because the presence of liquefiable soils can substantially increase project costs, it is important to thoroughly characterize the site. The site exploration needs to determine if the liquefiable soils are extensive enough, both laterally and vertically, to constitute a hazard. Depending on the field conditions, geophysical methods, and CPT soundings can provide information on the lateral extent of liquefiable layers more economically than SPT borings.

CPT Soundings

CPT soundings offer advantages over other methods of estimating liquefaction resistance in both the detection of thin layers that may influence liquefaction triggering and subsequent pore pressure redistribution and in the reproducibility of results. CPT results are less dependent on the equipment operator than most other in situ test methods and CPT can be performed quickly and cheaply. The seismic CPT can also measure soil shear wave velocity (National Academies of Sciences, 2016) CPT does not only identify the presence of liquefiable soils, but it can also show if the liquefiable soils are extensive enough, both laterally and vertically, to constitute a hazard.

When the CPT, without collecting any soil samples, is the primary investigative tool it is highly recommended that at least one SPT boring be performed “side by side” to a CPT sounding location for each bridge structure. This provides soil samples for laboratory testing to determine the required soil parameters (e.g., fines content, Atterberg Limits) and at least some limited site-specific correlations between SPT and CPT results. Whenever feasible, consider using CPT in the final liquefaction evaluation, in particular for sites with thinly bedded and/or highly variable subsurface soil conditions.

Standard Penetration Test (SPT)

SPT has been the most commonly used investigative tool for liquefaction evaluation. For large or important projects, in particular with highly variable subsurface conditions, SPT should be supplemented with CPT. Unlike the CPT, the SPT provides actual soil samples that can be visually examined and tested in the laboratory to evaluate soil parameters needed for more reliable liquefaction analysis.

Do not use hollow stem augers below the water table, or any non-standard exploration and/or testing methods for liquefaction assessment using either the SPT and/or the CPT.

Ground Water Level

Measure the elevation of the stabilized groundwater table or, where appropriate, of the piezometric surface in the borehole or piezometer. The CPT pore pressure dissipation test can also be used to determine the elevation of the groundwater table or the piezometric surface. Use a higher elevation than measured only if there is clear evidence for seasonal or long-term fluctuations. Do not use abnormally high or temporary groundwater level.

Geophysical Investigation

In gravelly soils where SPT blow counts are unreliable (or at depths greater than 70 feet) consider the shear wave velocity (Vs) method for performing liquefaction assessments (Andrus and Stokoe, 2000; Youd et al., 2001). The seismic cone, P-S logging, or surface wave methods are available to obtain shear wave velocity.

Laboratory Testing

Laboratory tests for quantitative liquefaction evaluation include:

- Particle Size Analysis (ASTM D 422)
- % Finer Than the No. 200 (75 μm) Sieve (ASTM D 1140)
- Atterberg Limits (ASTM D 4318)

The exact type(s) and the number of laboratory tests performed shall be selected based on the visual-manual description and identification (ASTM D2488), and the extent and field variability of the identified potentially liquefiable soils.

Results from Atterberg Limit test(s) are necessary when the amount (%) and/or the plasticity index of the fines content are used in the final determination of an otherwise liquefaction susceptible soil as not liquefiable.

Quantitative Liquefaction Analysis

Quantitative liquefaction analysis uses site-specific field and laboratory test data. Use the procedure of Youd, et al (2001): *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils October 2001* for quantitative liquefaction analysis. The procedure for CPT and SPT analysis consists of the following general steps:

1. Determine the design PGA and earthquake moment magnitude (M)
2. Determine the design groundwater table elevation
3. Determine which soil layers are susceptible to liquefaction and thus need to be evaluated based on a quantitative liquefaction analysis
4. Determine the total soil unit weight for all soil layers or sublayer located above the bottom elevation of the lowest soil layer susceptible to liquefaction. Then perform the following steps for each of the soil layer/sub layer identified in Step 3 as susceptible to liquefaction
5. Determine the Cyclic Stress Ratio (CSR) at the mid elevation
6. Determine the representative values of the field measured CPT tip and frictional resistances or the SPT blow count
7. Correct and normalize the measured CPT tip resistance/SPT Blow Count
8. Determine the Fines Content Correction and the clean-sand equivalent CPT
9. Determine the clean-sand equivalent normalized CPT tip resistance/SPT Blow Count and Calculate Cyclic Resistance Ratio $(CRR)_{7.5}$
10. Calculate the Magnitude Scaling Factor (MSF), overburden correction (K_σ) and the sloping ground correction factor (K_α).
11. Calculate the Factor of Safety Against Liquefaction

Modifications or elaborations to Youd et al (2001) are as follows:

- Use Youd et al (2001) to depths of 50 feet; with caution to 70 feet; do not use below 70 feet.
- Liquefaction evaluation below 70 feet require special analysis and consideration that is beyond the scope of this module.
- Do not combine liquefaction analysis with other extreme events or conditions (e.g., vessel impact, and abnormally high or temporary groundwater levels).
- Consider scour in liquefaction evaluation as specified in Article 10.5.5.3 of the AASHTO LRFD Bridge Design Specifications with California Amendments.

- Do not use the "Modified Chinese Criteria" as it is unconservative for determining if certain fine-grained soils are liquefiable. Use Boulanger and Idriss (2006) to determine if fine grained soils are likely to behave as sand-like material and are thus should be considered as susceptible to liquefaction.
- Use the design PGA (5% probability of exceedance in 50 years or 975-year return period) in units of g (e.g., PGA=0.4g) evaluated as per Appendix B of the Seismic Design Criteria (SDC) and using ARS Online webtool. Here, g is acceleration of gravity. (Note: Youd et al, 2001 uses the parameter a_{max} for PGA in units of g)
- Use the de-aggregated mean earthquake moment magnitude (M) for PGA as the design earthquake magnitude (M_w). (Note: Youd et al, 2001, uses both the symbols M and M_w to denote the moment magnitude of the design earthquake)
- Use a factor of safety against liquefaction of 1.0
- See Appendices A and B for examples of CPT and SPT based liquefaction evaluation, respectively.

Reporting

Liquefiable soil can have significant impacts on a project's scope, schedule and budget; it is important to communicate liquefaction information to the PDT in a timely manner. The content and confidence of recommendations, especially during the planning phase, will depend largely on the type of information available. If as-built LOTB and laboratory data are available during the project early stages, then quantitative liquefaction analysis may occur earlier than typical, and the results presented in the DPGR or SPGR. If little information is available, then only a qualitative assessment, i.e. preliminary screening, can be presented

Liquefaction potential is discussed in the following reports:

- District Preliminary Geotechnical Report (DPGR)
- Preliminary Geotechnical Design Report (PGDR)
- Geotechnical Design Report (GDR)
- Structure Preliminary Geotechnical Report (SPGR)
- Preliminary Foundation Report (PFR)
- Foundation Report (FR)

Preliminary Reports

Language used in reporting liquefaction potential must be clear and direct. Do not use indefinite terms such as "*low*", "*moderate*" and/or "*high*". Acceptable language for preliminary reports (SPGR, PFR, DPGR, and PGDR) includes:

- "Liquefaction potential exists",

- “Liquefaction potential is unknown or cannot be determined based on the available information”,
- “Liquefaction potential does not exist”.

If the results of preliminary screening (reported in the SPGR or DPGR) indicate that liquefaction potential either exists or is unknown, the Project Development Team (PDT) should decide whether to perform some or all site investigations, including drilling, prior to type selection to more accurately evaluate liquefaction potential for the PFR and PGDR. For District items, such as standard plan structures and embankments, the PDT should decide whether liquefaction will be considered in the design. If the design will not be modified for liquefaction it is unnecessary to perform quantitative liquefaction analysis and related field work.

Final Reports

The results of the site investigation and quantitative analysis must be reported in the PFR and PGDR (if applicable), FR and/or GDR. The report should include:

- Areal limits of liquefaction
- Vertical limits of liquefaction
- Identification of liquefiable soils
- For complex projects, or if requested by the designer, include a three-dimensional plot of liquefiable soils at the site.

<Discussion of the consequences and mitigation of liquefaction hazards are, or will be, in separate modules. This would include: lateral spreading, seismic settlement, layer thickness, extent, connectivity, etc.>

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Liquefaction Evaluation Example (CPT Method)

Seal Beach Blvd Overcrossing Bridge replacement is a proposed 2 span cast-in-place prestressed concrete box girder structure. A quantitative liquefaction analysis is required following the procedures described in Youd et al, (2001).

The following information is required:

- Seismic design ground motion parameters including peak ground acceleration (PGA) corresponding to a return period of 975-years, and the de-aggregated mean earthquake moment magnitude (M) for PGA. These data are obtained by running ARS Online webtool using the latitude and longitude 33.7736 N, -118.0749 W.
- Soil data from CPT results. (Figure 1)
- Groundwater data as shown on the LOTB dated April 24, 2008. (Figure 2)

Figure 1: CPT Sounding Data

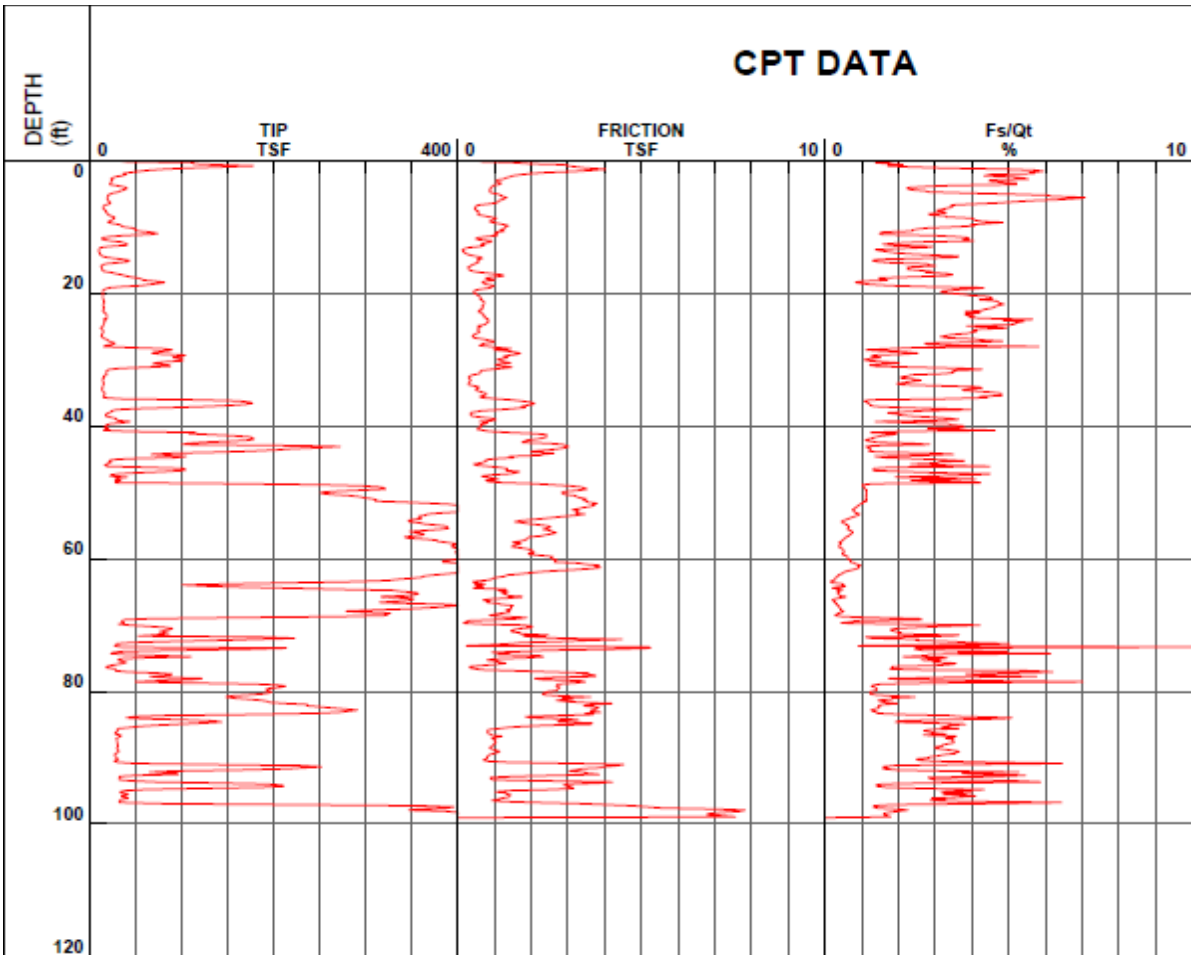
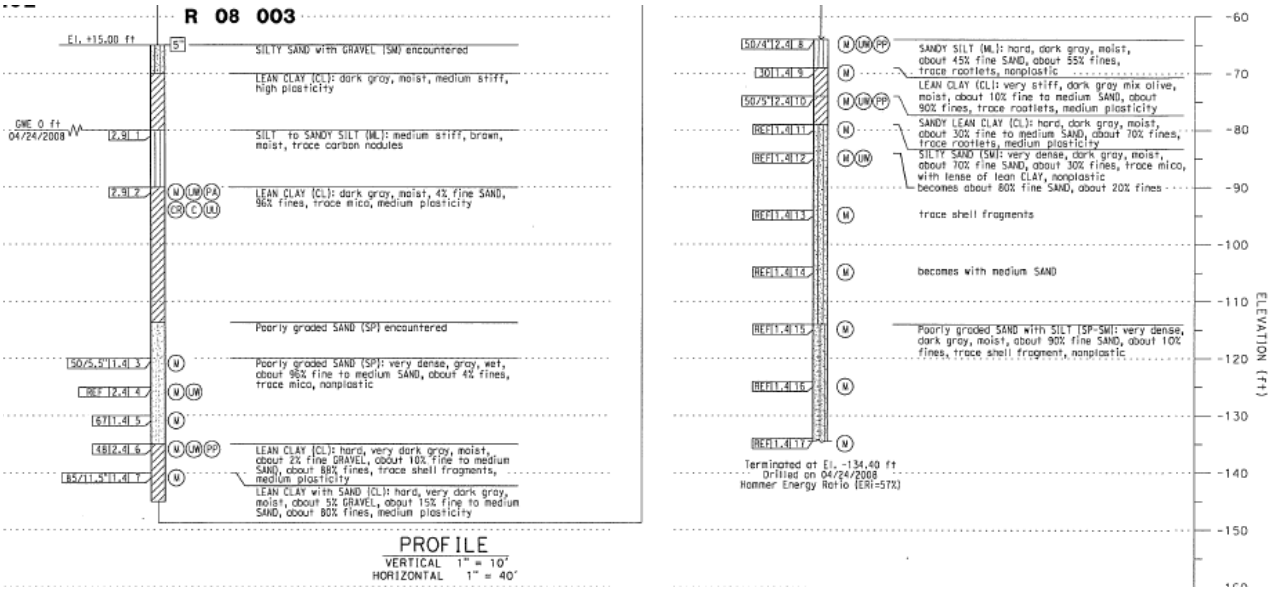


Figure 2: Log of Test Boring (LOTB)



Step 1: Determine Groundwater Elevation

The design groundwater elevation is the level shown on the LOTB for boring R 08 003. There is no reason to adjust the groundwater elevation. GW elevation is 0 feet, 15 feet below the ground surface. (Figure 1)

Step 2: Identify the Soil Layers for Quantitative Liquefaction Analysis.

The CPT 08-152 sounding provides the soil behavior type in increments of 0.16 feet. The following table illustrates some representative layers throughout the depth, where the soil behavior type is evaluated.

Table 1: Liquefaction Evaluation for CPT-08-152

Depth (feet)	Elevation (feet)	Soil Behavior Type	Layer Thickness (feet)	Potential for Liquefaction	Reason for Evaluation
1.15	13.9	Sand	0.16	Not Liquefiable	Above Ground Water Surface
15.09	-0.1	Sand	0.16	Liquefiable	Below Ground Water
15.26	-0.3	Sand	0.16	Liquefiable	Below Ground Water
15.42	-0.4	Sand	0.16	Liquefiable	Below Ground Water
15.58	-0.6	Clay	0.16	Not Liquefiable	Clayey Soil
15.75	-0.8	Clay	0.16	Not Liquefiable	Clayey Soil

The layer at depth of 15.26 feet (elevation of -0.3 feet) shows the potential for liquefaction based on table 1. For this example, this sandy layer will be analyzed.

Step 3: Obtain CPT Data

Obtain cone tip bearing and sleeve friction values from the CPT printout.

Cone Tip bearing: q_c = 40.2 tsf (depth 15.26 feet)

and

Cone Sleeve Friction: f_s = 0.6 tsf (Obtained from the CPT report.)

Step 4: Soil Unit Weight

Determine soil unit weight = 0.06 tcf

Step 5: Calculate/Determine the CPT Soil Behavior Type Index

From Youd et al (2001), equation (14)

I_c: Soil Behavior Index

$I_c = [(3.47 – \log Q)^2 + (1.22 + \text{Log } F)^2]^{0.5} = 2.30$

Where,

Q: Dimensionless normalized CPT penetration resistance

F: Normalized friction ratio

From Youd et al (2001), equation (15)

$Q = [(q_c – \sigma_{v0})/P_a] [(P_a/\sigma'_{vo})^n] = 43.16$

and

From Youd et al (2001), equation (16)

$F = [f_s/q_c – \sigma_{vo})] \times 100\% = 1.48\ \%$

n: Exponent = 1.0 (clay)

σ_{v0}: Total Overburden Pressure = 15.26 x 0.06 = 0.92 tsf

σ'_{vo}: Effective Overburden Pressure = (15.26 -15) x (0.06 - 0.031) + (15 x 0.06) = 0.91 tsf

P_a = Atmospheric Pressure = 1.04 tsf

First calculate Soil Behavior Index (I_c) with n=1. If the calculated I_c is greater than 2.6, the soil behavior is clayey and is not liquefiable. If the calculated I_c is less than 2.6, the soil is most likely granular in nature and Q should be recalculated using an exponent, n = 0.5.

Since Ic < 2.6, Q is recalculated, with n= 0.5

From Youd et al (2001), equation (17)

$Q = [(q_c - \sigma_{v0})/P_a] [(P_a/\sigma'_{vo})^n] = 37.8 \text{ x} 1.07 = 40.48$

n: Exponent = 0.5 (sand)

P_a = Atmospheric Pressure = 1.04 tsf

σ_{v0}: Total Overburden Pressure = 15.26 x 0.06 = 0.92 tsf

σ'_{vo}: Effective Overburden Pressure = (15.26 -15) x (0.06 - 0.031) + (15 x 0.06) = 0.91 tsf

Step 6: Normalize Cone Penetration Resistance

Cone penetration resistance is corrected for overburden stress as follows:

From Youd et al (2001), equation (12)

q_{c1N}: Dimensionless cone penetration resistance corrected for overburden stress

$q_{c1N} = C_Q (q_c/P_a) = 41.42$

Where,

From Youd, equation (13)

$C_Q = (P_a/\sigma'_{vo})^n = 1.07$

C_Q is a normalizing factor for cone penetration resistance.

Step 7: Calculate Clean Sand Equivalent Normalized Cone Penetration Resistance

Correct the normalized penetration resistance, (q_{c1N}), of sands with fines to an equivalent clean sand value, ($q_{c1N)\,cs}$:

From Youd et al (2001), equation (18)

$(q_{c1N})_{cs} = K_c q_{c1N} = 84.16$

Where the CPT correction factor for grain characteristics, K_c, is defined as:

From Youd et al (2001), equation (19a)

For $I_c \leq 1.64$ $K_c = 1.0$

From Youd et al (2001), equation (19b)

For $I_c > 1.64$ $Kc = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75\, I_c - 17.88$

$I_c = 2.3$ and $K_c = 2.03$ (8)

Step 8: Calculate Cyclic Resistance Ratio: CRR

From Youd et al (2001), equation (11b)

If $50 \leq (q_{c1N)\,cs} < 160$ $CRR_{7.5} = 93\,[(q_{c1N)\,cs}/1000]^3 + 0.08 = 0.14$

Step 9: Determine Cyclic Stress Ratio (CSR)

From Youd et al (2001), equation (1)

$CSR = 0.65\, a_{max}(\sigma_o / \sigma'_o) r_d$

Where:

- σ_o and σ'_o are total and effective vertical overburden stresses, respectively.
- a_{max} is peak horizontal acceleration (PGA) in g.
- r_d is a stress reduction coefficient.

For this example:

a_{max} = 0.6g

Determine Stress Reduction Coefficient, r_d.

Depth (z) is 15.26'= 4.65 m

r_d=1.0-0.00765 · z for z≤9.15 m

r_d =1.0-0.00765 · 4.65 = 0.96

Step 10: Calculate the Magnitude Scaling Factor (MSF)

For this example, M_w = Mean Earthquake Moment Magnitude, M = 7.0

MSF = $10^{2.24}/M_w^{2.56}$ = $10^{2.24}/7.0^{2.56}$ = 1.19

Step 11: Calculate the Factor of Safety against Liquefaction

FS = ($CRR_{7.5}$/CSR) x MSF

FS = (.14/.38) x 1.19 = 0.42

Since the FS < 1, the geotechnical report must state that liquefaction is predicted to occur in this layer.

To complete the liquefaction analysis for the site, repeat the above steps for each layer identified as susceptible to liquefaction.

Liquefaction Evaluation Example (SPT Method)

Live Oak Creek Bridge is a proposed single span bridge crossing Live Oak Creek. Preliminary screening, based on the "As-Built" LOTBs for the existing Live Oak Creek Bridge (Br. No. 57-0070), indicate the possibility of liquefaction. A quantitative liquefaction analysis is required following the SPT procedures described in Youd et al, (2001).

The following information is required:

- Seismic design ground motion parameters including peak ground acceleration (PGA) corresponding to a return period of 975-years and the de-aggregated mean earthquake moment magnitude (M). These data are obtained by running the ARS Online webtool using the latitude and longitude 33.31525 N, -117.194225W.
- Soil data from SPT results. Hammer efficiency and soil descriptions are shown on the LOTB dated 11-1-2012 (Attachment 1).
- Groundwater data as shown on the LOTB dated 11-1-2012 (Attachment 1).

Step 1: Determine Groundwater Elevation

The design groundwater elevation is the level shown on the LOTB for Boring RC-11-001. There is no reason to adjust the groundwater elevation.

Step 2: Identify the Soil Layers for Quantitative Liquefaction Analysis.

The LOTB shows two borings with soil data to be evaluated: RC-11-001 and RC-11-002.

RC-11-001 shows five soil layers; preliminary liquefaction evaluation results are presented in Table 1.

Table 1: Preliminary Liquefaction Evaluation for Boring RC-11-001

Layer	Elevation	Soil Type	Thickness	Preliminary Evaluation	Reason for Evaluation
1	210-205	SP	5’	Not Liquefiable	Above groundwater table
2	205-200	SM	5’	Not Liquefiable	Above GW table, fines content > 30%
3	200-194	SP	3’ 3’	Not Liquefiable above 197’ Liquefiable below 197’	Above the groundwater table Granular, <5% fine, SPT N<30, below the GW table
4	194-179	SW	15’	Liquefiable	Granular, <5% fines, SPT N<30, below the GW table.
5	179-150	SP	29’	Liquefiable	Granular, <5% fines, SPT N<30, below the GW table

Most of the soils below groundwater table show the potential for liquefaction based on preliminary qualitative analysis. Only the soil layer 4 at RC-11-001 is quantitively analyzed herein as an example.

Effective unit weights of the soil layers (from Soil Properties Module) are:

- Layer 1: 120 pcf.
- Layer 2: 110 pcf.
- Layer 3: 120 pcf from 200’ to 197’, 57.6 pcf from 197’ to 193’.
- Layer 4: 67.6 pcf.

Step 3: Correct SPT Blow Count Data

From Youd et al (2001), equation (8 and Table 2) the corrected SPT N-value is:

$(N_1)_{60} = N_m\ C_N\ C_E\ C_B\ C_R\ C_S$, where

- $(N_1)_{60}$ = corrected normalized SPT blow count.
- N_m= measured SPT blow count.
- C_N = depth correction factor = $C_N = (P_a / \sigma'_{vo})^{0.5}$ from Youd et al (2001) equation (9)
    - P_a = 1 atm = 2116 psf and σ'vo = effective overburden pressure at the time the SPT was done.
- C_E = hammer energy correction factor (ER_i / 60)
- C_B = borehole diameter correction factor.
- C_R = rod length correction factor
- C_S = correction factor for samplers with or without liner.

For this example:

- N_m= 16 (Measured Blow count at 25' depth)
- $C_N = (2116/2225)^{0.5} = 0.975$
- C_E = 68 / 60 = 1.13
- C_B = 1
- C_R = .95
- C_S = 1.2 (no liner used)

Thus

$(N_1)_{60} = N_m\ C_N\ C_E\ C_B\ C_R\ C_S$

= 16 x 0.975 x 1.13 x 1 x .95 x 1.2

= 20

Step 4: Determine Cyclic Stress Ratio (CSR)

From Youd et al (2001), equation (1), $CSR = 0.65\ a_{max}(\sigma_{vo} / \sigma'_{vo})r_d$

Where:

- σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively.
- a_{max} is peak horizontal acceleration (PGA) in g.

- r_d is a stress reduction coefficient.

For this example:

a_{max} = PGA-=0.4g

Step 4A: Calculate Overburden Stresses.

Use the approximate center elevation of the layer at 185' (depth = 25').

σ'_{v0}=120 x 5 + 110 x 5 + 120 x 3 + 3 x 57.6 + 67.6 x 8 =2225 psf

σ_{v0} = 120 x 5 +110 x 5 + 120 x 6 + 130 x 8 = 2910 psf

Step 4B: Determine Stress Reduction Coefficient, r_d.

Depth (z) is 25'= 7.6m

From Youd et al (2001), equation (2a),

r_d=1.0-0.00765 · z for z≤9.15 m

r_d =1.0-0.00765 · 7.6 = 0.94

Step 4C: Determine CSR

From Youd et al (2001), equation (1)

CSR = 0.65 $a_{max}(\sigma_{vo}/\sigma'_{vo})r_d$

CSR= 0.65 x (0.4) x (2910/2225) x 0.94 = 0.32

Step 5: Fines Content Correction

From Youd et al (2001), equation (5)

$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$, where

- $(N_1)_{60cs}$ is the blow count corrected for fines content
- α and β are coefficients that depend on the fines content.

For this example

- α= 0 (Youd et al, 2001, equation 6a since the sample has < 5% fines)
- β= 1.0 (Youd et al, 2001, equation 7a since the sample has <5% fines)

$(N_1)_{60cs} = \alpha + \beta(N_1)_{60}$

$(N_1)_{60cs}$ = 0 + 1(20)

= 20

Step 6: Calculate Cyclic Resistance Ratio $(CRR)_{7.5}$

From Youd et al (2001), equation (4)

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10(N_1)_{60cs} + 45]^2} - \frac{1}{200}$$

For this example:

$(N_1)_{60cs}$ = 20

$$CRR_{7.5} = \frac{1}{34 - 20} + \frac{20}{135} + \frac{50}{[10(20) \quad + 45]^2} - \frac{1}{200}$$

$CRR_{7.5}$ = 1/14 + 20/135 + $50/245^2$ - 1/200

= 0.071+0.148+0.001 - .005

= 0.22

$CRR_{7.5}$ can alternatively be read directly from Youd et al (2001) Figure 2 replacing $(N_1)_{60}$ with $(N_1)_{60cs}$ and using the liquefaction boundary curve identified as “SPT Clean Sand Base Curve”.

Step 7: Calculate the Magnitude Scaling Factor (MSF)

For this example, M_w =M= 7.6

From Youd et al (2001) equation (24)

$MSF = 10^{2.24}/M_w^{2.56}$

$MSF = 10^{2.24}/7.6^{2.56}$ = 0.97

Step 8: Calculate the High Confining Pressure Correction Factor (K_σ)

From Youd et al (2001) equation (31)

$K_\sigma = (\sigma'_{v0}/P_a)^{(f-1)}$

σ'_{v0}: Effective Overburden Pressure

P_a: Atmospheric Pressure

f: exponent that is a function of site conditions, including relative density, stress history, aging and overconsolidation ratio

From Kulhawy and Mayne (1990) , Equation 2-16:

$D_r^2 = (N_1)_{60} / (C_p.C_A.C_{OCR})$

For normally consolidated (NC) unaged sand:

$C_p = 60 + 25 \log D_{50}$

C_p: Parameter for grain size

D_r: Relative density (%)

D_{50}: Median diameter of sand (mm)

For aged sand:

$C_A = 1.2 + 0.05 \log (t/100)$

C_A: Parameter for aging

t: Time since deposition in years

For overconsolidated (OC) sand:

$C_{OCR} = OCR^{0.18}$

C_{OCR}: Parameter for overconsolidation

OCR: Overconsolidation ratio: σ'_p / σ'_{v0}

Where:

σ'_p: Preconsolidation pressure

σ'_{v0}: Effective overburden pressure

For this example:

Soil is normally consolidated sand and unaged

$C_p = 60 + 25 \log D_{50}$

$D_{50} = 0.5$ mm

$C_p = 60 + 25 \log (0.5) = 52.47$

$C_A = 1$

$C_{OCR} = 1$

$D_r^2 = (N_1)_{60cs} / (C_p.C_A.C_{OCR})$

$(N_1)_{60} = 20$

$D_r^2 = 20/52.47 = 0.36$

$D_r = 60\%$

From Youd et al (2001), figure 15

For $D_r = 60\%$, $f = 0.7$

$K_\sigma = (2225/2116)^{(0.7-1)}$

$= 0.985$

Step 9: Calculate the Sloping Ground Correction Factor (K_α)

For this example:

Level ground**:** $K_\alpha = 1$

α: Static stress ratio

$\alpha = \tau_h / \sigma'_{v0}$

τ_h: Static shear stress on the horizontal plane

σ'_{v0}: Effective overburden pressure

Step 10: Calculate the Factor of Safety against Liquefaction

From Youd et al (2001), equation (30):

$FS = (CRR_{7.5}/CSR) \times MSF \times K_\sigma \times K_\alpha$

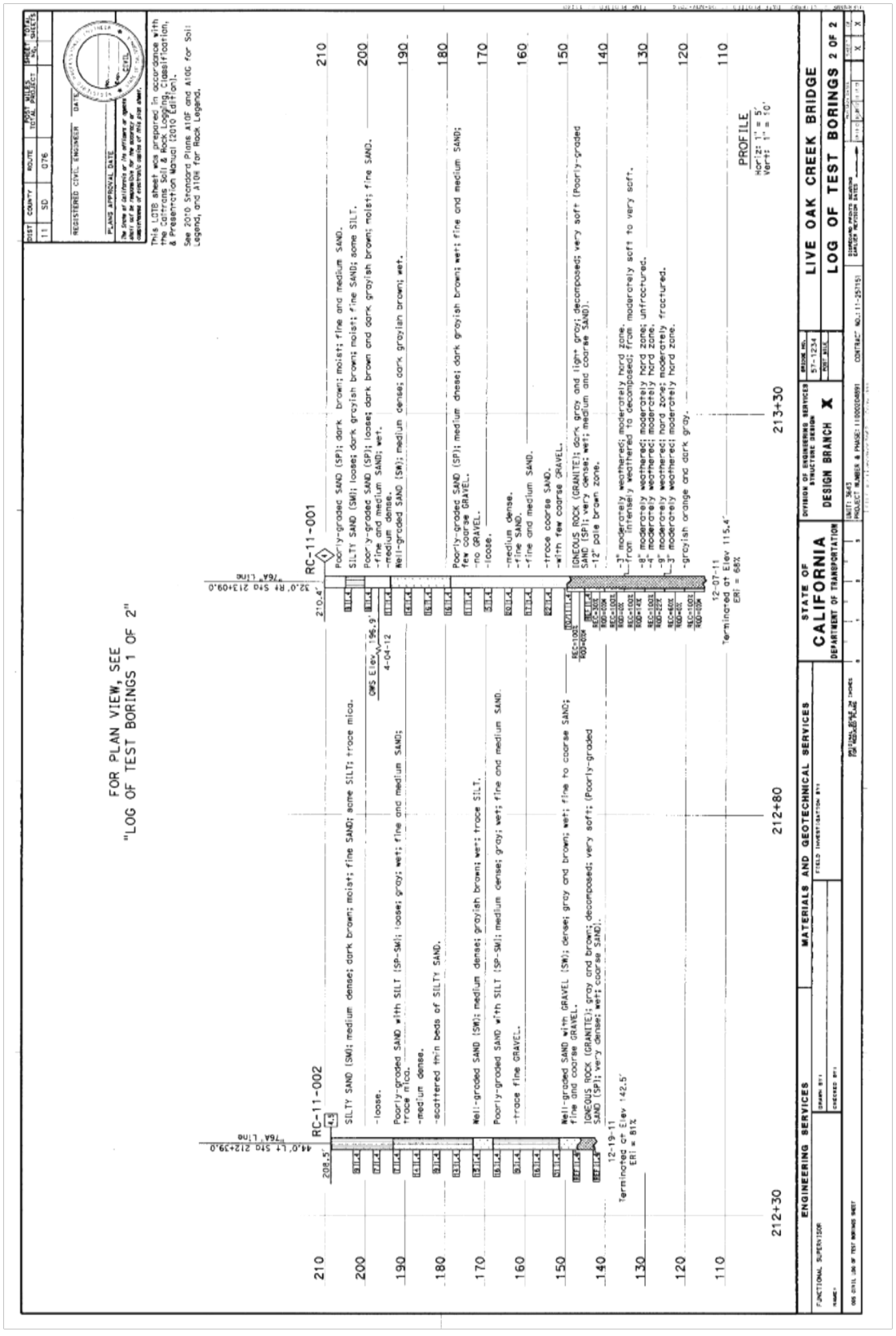
FS = (.22/.32) x .97 x 1.0 x 0.985

=0.66

Since FS is less than 1, the geotechnical report needs to state that liquefaction is predicted to occur in this layer.

To complete the liquefaction analysis for the site, repeat the above steps for all other soil layer identified as potentially liquefiable.

Attachment 1



Fault Rupture

Caltrans bridges crossing active or potentially active faults may be subjected to surface fault rupture and ground displacement during or immediately after an earthquake. Surface fault rupture can cause large deformations in structures depending on the type of fault, total displacement, and angle of incidence of the fault rupture offset relative to the bridge. Memo to Designers (MTD) 20-10 requires a Surface Fault Rupture Displacement Hazard Analysis (SFRDHA) where any portion of the structure is located:

- Within an Alquist–Priolo Earthquake Fault Zone, as defined by the California Geological Survey.

- Within 1,000 feet of an unzoned fault (not located Alquist–Priolo Earthquake Fault Zone) that is Holocene (11,000 years) or younger in age.

Caltrans uses the Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3) model for its fault data base. This model does not separate Holocene aged faults (11,000 years) from Holocene-Latest Pleistocene (active within the last 15,000 years). Accordingly, structures located near faults that are Holocene-Latest Pleistocene age or younger must be evaluated for potential fault rupture hazard.

Evaluation of fault rupture occurs in two steps:

<u>Step 1: Preliminary Screening</u>

- Performed by the project Geoprofessional, preliminary screening consists of literature reviews and evaluation of existing data. If preliminary screening finds that the structure sits within the Alquist-Priolo Earthquake Fault Zone (APEFZ) or less than 1000 feet from an unzoned active fault, then the Geoprofessional must request a Surface Fault Rupture Displacement Hazard Analysis by an experienced engineering geologist. If the structure does not sit within the aforementioned boundaries, then the Geoprofessional must document the preliminary screening findings in an appropriate report.

<u>Step 2: Surface Fault Rupture Displacement Hazard Analysis (SFRDHA)</u>

- Performed by an experienced engineering geologist. May consist of a more in-depth literature review, site reconnaissance, geological mapping, and fault trench excavation to accurately locate and age-date the fault and/or splays with respect to the bridge. In cases where there is a confirmed fault rupture hazard, the SFRDHA will include calculation of the magnitude of anticipated surface displacement.

Preliminary Screening for Fault Rupture Potential

To screen for fault rupture, first determine whether the structure is in or near an earthquake fault zone.

DO NOT use the map presented in the ARS Online tool for preliminary screening for fault rupture. The Caltrans ARS Online tool for seismic response does not return accurate fault to site distances. The faults are simplified into the traces shown for the purpose of estimating ground motion.

If the structure currently being evaluated is located within 1000 feet of an active fault (15,000 years or younger) shown in any of the following references, then the site has the potential for surface fault rupture and a Surface Fault Rupture Displacement Hazard Analysis is required. Consult the following resources to determine if the structure is in or near an earthquake fault zone.

1. Caltrans Geotechnical Archive (GeoDOG)

   Surface Fault Rupture Displacement Hazard Analyses were conducted for over 250 bridges and culverts in California. A list of the bridges is on the Fault Rupture webpage and reports can be found in GeoDOG.

2. CA Geological Survey, Alquist-Priolo Earthquake Fault Zone Maps
   a. Review the Earthquake Fault Zone Map
   b. Review the Fault Evaluation Report(s) including the attached maps

3. USGS Quaternary Fault and Fold Database (Google Earth, KML file)
   a. Review (select) both the "Historic" and "Holocene to Latest Pleistocene"
   b. Evaluate whether the site is within 1000 ft. of the site using the "ruler" tool.
   c. Select the fault trace to obtain the fault name and web link to the fault report, which will provide the fault type, activity, and slip rate data to be used in the SFRDHA.

The site must have a Surface Fault Rupture Displacement Hazard Analysis performed if the preliminary screening determines that the structure:

- Is located within an Alquist–Priolo Earthquake Fault Zone, or
- Is located within 1,000 feet of an unzoned fault (not located Alquist–Priolo Earthquake Fault Zone) that is Holocene/Latest Pleistocene (15,000 years) or younger in age,

If the structure currently being evaluated meets any of the above criteria, contact your Office's designated Engineering Geologist and request that they prepare a *Fault Rupture Report*.

Surface Fault Rupture Displacement Hazard Analysis

A Surface Fault Rupture Displacement Hazard Analysis (SFRDHA), performed by an engineering geologist experienced in fault rupture evaluations, includes the following steps:

Step 1: Hazard Confirmation

- Confirm that the fault rupture hazard exists by repeating some or all of the preliminary screening performed by the Geoprofessional.

Step 2: Surface Fault Rupture Displacement Hazard Analysis

- If a hazard exists, perform the Surface Fault Rupture Displacement Hazard Analysis, which includes determining the magnitude and direction of anticipated ground surface displacement along the fault. The investigation should follow the applicable sections of California Geological Survey Note 49, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, and may include geologic mapping, fault trenching, borings, CPT soundings and/or geophysical studies.
- No single or predetermined combination of methods for the investigation and evaluation of surface fault rupture hazard potential should be considered as being complete due to the variability of site geological conditions and complexity of the faulting.

Examples of the calculations used and types of reports are presented on the Fault Rupture webpage.

Displacement Analysis

Use the appropriate "spreadsheet" analysis tool, located on the Fault Rupture webpage, to estimate the potential displacement. Both a deterministic fault displacement analysis (DFDHA) and a probabilistic fault displacement analysis (PFDHA) of 5% in 50 years probability of exceedance (975 year recurrence interval) must be performed using magnitude, slip rate (for PFDHA), mapping and base map errors, and likelihood of secondary fault traces.

Caltrans bases the design fault rupture displacement on the larger of the deterministic or probabilistic (5% in 50 year) displacement hazard values. For strike-slip and normal faults use the method developed by Wells and Coppersmith (1994), Abrahamson (2008), and Petersen, et al (2011). For reverse-slip faults use the probabilistic fault displacement analysis by Moss and Ross (2011).

If the estimated deterministic displacement is larger than the probabilistic displacement, consult with the Office Earthquake Engineering to determine the site-specific design parameters.

Reporting

Preliminary screening is typically reported in the SPGR, PFR, and FR. Stand-alone reports, if prepared, must be titled *Fault Rupture Hazard Evaluation Report*. If the preliminary screening determines that a fault rupture hazard does not exist then the reporting can be limited to a brief description of the investigation performed followed by the conclusion.

When a SFRHDA has been performed, prepare a *Fault Rupture Hazard Displacement Analysis Report* to document the investigations performed (both literature search and field work), the displacement hazard analysis (both DFDHA and PFDHA), findings, and recommendations. The report should contain the following elements:

- Literature Review: Identify the sources and types of literature, reports, images, maps, and plans used for the detailed evaluation.

- Bridge Review: Bridge name, location, Logs of Test Borings, depth to groundwater, "As-Built" foundation plans, modifications or retrofits, bridge inspection reports, structure type, foundation types, centerline trend, skew of crossing, skew relative to fault being evaluated.

- Fault evaluation: Fault name, verified distance to site, fault type, moment magnitude, recurrence interval, slip rate, amount and direction of potential offset, history of movement, age of fault, and zoning (if any).

- Field Investigations: Summarize site reconnaissance, local geologic mapping, fault trenching, geophysics explorations, subsurface borings, and/or CPT soundings used for the investigation. Append any logs, LOTBs, or reports as necessary to support the findings.

- Fault Rupture Analysis: Summarize the deterministic fault displacement analysis (DFDHA) and a probabilistic fault displacement analysis (PFDHA) of 5% in 50 years. Include tables and graphs presenting the findings of the analysis.

References

1. Abrahamson, N., 2008, Appendix C, Probabilistic Fault Rupture Hazard Analysis, San Francisco PUC, General Seismic Requirements for the Design on New Facilities and Upgrade of Existing Facilities.

2. Bryant, William A., Hart, Earl W., 2007, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act With Index to Earthquake Fault Zones Maps, Special Publication 42.

3. California Geological Survey, 2002, *Guidelines for Evaluating the Hazard of Surface Fault Rupture*, CGS Note 49.

4. Caltrans, Memo to Designers 20-10, *Fault Rupture.*

5. Moss, R. and, Ross, Z., 2011, Probabilistic Fault Displacement Hazard Analysis for Reverse Faults, BSSA Vol. 101, pp 1542 – 1553.

6. Petersen, M., and others, 2011, Fault displacement hazard for strike-slip faults: Bulletin of the Seismological Society of America, Vol. 101, No. 2, pp. 805–825.

7. US Geological Survey, Faults in Google Earth, 2013.

8. US Geological Survey, 2008, National Seismic Hazard Maps and Data, Lower 48 Maps and Data.

9. Wells, D., and Coppersmith, K., 1994, New empirical relationships among magnitude, rupture length, rupture width, rupture area, and surface displacement: Bulletin of the Seismological Society of America, Vol. 84, No. 4, pp. 974-1002.

Liquefaction-Induced Lateral Spreading

Earthquake-induced ground deformations within the depth of engineering significance are related to either fault surface rupture or ground motion. In geotechnical engineering applications, ground deformations are usually reported in terms of vertical and horizontal components, which are commonly referred to as *settlements* and *lateral deformations,* respectively. The terms *lateral deformation, lateral displacement* and *lateral movement* are often used interchangeably without any distinction. Earthquake-induced ground deformations can occur in both liquefied and non-liquefied soils. However, relatively large seismic lateral ground movements that can cause collapse or failure of highway facilities, including bridges, slopes and earth retaining structures (ERS) are most often associated with soil liquefaction. The scope of this module is limited to this type of relatively large seismic lateral ground deformation, a phenomenon commonly referred to as "*liquefaction-induced lateral spreading.*"

Unless specified otherwise in a Project-Specific Seismic Design Criteria (PSDC), this module is applicable to bridges designed or required to be retrofitted in accordance with the Caltrans Seismic Design Criteria (SDC), and other appurtenant highway facilities including ERS, slopes and embankments.

Background

The phenomenon of "*liquefaction-induced lateral spreading"* during earthquakes has been defined and interpreted variably in the research literature as well as in practice.

Liquefaction-induced lateral spreading was identified and documented as one of the main causes of damage to highway facilities during strong earthquakes in the late 19th century but did not receive much attention until the late 20th century. However, significant progress has since been made on the subject of predicting the occurrence of soil liquefaction at project sites during future earthquakes. The fundamental soil mechanics principles and other important factors associated with soil liquefaction during earthquakes has since been identified and extensively studied, and prediction methods developed that are now widely used in practice.

Liquefied soils can experience a significant reduction in shear strength, which in turn can cause a drastic reduction in the static soil lateral resistances available to support a sloped or mechanically restrained soil masses (e.g., bridge abutment, retaining wall etc.) that are subjected to sustained destabilizing static shear stresses. As a result, such soil masses often experience large permanent lateral movements (or spreading) during or immediately after earthquake-induced ground shaking. California is one of the highly active seismic regions of the world, and liquefaction-induced lateral spreading of grounds poses a serious threat to bridges and other transportation facilities.

Lateral spreading hazards, particularly at bridge support locations, have been intensely researched during recent decades. The subject, however, has proven to be difficult,

specifically for sites with mechanically restrained ground due to the complex seismic soil-foundation-structure interaction mechanisms involved.

Caltrans has been considering lateral spreading hazards at project sites based on the rapidly evolving state of the art knowledge. Recently, the Department published MTD 20-15 documenting a recommended lateral spreading hazard analysis procedure for deep-foundation supported new and existing bridges. The recommended procedure involves both geotechnical and structure analyses in an iterative manner and requires common understanding and timely communications between the project Geotechnical Designer (GD) and the Structure Designer (SD).

This module presents the underlying basic geotechnical concepts to facilitate a thorough and uniform understanding among the geo-professionals of the fundamental mechanisms involved with liquefaction-induced lateral spreading, and of related Caltrans' current analysis and design practices. The state-of-knowledge as well as the state-of-practice regarding lateral spreading is rapidly evolving. As such, this module will be updated periodically as new information becomes available.

The geotechnical scope of work associated with a liquefaction-induced lateral spreading hazard analysis for a project involves the evaluation of site-specific soil liquefaction hazards, residual shear strength of liquefied soils, ground lateral or slope stability and permanent lateral ground displacements. Several analysis procedures are available for the evaluation of earthquake-induced soil liquefaction hazards at a project site. For Caltrans projects, soil liquefaction is evaluated in accordance with the procedure in the "*Soil Liquefaction Evaluation"* module of the Geotechnical Manual (GM).

At this time, the relevant ground motion parameters for both liquefaction evaluation and lateral spreading analysis, include the Horizontal Peak Ground Accelerations (PGA) and the moment magnitude (M) of the associated earthquake event. For Safety Evaluation Earthquake (SEE), as defined in SDC, the design horizontal PGA corresponds to a return period of 975-years. For clarity, the design Horizontal PGA (in g) is referred to herein as the design HPGA (in g), where g is the acceleration of gravity. The design earthquake magnitude (M) is taken as equal to the moment magnitude of the deaggregated mean earthquake for the design HPGA. Other aspects of a liquefaction-induced lateral spreading analysis are discussed in the following sections.

Liquefaction-Induced Lateral Spreading

In general, soil masses under the influence of static driving shear stresses, such as sloping grounds, ERS and bridge abutments, are susceptible to liquefaction-induced lateral spreading during earthquakes.

The seismic phenomenon of small (1.0 inch) to very large (>30 ft) permanent lateral ground deformation involving liquefied soils, termed as the *liquefaction-induced lateral spreading,* can be divided into three broad categories:

- Moderate to steeply sloping or mechanically retained soil masses, including embankment slopes, ERS and bridge abutments, are susceptible to flow type landslides when the available reduced lateral sliding resisting capacities (or lateral nominal sliding resistances) due to soil liquefaction falls below the destabilizing total lateral static driving forces. In such cases, the soil mass experiences a large lateral displacement or movement. The magnitude of such large lateral movement is unpredictable. For all practical purposes, such soil mass can be considered to have experienced unlimited lateral movement, and thus have failed or collapsed. This type of ground failure is termed as a *flow failure*, which usually occurs after the cessation of the ground shaking. Once initiated, the magnitude of the lateral movement that such a soil mass may experience is independent of the ground motion parameters.

- In the case where the reduced residual lateral resisting capacity remains greater than the total destabilizing static lateral forces, the soil mass experiences, if any, limited lateral ground deformations or movements during ground shaking. Such lateral movement occurs in an incremental manner during only those seismic inertial loading cycles when the total (static +seismic) destabilizing lateral load exceeds momentarily the available reduced total lateral resistance. The soil mass stop moving as soon as the total lateral load falls below the available total resistance due to reduction in the inertial lateral load. The soil mass may experience large but limited lateral displacement before coming to a full stop at or before the cessation of the ground shaking. In liquefied soils, this type of lateral ground displacement or lateral spreading occurs during ground shaking under the combined effects of the reduced shear strength and the lateral inertial forces.

- During past major earthquakes, relatively thin surficial blocks of mildly sloping ground and flat ground with free boundary surface, mostly near water bodies with shallow groundwater, have experienced liquefaction-induced small to very large downslope ground movements. Little or no static driving shear stresses exist in these type of soil masses. Lateral ground deformations occurred mainly due to the accumulation of ground motion-induced cyclic shear strains within a shallow or surficial liquefied soil layer.

  Non-liquefied soil crust layers overlying the liquefied layer, where present, may experience additional compressional deformation in the downslope direction due to self-weight and inertial forces. The overlying soil crust may simply ride downslope on its contact with the underlying liquefied soil layer. In this case, the underlying liquefied soil layer acts essentially as a base isolation. This type of permanent lateral ground displacement or lateral spreading can occur both during and immediately after the cessation of the ground motion.

Shear Strength of Liquefied Soils

Stress-Strain Behavior During Undrained Monotonic Loading

Saturated, loose to medium dense cohesionless soils with little or no plastic fines develop positive excess pore water pressure (Δu) due to the cyclic shear stresses induced by earthquake shaking. These soils tend to contract when sheared but are unable to do so during seismically induced shear loading. This is because the permeability of these soils, compared with the very fast rate of loading, is inadequate to allow drainage to occur during the load application. This in turn results in the development of positive excess pore pressure within these soils during ground shaking. That is, such cohesionless soils may be free draining under normal conditions but behave as undrained (similar to cohesive soil) soils during earthquakes.

The positive excess pore pressure reduces the effective confining stresses in the soils causing significant reductions in the soil shear strengths and stiffnesses during ground shaking. In some cases, fully reduced shear strength and stiffness conditions continue to prevail until sometime well after the cessation of the seismically-induced ground shaking when the generated excess pore pressure starts to dissipate.

Figure 1 depicts schematically, based on original research work performed by Castro (1969), the undrained response of saturated cohesionless soils when sheared monotonically. In these cases, the rate of loading is significantly slow compared to earthquake-induced loading. However, soil contraction or drainage of the porewater from these soils is artificially prevented so that shearing occurs under undrained conditions. The effects of the shear loading, despite slower rate but being undrained, would be expected to be like those during fast seismically-induced shear loading. Understanding these fundamental aspects of the undrained shear behavior of saturated cohesionless soils is a basic prerequisite for understanding soil liquefaction during earthquakes and its potential impacts on the deformation and stability of grounds as well as the facilities constructed of, within or on these types of soils.

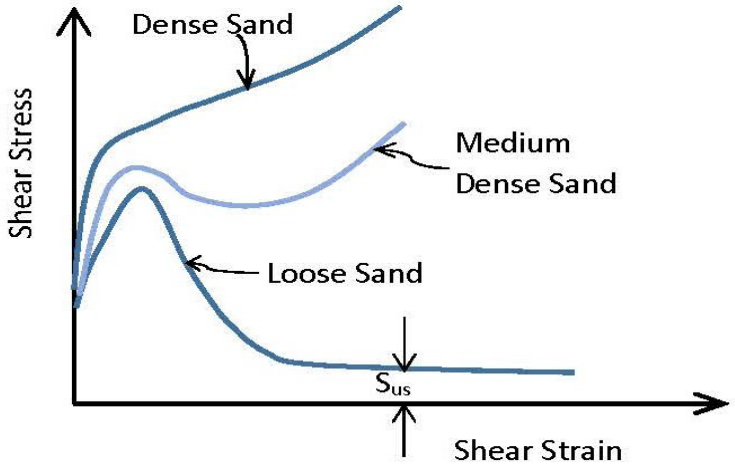


Figure 1: Undrained Shear Stress-Strain Behavior of Saturated Cohesionless Soils Under Monotonic Loading (Schematic)

Point A in Figure 1 represents the *initial states* of the three (3) saturated sand soil samples sheared monotonically under undrained conditions. For the loose sand sample, as the applied shear stress, over the sustained static shear stress (τ_s), increases, the excess pore pressure also increases. The shear stress-strain curve quickly reaches a peak, which is equal to the *peak undrained shear strength* $(S_u)_{peak}$ of the soil. After this point the applied shear stress that the soil can sustain starts to decrease rapidly until it drops to a minimum value (S_{us}) at some large strain. At this point, the loose sand continues to experience shear deformation at a constant rate without any increase or decreases in the applied shear stresses. This state of soil, defined in term of the soil density (which is the same as the initial density) and the operating effective confining stress, is termed as the steady state (Poulos and Castro, 1974).

When the undrained shear strength, after attaining the peak value, drops to the minimum value, the loose soil is said to have completely liquefied since beyond this point the undrained soil shears like a liquid. The available minimum undrained unit shear resistance for a completely liquefied soil is termed as the *steady state shear strength* (S_{us}) of the soil (Poulos and Castro, 1974) (Figure 1).

At the instant the peak undrained shear strength is exceeded, the loose sand is said to have experienced initial liquefaction or liquefaction triggering. In completely liquefied soils, the excess pore water pressure continues to rise during further shearing until the undrained shear strength drops to the steady state shear strength, which remains the same during further undrained shearing.

The saturated, medium dense sand, (Figure 1) initially behaves in a manner very similar to the loose sand. The medium dense sand experiences initial liquefaction and then shear resistance drops to a minimum value. However, the medium dense sand experiences less reduction in the available undrained shear strength after reaching the initial peak strength (i.e., initial liquefaction). Furthermore, the minimum undrained shear strength for the medium dense sand is reached at a lower shear strain than the loose sand.

Most significantly, soon after reaching the minimum shear strength condition, the undrained shear resistance of the medium dense sand starts to increase again. This occurs because after experiencing the minimum shear strength condition, the state (the combination of density and stresses or strains) of this medium dense sand has transitioned from a tendency to contract to a tendency to dilate, which in turn results in the reduction of the pores water pressure. From this point on, due to continued tendency of the soil to dilate as the sample is further sheared, the pore water pressure continues to decrease, which in turn results in a continued increase in the undrained shear resistance of the soil.

In Figure 1, both the loose and the medium sand have experienced initial liquefaction. However, the medium dense sand regained its pre-initial liquefaction peak shear strength after a small amount of post-initial liquefaction shear deformation. Upon further shearing, the undrained shear strength of the medium dense continues to increase above the initial peak undrained shear strength. Whereas the undrained shear strength of the loose sand not only drops to a much lower minimum undrained shear strength and at a much larger

shear deformation, further shearing does not result in an increase in the undrained shear strength. The loose sand sample continues to shear at constant effective confining stress, that is, the undrained shear strength remains the same.

As soon as the available undrained shear strength, after reaching the peak, dropped below the sustained static shear stress (τ_c), the loose sand essentially collapsed, and experienced uncontrolled viscous fluid flow causing large shear deformation. This type of undrained shear behavior experienced by saturated loose sand is termed as "*Flow Liquefaction."* The undrained shear behavior exhibited by the medium dense sand is termed as "*Limited Liquefaction.*"

On the other hand, as shown in Figure 1, the undrained stress-strain response of the saturated dense cohesionless soil sample is characterized by a stiff curve with no reduction in the available undrained shear resistance during shearing. Such soils are thus considered not susceptible to liquefaction or significant lateral deformations during earthquakes.

Note that the undrained shear stress-strain response of a given saturated cohesionless soil depends on its initial state defined in terms of the initial density and initial effective confining stress. Depending on the combination of these two major controlling parameters or the initial state, the undrained stress-strain responses will transition gradually from one type of behavior to the others (Figure 1). No simple criteria exist that can be used to distinctly delineate the boundaries between the three typical types of undrained behavior of saturated cohesionless soils depicted in Figure 1.

It can be seen from the above that the initial density (or the initial relative) alone can not be used to predict the undrained stress-strain behavior or liquefaction susceptibility of a cohesionless soil. It the initial state, i.e., the combination of the initial density (or initial relative density) and the initial effective confining stress, of a given saturated cohesionless (or sand-like) soil that controls its undrained shear behavior or liquefaction susceptibility during earthquakes. The amount of lateral deformations that a liquefied soils may experience also depends on additional factors, including the destabilizing sustained or initial static shear stress or topography, drainage and boundary conditions (e.g., free face versus laterally supported earth mass) as well as the magnitude, frequency content, duration and other characteristics of the earthquake induced base ground motion as well as the ground motion experienced by the laterally spreading soil mass.

Case History of Excess Pore Pressure Generation in Liquefied Soil

Figure 2 shows two acceleration-time histories recorded in the field during an actual earthquake event (1987 Superstition Hills Earthquake of M_w=6.6). The recording station was located on a relatively flat soil site. The soil profile at the recording station (known as the Wildlife Site) consists of about a 2.5 m (8.0 feet) thick upper layer of silt to clayey silt soils underlain by a 4.3 m (≅ 14 feet) thick silty sand to sandy silt layer, which in turn is underlain by a 4.7 m (15.5 feet) thick clay layer. The groundwater table at the time of the earthquake was 1.5 m below the ground surface.

The 4.3 m thick silty sand to sandy silt layer (between 2.5 to 6.8 m below the ground surface) liquefied during the recorded seismic event.

Of the two acceleration-time histories recorded at the site (Figure 2a), one was at the surface (depth =0) and the second was at 7.5 m below the ground surface. The excess porewater pressures (Figure 12b) in terms of the ratio r_u, were recorded at four different depths within the 4.0 m thick liquefied soil layer. The symbols P5, P2, P1 and P3 represent the piezometers located at depths of 2.9, 3.0, 5.0 and 6.6 m, respectively.

Zeghal and Elgamal (1994) divided the acceleration-time and the excess porewater pressure ratio-time histories (Figure 2) into four stages in term of time starting from the onset of ground shaking.

During Stage 1 (0.0 to 13.7 sec), ground acceleration amplitude was low and the excess porewater pressures at all depths within the liquefied soil layer increased slowly a small amount.

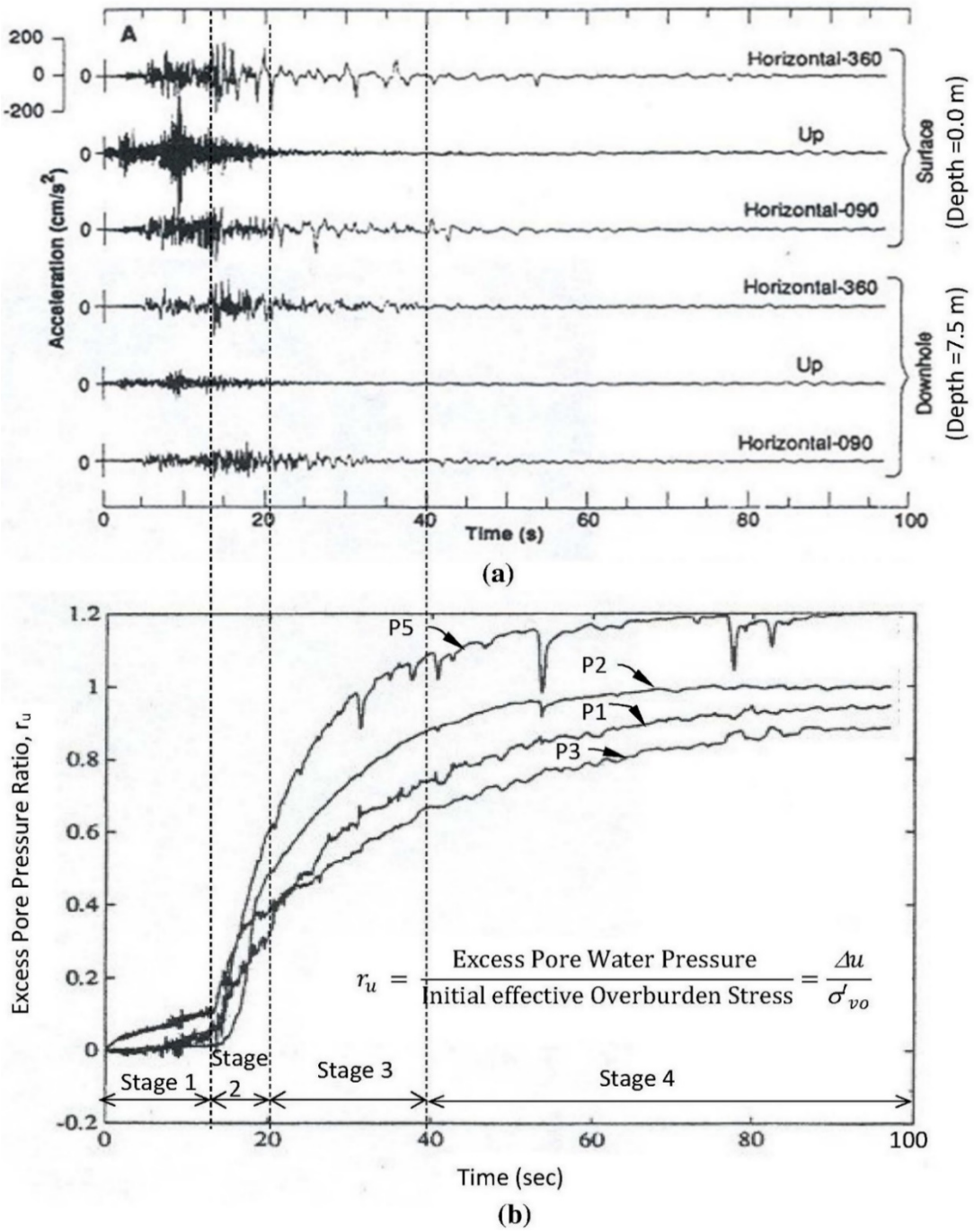


Figure 2: Results of Field Monitoring of the Wildlife Array Instruments During the 1987 Superstition Hills Earthquake: (a) Recorded Traces of Accelerations and Pores Pressures; (b) Recorded Pore Pressure as a Ratio of the Estimated Initial Effective Overburden Stress Versus Time (Modified after Holzer et al., 1989, Dobry et al., 1989 and Youd and Carter, 2005)

During Stage 2 (13.7 to 20.6 sec) the ground motion was intense during which time the PGA values of 0.17g and 0.21 g, occurred at 7.5 m and 0.0 m depths, respectively. During this shaking period, excess pore pressure increased rapidly.

During Stage 3 (20.6 to 40.0 sec), ground motion intensity was lower with the maximum acceleration of about 0.06g. The excess pore-pressure, however, continued to increase rapidly. At the end of the 3rd Stage, the excess pore pressures reached relatively high magnitudes.

During Stage 4 (40.0 to 96 sec) the ground motion intensity was low. The excess pore pressure continues to rise, albeit at a much slower rate, until the end of shaking.

The ground motion records in Figure 2(a) show that for all practical purposes the earthquake-induced input ground motion at the base of the liquefied soil layer (at depth 7.5 m) ended about 40 to 50 seconds after the onset of the ground motion. Note, however, that the excess pore pressure (Figure 2b) continues to increase (or at least did not start to drop or dissipate) for sometimes well after 40 to 50 seconds or the cessation of ground motion at the non-liquefiable base layer (at 7.5 m depth).

The characteristics of ground motion records at the surface (Figure 2a) and the excess pore pressure records (Figure 2b) indicate that the 4.3 m thick silty sand and sand silty layer, sandwiched between two non-liquefiable soil layers, experienced initial liquefaction (or triggering) at about the time the average excess pore pressure ratio of about 0.45 to 0.5 for the entire layer.

Residual Shear Strength of Liquefied Soils

The analysis procedure in the “Liquefaction Evaluation” module of the Geotechnical Manual predicts whether potentially liquefiable soils at a project site will experience initial liquefaction due to the specified design HPGA (and the design M) at the ground surface obtained by assuming no liquefaction.

In liquefied soils that are susceptible to lateral spreading displacements of magnitudes significant for the SEE seismic design, i.e., to cause seismic failure or collapse of ground or structures, the excess pore pressure during the seismic event is likely to continue to increase (positive) past initial liquefaction until the excess pore pressure ratio (r_u) reaches a high value close or equal to 1.0. The excess pore pressure ratio, r_u, at any soil depth is defined as the ratio of the excess pore pressure (Δu) to the initial effective overburden stress (σ'_{vo}) (Figure 2). These types of soils are highly susceptible to liquefaction, as would generally be predicted as such by the relatively low value of the corrected and normalized clean-sand equivalent SPT blow count $(N_1)_{60\text{-}cs}$ as defined in Youd et al (2001) or the normalized clean-sand equivalent CPT cone tip resistance, ($Q_{tn,\,cs}$) as defined in Robertson (2009). Note that Q_{tn} = Q as defined in Youd et al (2001), and $Q_{tn.cs} = K_c\, Q_{tn}$. Here K_c, the correction factor for grain characteristics (combined influence of fines content, mineralogy, and plasticity) and $(N_1)_{60\text{-}cs}$ are as defined in Youd et al (2001).

Studies of a large numbers of case-histories (Bartlett et al, 1995, Kramer and Wang, 2015, Robertson, 2009) also indicated that almost all cases of liquefaction-induced lateral spreading involving significant lateral ground displacements, including flow failures, have been limited to liquefied soils with $(N_1)_{60\text{-}cs} \leq 15$ or $Q_{tn,cs} \leq 70$.

Therefore, it can be assumed that in liquefied soils that are likely to experience significant liquefaction-induced lateral spreading displacements, the excess pore water pressure either continues to rise slowly or remains at about the same, after reaching a peak r_u value equal or close to 1.0 during ground shaking, until the cessation of ground shaking.

The contractiveness of a cohesionless soil deceases with increase in $(N_1)_{60\text{-}cs}$, which represent the equivalent *initial state* of a clean sand. As $(N_1)_{60\text{-}cs}$, increases, for example to slightly above 15, the liquefaction behavior of a clean sand transitions from complete or flow type to near-complete or somewhat limited liquefaction. The excess pore pressure in such soil drops, if any, only slightly due to dilation after reaching a significantly high peak r_u value (i.e., close to 1.0) during the ground shaking period. These types of liquefied soils may experience relatively large but limited lateral spreading.

As $(N_1)_{60\text{-}cs}$ further increases, the contractiveness and the peak r_u value of a granular soil continue to decrease. A clean sand that is only somewhat to slightly contractive, for example when $(N_1)_{60\text{-}cs}$ is in the range of 20-25 may experience even smaller peak R_u value but sufficiently high for the initial liquefaction to occur during ground shaking. However, the initial liquefaction of such soils will be immediately followed by a significant drop in the excess pore pressure due to its transition from the contractive to dilative state. These types of soils even when identified as liquefiable based on the prediction of initial liquefaction (or liquefaction triggering) are not susceptible to significant lateral spreading displacements.

In general, any reduction in the excess pore pressure either due to dilation or drainage after reaching the peak during shaking will result in less deformation than if there were no such reduction during shaking. Thus, for lateral spreading analysis it is conservative to assume that liquefied soils experience no pore pressure reduction after reaching the peak and until after the cessation of ground shaking or inertial loading.

In summary, it can be assumed that fully liquefied soils are susceptible to significant lateral spreading displacements, including flow failures.

Like the loose sand sample (Figure 1), field soils that experience full or complete liquefaction during earthquake shaking retain a certain minimum amount of undrained shear resistance even after experiencing very large shear displacements. For the ideal field case, this minimum shear strength should be the same as the steady state shear strength (S_{us}) for the same soil. In practice, the ideal laboratory testing conditions may not occur, and it is very difficult to evaluate the steady state shear strengths in both the laboratory and the field. As a result, significant research has been conducted to estimate the average minimum shear strengths that were operating in the field when lateral spreading occurred during past earthquakes. The minimum shear strength thus evaluated is termed as the undrained residual shear strength (S_r) of the liquefied soils.

Based on these efforts, several correlations have been proposed between S_r and the corrected and normalized field measured SPT blow count $N1_{60}$ (and CPT tip resistance) with or without fines-corrections.

For a given soil the magnitude of S_r, like S_{us}, depends only on the "initial state", defined in terms of the initial density and initial effective overburden stress. For projects, the empirical correlation proposed by Kramer and Wang (2015), is recommended in MTD 20-15 to evaluate S_r of the soils that are predicted to liquefy and are susceptible to liquefaction-induced lateral spread hazards when subjected to the design ground motions:

$$S_r(\text{psf}) = 2116 \exp\left\{-8.444 + 0.109N_{1_{60}} + 5.379\left(\frac{\sigma'_{vo}}{2116}\right)^{0.1}\right\} \quad (1)$$

Where,

$N1_{60}$ = Energy corrected and overburden pressure normalized SPT blow count at the depth under consideration. The field measured SPT blow count (N_m) at any depth is corrected to standard 60% hammer engery and normalized to an initial effective overburden stress equal to one (1.0) atmospheric pressure (≅ 2116 psf). See "Liquefaction Module" of the Geotechncial Manual for a detailed calculation procedure for $N1_{60}$.

σ'_{vo} = Initial in-situ effective overburden stress at the depth under consideration evaluated based on the ground surface and groundwater elevations applicable to seismic design (Extreme Event Limit State I).

In the above correlation the normalized SPT blow count $N1_{60}$, represents a measure of the in-situ initial state of the soil. Note that, unlike for the liquefaction hazard evaluation, $N1_{60}$ in the above correlation is ***not*** corrected for fines content.

Design Ground Motion Parameters

Unless specified or required to be specified in a Project-Specific Seismic Design Criteria (PSDC), current seismic design procedure for bridges consists of a Safety Evaluation Earthquake (SEE), as defined in the Seismic Design Criteria (SDC).

Unless a site-specific time-history type analysis is performed, the SEE design ground motions evaluated as per SDC is in the form of a single uniform "Design Spectrum" representing the design horizontal ground motions at the surface of the site corresponding to a return period of 975-years (or 5 percent probability of exceedance in 50 years). MTD 20-1 specifies the acceptable minimum performance criteria for SEE seismic design as "significant damage" and "no collapse" due to the "Design Earthquake".

Currently, for SEE seismic design, the following design ground motion parameters are generally necessary for the analysis and evaluation of liquefaction-induced lateral spreading. As stated earlier, these parameters are determined in accordance with the procedure specified in Appendix B of the SDC.

1) Design HPGA corresponding to a return period of 975-years, and
2) Design earthquake magnitude (M) which is taken as equal to moment magnitude of the deaggregated mean earthquake for the design HPGA, and
3) The deaggregated mean site-to to source distance (R_{rup}) for design HPGA

Hereafter, in this document the moment magnitude of the design earthquake (M) will be referred to as the "Design M". Note that at this time, the design HPGA for lateral spreading analysis is taken as equal to the design spectral acceleration (S_a) at structure period, T=0.0 sec.

Seismic Lateral Stability and Deformation Analysis

For SEE design, steep slopes and earth-retaining system (ERS) with or without mechanical restraining elements (e.g., piles, ground anchors etc.) are usually analysed for seismic lateral instability, including lateral defomations or displacements. Soil liquefaction-induced lateral spreading and other ground distress mechanisms of ground, particularly those involving mechanical lateral resisting elements where soil-foundation-structure interactions play a signficant role, are complex phenomena and the subjects of intense on-going research.

To date, the most pertinent and reliable analysis methodology that can be easily used in the design of common projects has been developed based on work involving unrestained sloping ground that is not affected by soil liquefaction. This methodology involves: (1) A seismic Lateral Stability Analysis. A pseudo-static slope stablity analysis is usually performed to evaluate lateral stability during an earthquake, and when necessary, (2) A lateral displacement analysis as per Newmark Displacement Analysis Method (NDAM). NDSAM provides an estimate of the lateral displacement that occurs during ground shaking.

A brief description of the basic elements of this analysis methodology is presented first in the following section since it provides the necessary background for the methodology included in the MTD 20-15 (2017) for the analysis of liquefaction-induced lateral spreading hazards for new and existing bridges. With regard to the restrained ground or ERS, it should be noted that seismic lateral stability and lateral movements discussed in this module pertain only to the global or overall slope stablity type lateral sliding mechanisms. Lateral sliding hazards along the base of the shallow foundations, where involved, are not covered in this module. However, many of the concepts discussed herein are equally applicable to such cases.

Seismic Lateral Stability Analysis

Seismic Coefficient

Seismic lateral sliding stablity of non-liquefied, moderately to steeply sloping ground and the overall (or global) stability of ERS, including those mechnically restrained by structural elements (e.g., piles, ground anchors, etc.), during earthquake-induced ground shaking events are usually evaluated using a limit-equlibrium based pseudo-static slope stability analysis. Figure 3 presents a typical schematic model to illustrate the basic concepts involved with the limit-equlibrium based pseudo-static slope stability analysis of a slope.

Despite significant limitations, the most commonly practiced analysis procedures to evaluate the first two categories of lateral spreading hazards discussed in the *Liquefaction-Induced Lateral Spreading* section, including the Newmark' rigid body displacement analysis in MTD 20-15, make use of the limit-equlibrium based pseudo-static slope stability analysis.

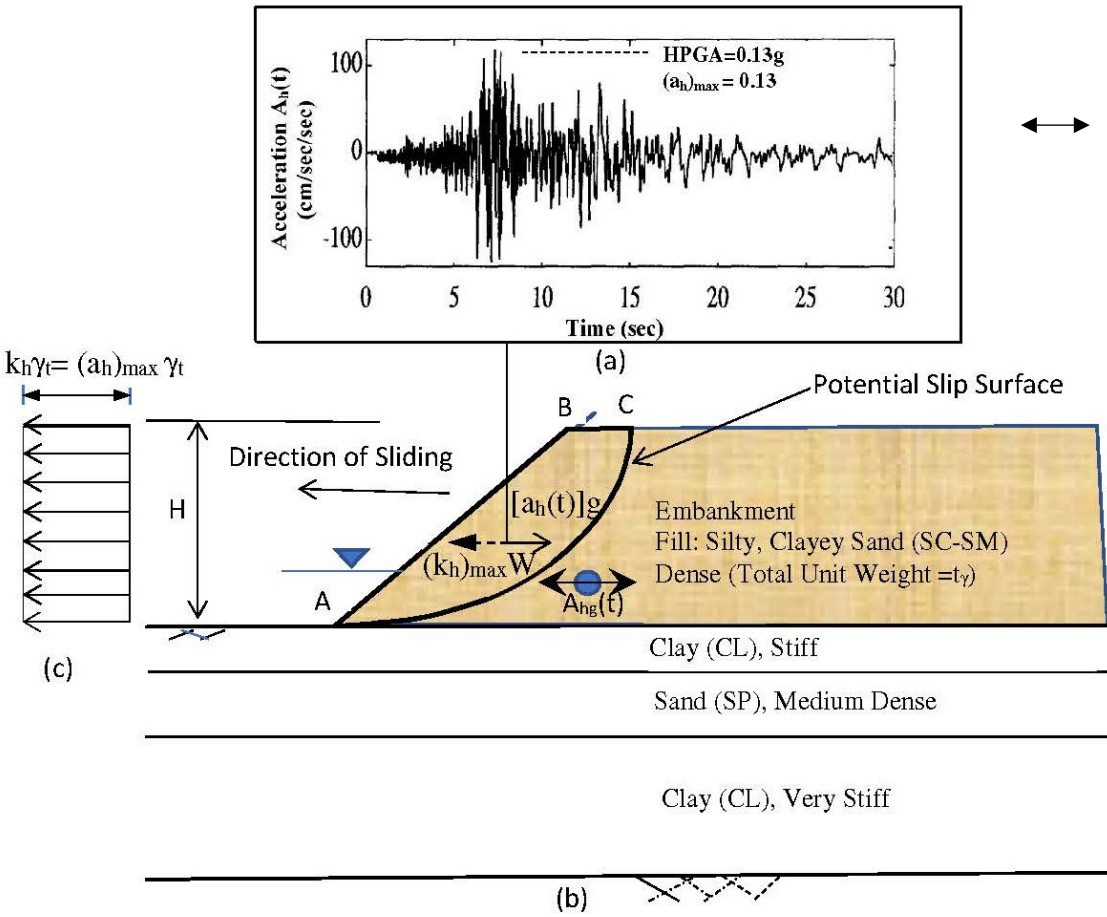
A pseudo-static slope stablity analysis is a force-based seismic lateral sliding stability analysis utilizing the same limit-equlibirum concept as the one used in static slope stablity analyses.

In a pseudo-static lateral sliding stability analysis, in addition to the applicable sustained or static forces, an additional destablizing dynamic lateral force is applied at the centroid of the mass being analysed for stablity. This additional dynamic force, $F_h(t)$, is due to the inertia associated with mass (m) or weight (W) at time (t) since the onset of ground shaking can be evaluated as follows:

$$F_h(t) = a_h(t) \times W \qquad (2)$$

Where, $$a_h(t) = \frac{A_h(t))}{g} \qquad (3)$$

$A_h(t)$ = Magnitdue of the horizontal ground acceleration at time (t) acting on the soil mass being analzyed for stablity acting in the direction opposite to the direction of the force $F_h(t)$.



Schematic Model for Pseudo-Static Slope Stability Analysis of a Slope; (b) Example of a Horizontal Acceleration-Time History Measured at the Ground Surface (Horizontal Peak Ground Acceleration (HPGA)=0.13g; and (c) Sustained Horizontal Ground Acceleration. $(k_h)g = (a_h)_{max}\,g$ for Pseudo-Static Lateral Stability Evaluation.

W = Total weight of the potential sliding mass

g = Acceleration due to gravity, and

t = Time starting from the onset of ground-shaking

The parameters $a_h(t)$ is the coefficient of horizontal acceleration $A_h(t)$ at time (t). Note that units for the various parameters defined above must be consistent with the unit used for the acceleration due to gravity, g.

Based on Equation 3, the maximum value of the coefficient $a_h(t)$ for a horizontal acceleration-time history with a peak acccleration of $(A_h)_{peak}$ is given by:

$$(a_h)_{max} = \frac{(A_h)_{peak}}{g}$$

As an example, for the ground motion shown in Figure 3(a), which acts at the centriod of the mass of the soil for which sliding stability along the slip surface shown is being analyzed:

$$(a_h)_{max} = \frac{(A_h)_{peak}}{g} = \frac{128\ cm/sec/sec}{980\ cm/sec/sec} = 0.13$$

In a limit-equilibrium based pseudo-static slope stability type analysis, the commulative effects of the dynamic lateral forces, $F_h(t)$, acting on the soil mass during the entire ground shaking period are presented by applying sustained (i.e., similar to a static force) constant magnitude lateral force on the soil mass acting in the direction of potential lateral instability. This force is termed as the "pseudo-static" force to distinguish its dyamic nature from that of the static loads. The magnitude of this single equivalent pseudo-static interial lateral force, (F_h) is calculated as follows, where conventionally the parameter k_h is used in place of the parameter to $(a_h)_{max}$ to represent the commulative affects of the acceleration-time history [$A_h(t)$] as discussed above.

$$F_h = k_h W \qquad (4)$$

The parameter k_h is the coefficient of the equivalent pseudo-static horizontal ground acceleration, or simply the "Seismic Coefficient".

The value of the parameter k_h or $(a_h)_{max}$ corresponding to the maximum value of pseudo-static force $(F_h)_{max}$ that the soil mass can support without sliding is termed as the coefficient of the yield acceleration, designated herein as $(k_h)_y$. For given and specific strength conditions (e.g., drained, undrained, liquefied, non-liquefied etc), the parameter $(k_h)_y$ represents its capacity against lateral sliding in terms of the coefficient of the peak ground acceleration, $(A_h)_{peak}$, that a soil mass can support without sliding. That is, the parameters $(k_h)_y$ can be thought of as the "capacity coeffcient" in terms of seismic coefficient" for a slope or ERS. In the literature this coefficient of the yield acceleration is often shown as k_{yield}, k_y, or k_c etc.

In a GLE based pseudo-static slope and overall (or global) stability of ERS, unless specified otherwise for unusually high slopes or ERS, the earthquake-induced ground motion represented by $A_h(t)$ is assumed to act uniformly througout the soil mass for which stability is being analysed. Similarly, the equivalent horizonal pseudo-static ground acceleration, represented by k_h, is also assumed to act uniformly through the soil mass contained within the slip surface, as shown schematically in Figure 3.

The horizontal acceleration-time history [$A_h(t)$] assumed to be acting uniformly on the soil mass being analyzed can be considered as the average ground motion that has been transfered from the basement soil through a shear zone of finite thickness (the slip surface).

The earthquake-induced average ground motion at the top of basement soil on which sliding, if any, will occur is thus the input motion. The input or the basement ground motion can be represented by acceleration-time history $A_{hg}(t)$. The corresponding values

of the coefficient of the input horizontal ground accelerations at the top of the basement soil as a function of time can be determined as follows:

$$a_{hg}(t) = \frac{A_{hg}(t)}{g}$$

The maximum value of the coefficient $a_{hg}(t)$ for an input acceleration-time history with a peak acccleration of $(A_{hg})_{peak}$ is given by:

$$(a_{hg})_{max} = \frac{HPGA\ at\ the\ top\ of\ the\ basement\ soil}{g} = \frac{(A_{hg})_{peak}}{g}$$

For a pseudo-static lateral stability analysis, the parameter $(a_{hg})_{max}$ can be thought of as the "demand coefficient" in terms of the coefficient of the input (basement) peak ground acceleration, $(A_{hg})_{peak}$.

In general, for a soil mass or ERS to be considered stable against lateral sliding, it is necessary that:

Capacity Coefficient ≥ Demand Coefficient

or, $$(k_h)_y \geq (a_{hg})_{max} \quad (5a)$$

Alternatively, the soil mass or ERS will be considered susceptible to lateral instability, if

$$(k_h)_y < (a_{hg})_{max} \quad (5b)$$

For SEE, the design ground motion represented by the parameters "Design HPGA" and "Design M" is the input (or basement) ground motion. That is,

$$(a_{hg})_{max} = \frac{Design\ HPGA\ (in\ g)}{g}$$

The coefficient $(a_{hg})_{max}$ corresponding to the Design HPGA is represented in the literature using various symbols, including k_{max} or a_{max}.

Thus, for SEE design:

1) A soil mass or ERS will be considered stable against lateral sliding, if

$$(k_h)_y \geq \left(\frac{Design\ HPGA}{g}\right) \quad (6a)$$

2) Alternatively, the soil mass or the ERS will be considered susceptible to lateral instability, if

$$(k_h)_y < \left(\frac{Design\ HPGA}{g}\right) \quad (6b)$$

However, as discussed earlier, unless a soil mass is affected by flow type liquefaction and to such extent that the total available minimum lateral resistance corresponding to the undrained residual shear strength (S_r) of the liquefied soil layer(s) falls below the total destablizing static lateral force, the relative sliding displacement during any loading cycle will stop soon after the input (basement) ground motion falls below the yield acceleration $(k_h)_y g$. The soil mass will regain a stable condition relative to the basement soil mass although shaking is continued, until after the next cycle when instataneous input acceleration [A_{hg}(t)] exceeds $(k_h)_y g$ again. The soil mass will experience additional relative displacement during this cycle but like earlier, the relative displacement will stop soon after the value of instataneous input acceleration [A_{hg}(t)] drops below $(k_h)_y g$.

The relative sliding displacement occurring during a given loading cycle for which [A_{hg}(t)] excceded $(k_h)_y g$ will be a relatively small amount. These relative lateral displacements occuring during each such cycle will accumulate. However, after each of these cycles the soil mass will regain its lateral stablity. The soil mass will achieve and maintain a relative stable condition after the last lateral displacement producing loading, which is likely to occur well before the cessation of ground shaking.

As will be discussed later, the amount of total relative sliding displacement that a soil mass may experience due to the input ground motion event depends on the ratio of the yield acceleration $(k_h)_y g$, and the peak ground acceleration [A_{hg}(t)] ($(k_h)_y / a_{hg}$), and the duration of ground shaking, which is a measure of the earthquake magnitue (M_w).

Based on the criteria expressed by the Equation 6 above, for SEE design, a slope or an ERS (including bridge abutments) will experience lateral instability or lateral sliding displacements if $\frac{(k_h)_y}{(\frac{Design\ HPGA}{g})} < 1.0$. However, for SEE design this does not necessarily mean that the slope or the ERS does not meet seismic design requirement to prevent failure or collapse. First it only means the ground or the earth structure will experience lateral displacements during the design seismic event. The soil slope, ERS or bridge abutment may displace laterally by a certain minimum amount (Δ_{min}) before it will be considered being failed or collapsed that would pose a threat to life-safety. This minimum displacement (Δ_{min}) is equivalent to the displacement capacity $\Delta_{capacity}$ specified in a displacement-based seismic design. In this case, the seismic design criterion can be expressed as:

$$\frac{\Delta}{\Delta_{capacity}} \leq 1.0 \qquad (7)$$

Where, Δ is the calculated lateral displacement due to the design ground motion (HPGA, M). Therefore, given a value for $\Delta_{capacity}$, a lateral displacement analysis will be necessary for a slope or an ERS when a pseudo-static slope stability analysis indicates that:

$$\frac{(k_h)_y}{(\frac{Design\ HPGA}{g})} < 1.0.$$

Pseudo-Static Lateral Slope Stablity Analysis

Pseudo-static slope stability analyses should be based on a Generalized Limit Equilibrium (GLE) based pseudo-static slope stablity analysis method. A computer program is usually used to perform GLE based pseudo-statc slope stability analyses. Similar to a static slope stability analysis, the user needs to select a resonable potential sliding mechanism(s) based on the site-specific topographic and subsurface conditions and other slope/soil mass loading and resistance characteristices, and then a range of kinematically feasible potential slip surfaces for each value of coefficent k_h.

Results of a GLE based pseudo-static analysis are expressed in terms of a factor of safety (FS) against sliding. It is the ratio of the available stabilizing total lateral nominal resistance to the total destabilizing lateral load acting in the direction of potential sliding. The final result of a set of pseudo-static slope stablity analyses (runs) performed for a given k_h value, all other conditions remaining the same, is a minimum factor of safety (FS_{min}) and a corresponding most critical slip surface.

To evaluate the lateral stability directly due to the design ground motion, this analysis is performed with k_h = (Design HPGA)/g.

If the results of this analysis indicate a $FS_{min} \geq 1.0$, the slope or the soil mass analyzed is considered stable against lateral instability during the design ground motion event. For an unsupported slope, a $FS_{min} \geq 1.0$ also indicates that the slope, for the purposed of SEE design, is not susceptible to any lateral deformation or liquefaction-induced lateral spreading. If the results of the above analysis indicate a $FS_{min} < 1.0$, the slope or soil mass analyzed is considered susceptible to lateral ground deformations or spreading displacements, but not necessarily to collapse or failure for the purpose of SEE design. In this case, additional analyses are necessary involving stablity and lateral deformations, as discussed below.

Example: Pseudo-Static Slope Stablity Analysis

Results of an example GLE based pseudo-static slope stablity analysis performed for a variation of the slope shown in Figure 3 are in Figure 4.

The selected soil parameters for this example soil profile are in Table 1.

Table 1. Soil Parameters

Layer No.	Soil Description	Unit Weight	Soil Strength Parameters		
			Short-Term	Long-Term	Seismic
1	Fill: Silty Sand (SM), with clay, dense	125	c= 500 psf ϕ= 34^o	c'=200 psf ϕ= 34^o	c=500 psf ϕ= 34^o
2	Clay (CL), stiff	128	S_u=1500 psf	c'=300 psf ϕ= 30^o	S_u=1500 psf
3	Sand (SP), medium dense	110	c= 100 psf ϕ= 35^o	c= 0.0psf ϕ= 35^o	Liquefied S_r =700 psf
4	Clay (CL), very stiff to hard	130	S_u= 2700 psf	c'=500 psf ϕ= 28^o	S_u= 2700 psf

The design ground motion parameters for this example are: Design HPGA = 0.6g and Design M_w = 7.5

It is assumed that that there is no groundwater and no liquefaction hazard. The short-term soil parameters presented in Table 1 are applicable. This example analysis uses the GLE based Morgenstern-Price method. A FS_{min}= 0.81 was obtained (Figure 4).

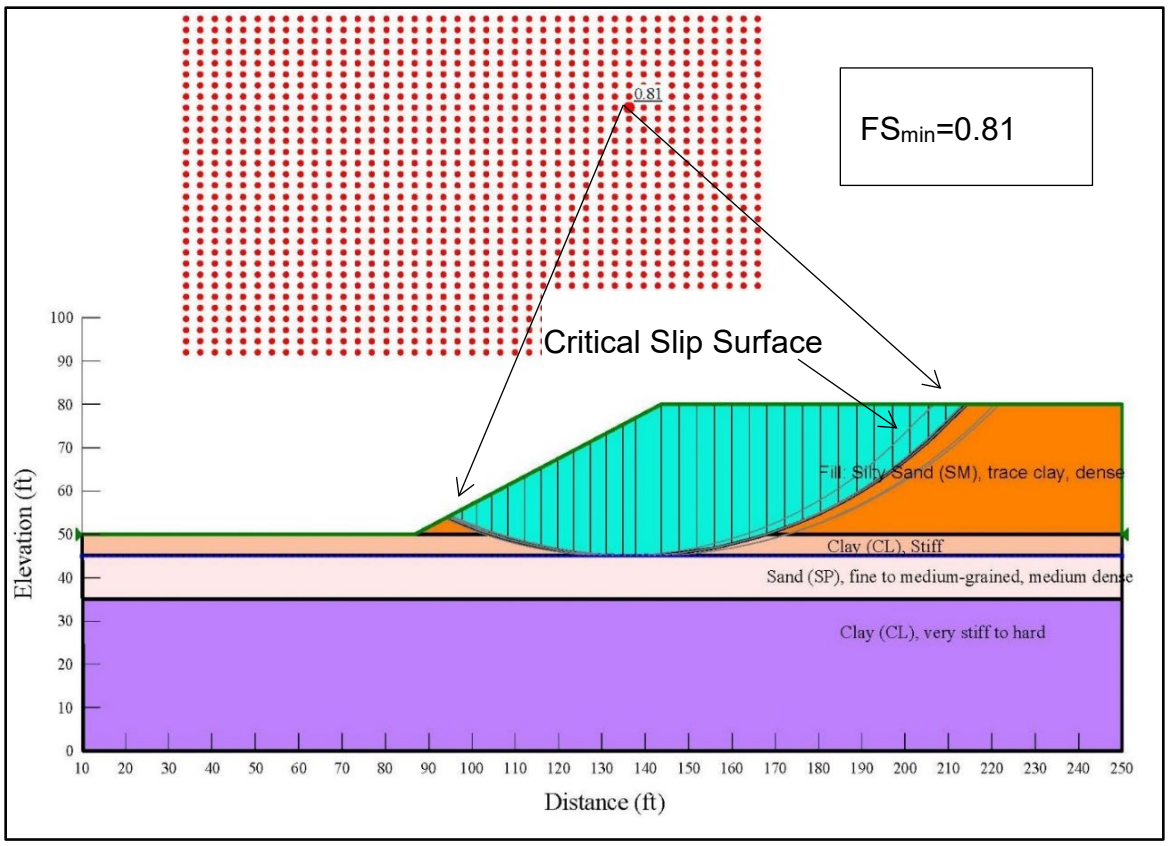


Figure 4. Results of An Example Pesudo-Static Slope Stablity Analysis

The above value of the FS_{min} indicates that the example slope is susceptible to lateral sliding during the design ground motion event. Unlike static design, this does not not necessarily constitute a failure or collapse for SEE seismic design. Further analyses and evaluations (discussed below) are necessary to estimate the amount of sliding displacement that the soil may experience during the design ground motion event.

Pseudo-Static Horizontal Yield Acceleration

For a given soil mass, a set of pseudo-static slope stability analyses can be performed by varying the value of the seismic coeffcient k_h, while all other factors remain the same. This will result in a set of FS_{min} values, one for each of the selected values of k_h. Results of such an analysis for the slope in Figure 4 are in Table 2.

Table 2. Results of Pseudo-Static Analyses for Yield Acceleration

k_h	FS_{min}
0.0	2.56
0.1	2.02
0.2	1.61
0.3	1.30
0.4	1.07
0.5	0.92
0.6	0.81

The above results can be plotted to determine the value of k_h corresponding to FS_{min}=1.0 (Figure 5). The value of k_h =0.44 corresponding to FS_{min}=1.0 is the coefficient of the pseudo-static horizontal yield acceleration $(k_h)_y$ defined earlier. For the above slope, (kh)y=0.44.

The slip surface associated with the yield acceleration $[(k_h)_y]g$ =0.44g, as shown in Figure 6, is the *most critical slip surface for this slope since this is the surface* along which sliding will occur during all seismic shaking events during which the input peak ground acceleration $[(a_{hg})_{max}]$ exceeds the yield acceleration $[(k_h)_y]g$, including the above SEE "Design Earthquake" event.

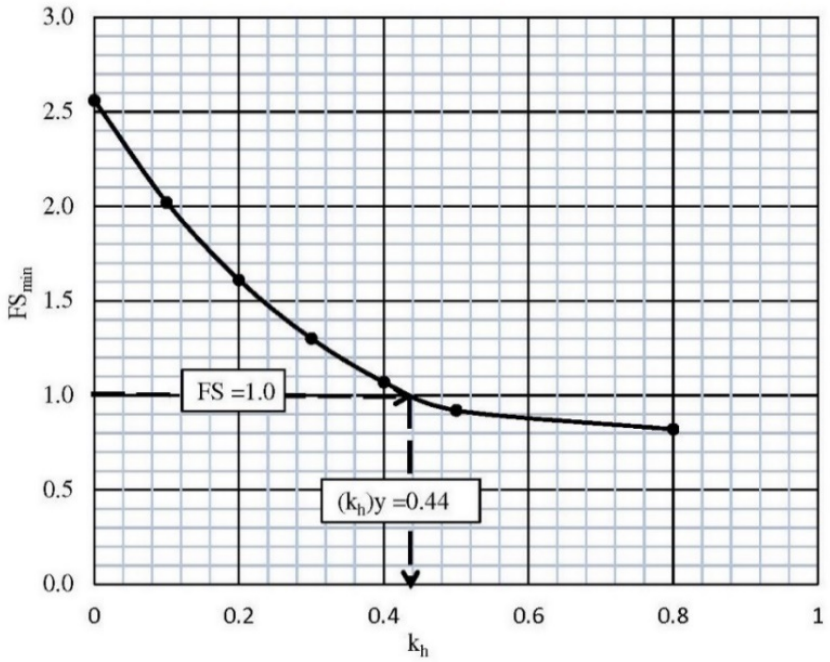


Figure 5. Determination of the Cofficient of Yield Acceleration

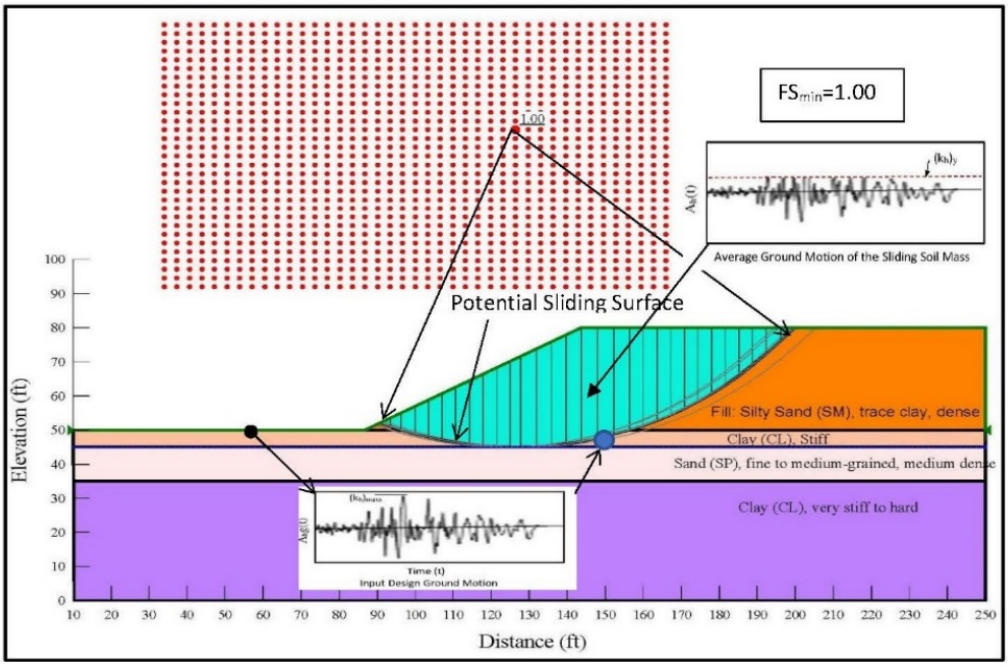


Figure 6. Determination of the Coefficient of the Yield Acceleration and Sliding Surface

As expected based on FS_{min} = 0.81 (Figure 4), the coefficient of yield acceleration or the capacity coefficient $(k_h)_y$ = 0.44 is less than the demand or the maximum coeffcient of the input peak ground acceleration, $(A_{hg})_{max} = (\frac{Design\ HPGA}{g}) = 0.6$

For the given slope and the magnitude of the available total nominal lateral resistance, the coefficent $(k_h)_y$ and the associated sliding surface are unique in the sense that neither depends on the input or the design ground motions.

Note that as seismic pseduo-static slope stability analyses are currently performed, the slip surfaces, even for a given slope and lateral resisting conditions, vary with the magnitude of the input horizontal peak ground acceleration (k_h). Based on the above discussion on the uniqueness of the sliding surface, for a given slope and lateral resistance condition, it is recommend that when performing pseudo-static slope stability analyses for all values k_h, the slip surface be fixed to “sliding surface” associated with the yield acceleration. For given slope, any change in the lateral resisting strength parameters will result in a change of the value of $(k_h)_y$ as well as the corresponding “sliding surface”.

Based on the above analysis, for this design:

$$(k_h)_y \leq (\frac{Design\ HPGA}{g})$$

Thus, further analysis is necessary to estimate lateral displacements that may occur by sliding during the design seismic ground motion event.

Newmark Displacement Analysis Method (NDAM)

Newmark (1965) proposed a very simple method for estimating the co-seismic relative lateral displacement between a finite, rigid mass of soil resting on the surface of a semi-infinite, rigid basement soil mass, when the later is subjected to an earthquake-induced ground motion event.

Due to the simplifications involved with this analysis method, the displacement estimated based by NDAM was intended to be used as an index, rather than as a specific value, for assessing the degree of the potential hazards associated with the co-seismic relative lateral movements between the soil mass and the basement soil. Furthermore, the direction of the relative lateral displacement estimated using NDAM is assumed to be horizontal, irrespective of the directions of the slip surface.

Newmark deformation is purely plastic deformation occurring by sliding. No elastic deformation that may occur prior to the onset of plastic deformation is considered in this analysis. For soil-to-soil sliding sliding only surface, any eleastic deformation should be small and may be neglected.

However, for a soil mass restrained by external supported mechnical elements, in particular by deep foundations, the amount of lateral deflection or deformation that may occur prior to reaching the plastic state of deformation assumed in the Newmark method may be significant. That is, a pile supported soil mass (e.g., bridge abutment) does not experience any Newmark type lateral displacement until a plastic hinge is formed in the pile (assuming soil does not flow around the pile). Significant lateral deflection or deformation will occur before the plastic hinge is formed. This lateral deformation is not durectly accounted for directly in the Newmark-type analysis. Furthermore, a plastic hinge forms in a pile when total lateral force (static +seismic) exceeds the pile nominal lateral resistance (R_N). Once the plastic hinge forms, any attempt to increase the total lateral force on the pile will result in a Newmark-type lateral sliding displacement since the soil-mass must have already reached the plastic state in order for the pile to deflect laterally by the amount necessary to form a plastic hinge.

The total lateral force on the pile will never exeeed R_N during any seismic event. R_N is also equal to the maximum lateral resistance that the pile will offer to the soil mass during an earthqauke. It should be noted the k_h value at which the system yielding occurs and Newmark type displacements start of occur is the equal to the coeffcient of the yield acceleration $(k_h)_y$ for the piles-soil mass system. As stated earlier, $(k_h)_y$ represents the lateral capacity of the system, and

A complete seismic lateral displacement analysis using the NDAM comprises of two types of analysis: a force-based analysis (peudos-static slope stability analysis), and a displacement-based analysis.

Step 1: Pseudo Lateral Stability Analysis to Determine Yield Acceleration $(k_h)_y g$

The procedure for evaluating the yield acceleration of a sloped soil mass that may be susceptible to lateral sliding displacements during the design ground motion event is presented earlier.

This section briefly discusses this procedure for a generalized soil-mass KLMN (Figure 7a), which can be used as a convenient tool to discuss the main features of the NDAM.

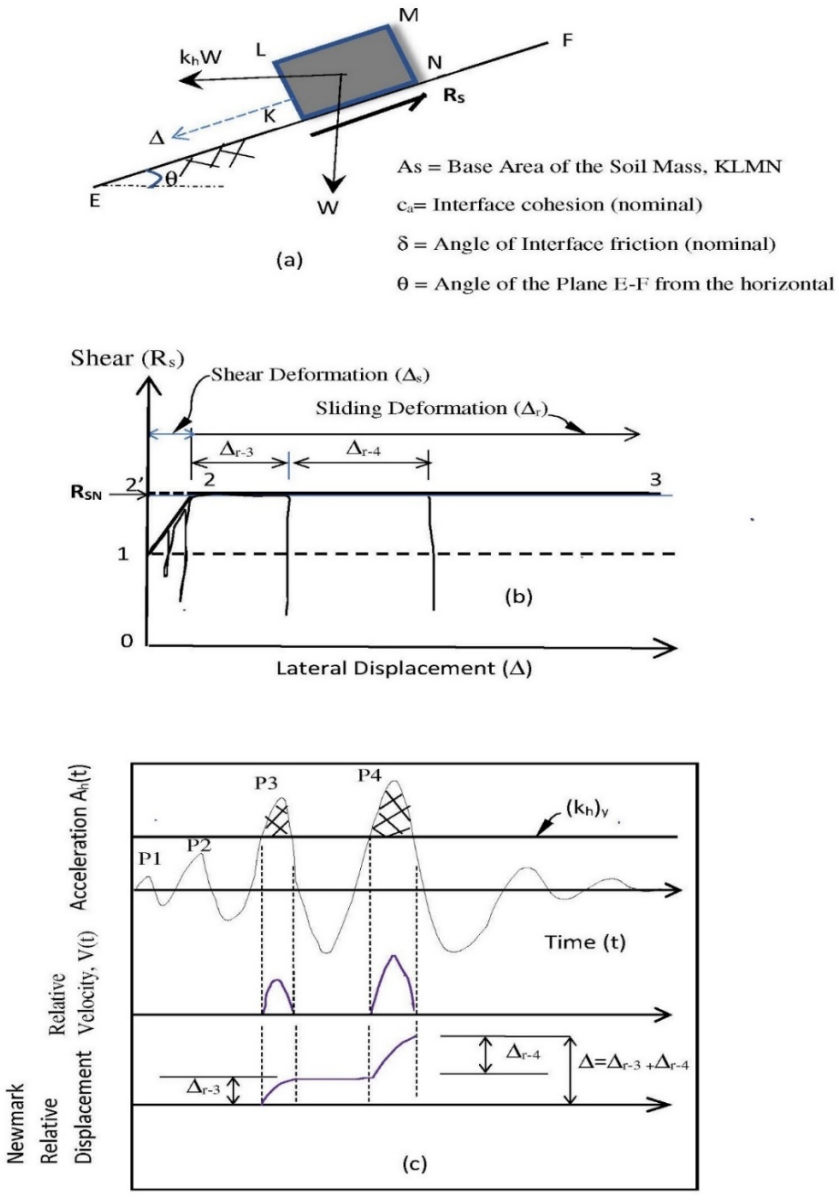


Figure 7. Schematic Diagram Depicting: (a) Newmark's Rigid Soil Mass on an Inclined Plane; (b) Interface Shear -Lateral Displacement Relationship;(c) Newamrk's Rigid Body Type Cosesimic Displacement (Δ).

In Figure 7(a), the potential lateral effects on the soil mas KLMN of ground shaking during an earthquake-induced ground motion event, similar to the pseudo-static stability analysis discussed above, is respresented by a pseudo-static lateral force ($k_h W$) acting horizontally to the left. It is assumed that the soil mass is stable against sliding in the upslope direction.

The pseudo-static factor of safety against lateral sliding stability of the soil mass KLMN along the interface NM in Figure 7(a) is:

$$FS = \frac{R_{SN}}{R_{Total}} \qquad (8)$$

Where, R_{SN}= total nominal lateral resistance of the soil mass, and R_{total}= total destabilizing lateral load acting on the soil mass

By solving Equation (8) for FS_{min} = 1.0, the coeffcient of the pseudo-static yield acceleration $(k_h)_y$ for the soil mass KLMN can be evaluated. If $(k_h)_y$> (Design HPGA/g) lateral sliding is indicated, the Newmark displacement analysis continued as discussed in the next step.

Step 2: Rigid Body Type Cosesimic Lateral Displacement

The step involves estimation of the co-seismic relative lateral displacement between the soil mass KLMN of weight W and basement soil mass. For this analysis, the input ground motion to the base of the soil mass KLMN is the design ground motion $A_{hg}(t)$ evaluated or specified at the surface (EF) of the basement soil mass at a point just below the base of the soil mass KLMN.

Stability against downslope sliding is provided by the shear resistance (R_s) mobilized on the interface or contact surface between the soil mass KLMN and the slope EF.

In general, the mobilized soil-to-soil interface shear resistance (R_s) versus shear deformation (Δ) relationships are non-linear. In practical applications, such a relationship is often simplified in the form of a bi-linear elasto-plastic curve. For lateral stability or large deformation analyses, such as the NDAM, such this relationship is often further simplified to a single horizonal line, which represents a perfectly-plastic interface shear behavior.

For this example, the interface undrained shear resistance-deformation relationships, R_s (Δ), when the soil mass KLMN is subjected to a monotonically applied shear force in the downslope, is shown in Figure 7(b). The simplified bi-linear R_s (Δ) relationship is represented by the lines “1-2” and “2-3” in Figure 7(b), where “1” represents the initial state or static shear component due to gravity ($W\sin\theta$) acting on the soil mass in the direction of potential sliding. Line 1-2 represents lateral displacement due to shear

deformation of the interface shear zone. Displacement beyond Point 2 occurs by sliding along a slip plane.

For soil-to-soil shearing, lateral deformation A'B due to pure shear is usually small (on the order of 0.1" to 0.4"). As noted earlier, this deformation due to pure shear is neglected in the NDAM, and the interface shear (R_s) versus lateral deformation (Δ) relationship is assumed to be represented by the line "2'-2-3" in Figure 7(b). This perfectly-plastic shear resistance-deformation relationship can be expressed as follows:

$$R_S(\Delta) = R_{SN} \quad (9)$$

In practice, the design horizontal ground motion $A_{hg}(t)$ is assumed to be the same as the free-field design ground motion evaluated or specified at the ground surface of a project site. For this discussion, it is assumed that the acceleration-time history shown schematically in Figure 7 (c) represents the design ground motion $A_{hg}(t)$. The positive horizontal direction of the accelerations shown in Figure 7(c) corresponds to the right (opposite to the direction of k_hW), as shown in Figure 7(a).

For these conditions, as soon as the base slope starts to experience ground motion, such as that shown in Figure 7(c), the soil mass KLMN will instantaneously experience the same ground motion resulting in the pseudo-static inertial loads $k_h(t)W = A_{hg}(t)W$. During the initial period the ground shaking is usually low in magnitude. For an interface characterized by a bi-linear shear-deformation relationship, such as the one represented by the points 1-2-3 in Figure 7(b), the soil mass KLMN is likely to experience some small lateral displacements due to shear deformation of the soils within an interface shear zone of finite thickness.

However, the maximum lateral movement of the soil mass KLMN due to the above shear deformation will remain $\leq \Delta_c$ as long as $R_{Total} \leq R_{SN}$. This condition is depicted in Figure 7(b) during the time the basement slope during the 1st and 2nd cycles of positive acceleration respresented by peak accelerations P-1 and P-2 in Figure 7(c). Shear deformation will also occur during the initial portion of the 3rd positive acceleration cycle until the time when R_{Total} becomes equal to R_{SN}, corresponding to $A_{hg}(t) = (k_h)y$.

A discussed earlier, for a given slope if the value of R_{SN} does not change during ground shaking, the value of the parameter $(k_h)_y$ will remain the same during the ground motion event. In that case, the corresponding yield acceleration $[(k_h)_y]g$ versus time (t) can be respresented by a straight line as shown in Figure 7(c). This line represents the maximum value of the coefficient k_h (t) or A_{hg} (t) that the soil mass KLMN can be subjected to without any sliding at the interface.

At any instant during shaking of the basement slope, $A_{hg}(t)$ becomes equal to $(k_h)_y$, which results in $R_{Total} = R_{SN}$, the interface is stressed to the yield condition as shown in Figure 7(b). At this point, any attempt to increase R_{Total} to a value greater than R_{SN} will cause downslope sliding movement along the interface.

In Figure 7(c), the above condition occurs for the first time during the 3rd cycle of positive acceleration starting at time $t_{1\text{-}1}$ and persisting until $t_{1\text{-}2}$ during which $A_{hg}(t)g$ remains greater than $[(k_h)_y]g$. Based on NDAM, due to shear yielding condition of the interface, during this time period the input acceleration $A_h(t)$ acting on the soil mass KLMN remains limited to $(k_h)_y$. This results in differences in the accelerations of soil mass KLMN and the basement slope causing differences in their velocities, which in turn results in relative movements between the two bodies by sliding along the interface slip plane

The relative velocity-time history, as shown on the middle panel of Figure 7(c), due to the 3rd cycle of basement slope acceleration, is obtained by integrating the hatched base acceleration-time history starting from $t_{1\text{-}1}$ and ending at $t_{1\text{-}3}$ at which point the relative velocity drops to zero. The corresponding Newmark Displacement ($\Delta_{r\text{-}3}$) is obtained by integrating the relative velocity from time $t_{1\text{-}1}$ to $t_{1\text{-}3}$. Once the relative velocity drops to zero, the two soil masses move together and no additional Newmark Displacement occurs until the positive basement slope acceleration $A_{hg}(t)g$ exceeds $[(k_h)_y]g$ again. For this example in Figure 7, this happens again during the next or the 4th cycle of positive acceleration cycle. The calculations performed for the 3rd cycle are then repeated to evaluate Newmark Displacment ($\Delta_{r\text{-}4}$) occurring due to the the 4th cycle of positive accelerations with peak P-4.

The cumulative Newmark Displacement at end of the 4th cycle of positive acceleration can be obtained by adding $\Delta_{r\text{-}3}$ and $\Delta_{r\text{-}4}$. The Newmark Displacements, if any, occurring during the subsequent cycles of positive basement accelerations are calculated in a similar manner, and the total Newmark Displacement (Δ) at the end of ground shaking is calculated by summing the Newmark Displacements ($\Delta_{r\text{-}i}$) estimated for each of the cycles during which $A_{hg}(t)g$ exceeds $[(k_h)_y]g$. For this example, in Figure 7, the positive basement acceleration does not exceed $[(k_h)_y]g$ during the remainder of the ground shaking period, and thus no additional sliding deformation occurs. The total Newmark Displacement (Δ) is thus obtained by summing $\Delta_{r\text{-}3}$ and $\Delta_{r\text{-}4}$ as shwon in Figure 7(c).

The maximum or the peak ground acceleration that a sliding soil mass is subjected is limited to the yield acceleration $[(k_h)_y]g$ as shown Figure 7.

Evaluation of Newmark Displacement Using Empirical Correlations

Newmark Displacement (Δ) can be estimated by using published empirical correlations. At this time, the use of the following empirical correlation (Bray and Tavasarou (2007)) is recommended:

$$\Delta\,(\text{inches}) = 0.3937\,\text{Exp}[-0.22 - 2.83\,\text{Ln}(k_h)_y - 0.333\,\{\text{Ln}((k_h)_y)\}^2 + 0.566\,Ln((k_h)_y)\,Ln((a_{hg})_{max}) + 3.04\,Ln((a_{hg})_{max}) - 0.244\,\{\text{Ln}((a_{hg})_{max})\}^2 + 0.278(M-7)] \quad (10)$$

For clarity, in the above correlation the original symbols k_y and PGA used in Bray and Tavasarou (2007) are replaced with the symbols $(k_h)_y$ and $(a_{hg})_{max}$, respectively. For the Department's projects, when the design ground motion is specified or evaluated at the ground surface in accordance with the procedure in the SDC, no modification to any of the design ground motion parameters, including $(a_{hg})_{max}$ (which is equal to the design PHGA/g), is necessary for use in the above correlation or any other such empirical correlations.

Equation 10 provides the estimated median lateral displacement (Δ). The estimated lateral displacement range is 0.5Δ to 2.0Δ.

Due to the sigificant simplified assumptions and the uncertanities involved with the above analysis procedures, it is recommended that any lateral displacement estimate less than 1.0 inch should be considered as equal to zero (i.e., no lateral spreading or displacement).

Liquefaction-Induced Flow Failure

Steeply sloping ground, and ERS or retained earth are under the influence of sustained (or static), destabilizing gravity-induced lateral forces compared with their available stabilizing nominal lateral resistances. Such grounds are particularly susceptible to lateral instability during seismic ground motion events. For example, landslides along steeply sloping mountains sides are common occurrences during strong motion events.

Past observations after large earthquakes indicted that steep slopes and retained-earth adjacent to near water bodies such as those associated with ports, dams, levees, and the banks of rivers and other water bodies are particularly prone to very large lateral ground displacements which, once initiated, occur under the action of gravity alone like flow type aseismic landslides.

Flow failure of a soil masses during earthquakes can occur when the available total lateral resistance drops, due to liquefaction, below the total static driving force.

At this time, a GLE based pseudo-static slope stability type analysis, as discussed earlier, is usually performed to evaluate if a soil mass is susceptible to liquefaction-induced flow failure. For this analysis, the available total lateral resistance of the soil mass or the ERS is evaluated based on the undrained residual strength of the soils that are predicted to liquefy when subjected to the design ground motion. The potential effects of soil liquefaction in the available lateral nominal resistance of lateral resisting elements, if any, such as deep foundations and ground anchors, also need to be considered.

The pseudo-static slope stability analysis for this purpose is performed by the setting, k_h=0.0 and using undrained residual shear strengths for the liquefied soil layer(s)

The soil mass analyzed is considered susceptible to liquefaction-induced flow failure if the results of this analysis indicate a FS_{min}<1.0. Otherwise, the soil mass is considered not susceptible to flow failures.

In the lateral case, the soil mass may still be susceptible to liquefaction-induced lateral spreading and needs to be further analyzed as discussed earlier.

Lateral displacements due to flow failures are usually very large. A practical method to estimate lateral flow displacement is not available currently. For design, it is assumed that a soil mass susceptible to flow failure is likely to experience large movements which for all practical purposes will constitute a failure or collapse, and thus need to be avoided. This can be achieved by improving or modifying the conditions of the potentially liquefiable soil to reduce or eliminate liquefaction hazards, and/or by using deep foundations to increase the available total lateral nominal resistance.

Liquefaction-Induced Lateral Spreading Displacements

Researchers have used several approaches, including empirical or semi-empirical correlations, numerical analysis and physical modeling, to estimate lateral ground deformations or displacements due to earthquakes. Most of these approaches are applicable to non-liquefied free-field ground conditions. Due to the complexities involved, both the tools available for and their capabilities to estimate liquefaction-induced lateral spread displacements during future earthquakes are very limited.

Unless specified otherwise in a project specific seismic design criteria, the following simplified methods may be used to estimate liquefaction-induced lateral spread displacement for most regular projects.

Liquefaction-Induced Lateral Spread Displacements for Free-Field Conditions

The following analysis procedures may be used to estimate liquefaction-induced free-field lateral displacement involving the third category of lateral spreading discussed in the liquefaction induced lateral spreading section.

Hamada et al (1986) Procedure

Hamada et al (1986) proposed the following simple empirical relationship, which may be used for rough or initial estimates of the free-field liquefaction-induced lateral displacements for the ground shown in Figure 9.

$$\Delta = 0.75\, H^{1/2}\theta^{1/3} \qquad (12)$$

Where, Δ = Permanent ground displacement in the lateral (horizontal) direction (m)

H= Thickness of the liquefied soil layer (m)

θ = The larger gradient of the ground surface or the lower boundary face of the liquefied soil layer (%)

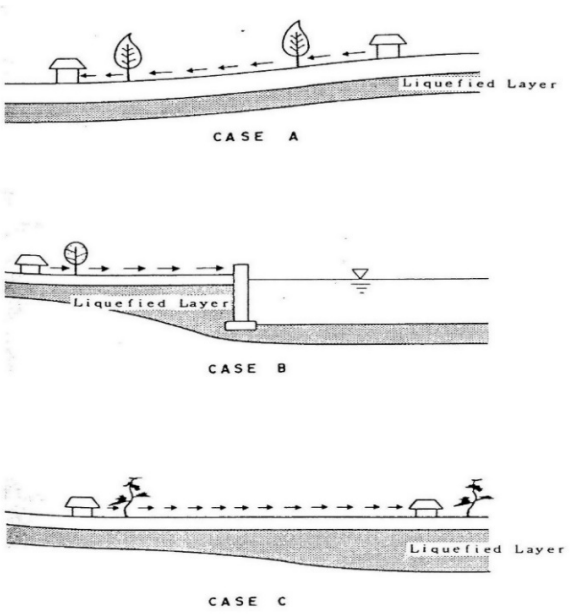


Figure 9. Types of Permanent Ground Displacements (Hamada et al., 1986)

When using Equation (12), the lateral ground displacements in the field should be expected to range from about ½ Δ to 2Δ . Based on Hamada et al (1986), in some of the field case histories used in the development of the Equation (12), lateral ground moments continued to occur after the cessation of the ground shaking.

Multilinear Regression (MLR) Procedure

Bartlett and Youd (1992, 1995) and Youd et al (2002) developed more details for the prediction of free-field liquefaction-induced lateral spread displacements. In the development of these empirical correlations, lateral spread in the simplest form is defined as the riding of a non-liquefied surficial soil layer or crust on a directly underlain liquefied soil layer either gently sloping ground toward a free face (i.e., not supported by any mechanical means, such as ERS) unsupported) downward grade change, such as that occurred due to incised river, canal, creek, lake, ponds, bluffs or other sudden depressions within an otherwise relatively flat area. The basic mechanisms involved with the above definition of lateral spread are depicted in Figure 9, although the associated mechanisms in the field are far more complex.

Bartlett and Youd (1995) found that liquefaction-induced lateral spread displacement is a function of earthquake, topographical and soil factors. Youd et al (2002) developed two different empirical correlations to estimate liquefaction-induced free-field lateral spread displacement (Δ), one for gently sloping ground (with no free face) condition and the other for both sloped and flat ground with a free face condition as presented in the following sections.

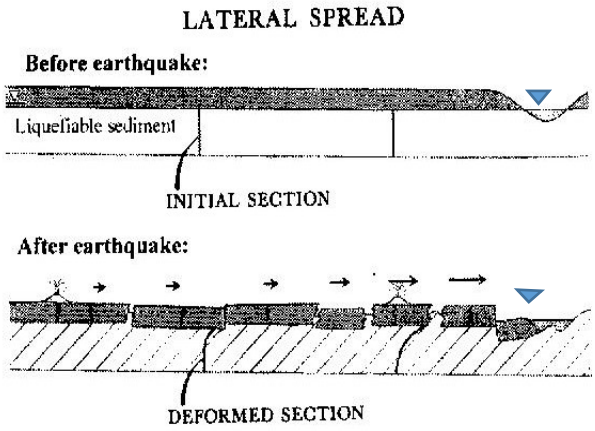


Figure 10. Liquefaction-Induced Lateral Spread (Youd, 2018)

Gently Sloping Ground Condition

The liquefaction-induced free-field lateral (horizontal) spread ground displacement (Δ in meters) in this case, which involves soil mass movements down a gently sloping ground, may be estimated based on the following empirical correlation:

$$Log\,\Delta = -16.213 + 1.532M - 1.406\,log\,R^{*} - 0.012\,R + 0.338\,log\,S + 0.540\,log\,T_{15} + 3.413\,log\,(100 - FC_{T15}) - 0.795\,log\,(D50_{T15} + 0.1\text{ mm}) \quad (13)$$

The various parameters in Equation (13) are defined in the following section. For these conditions, in addition to soil liquefaction-induced reduction in the shear strength of the underlying liquefiable soil layer(s), sustained or static lateral destabilizing mass forces due to gravity acting on lateral spread soil mass in the direction of the slope play a significant role.

Free-Face Ground Conditions

The following empirical correlation is recommended to estimate the lateral (horizontal) ground displacement (Δ in meters) toward a free face, as defined above:

$$Log\,\Delta = -16.713 + 1.532M - 1.406\,log\,R^{*} - 0.012\,R + 0.592\,log\,W + 0.540\,log\,T_{15} + 3.413\,log\,(100 - FC_{T15}) - 0.795\,log\,(D50_{T15} + 0.1\text{ mm}) \quad (14)$$

Where, $R^{*} = R_0 + R$ and $R_0 = 10^{(0.089\,M_w - 5.64)}$, and

$R=R_{rup}$ = mean site-to-fault rupture surface distance (km); T_{15} = cumulative thickness (m) of liquefied soil layers with corrected blow counts $N1_{60}$ <15; FC_{T15} = average fines (passing US #200 sieve) content (%), of the liquefied soil layers included in the T_{15} calculations; $D50_{T15}$ = average mean grain size (mm), of the liquefied soil layers included in the included T_{15} calculations; S = ground slope (%), and W = free-face ratio (%), defined as the ratio of the height (H) of the free-face to the horizontal distance (L) from the toe of the free face to the point on the ground Δ is evaluated for, as shown in Figure 11.

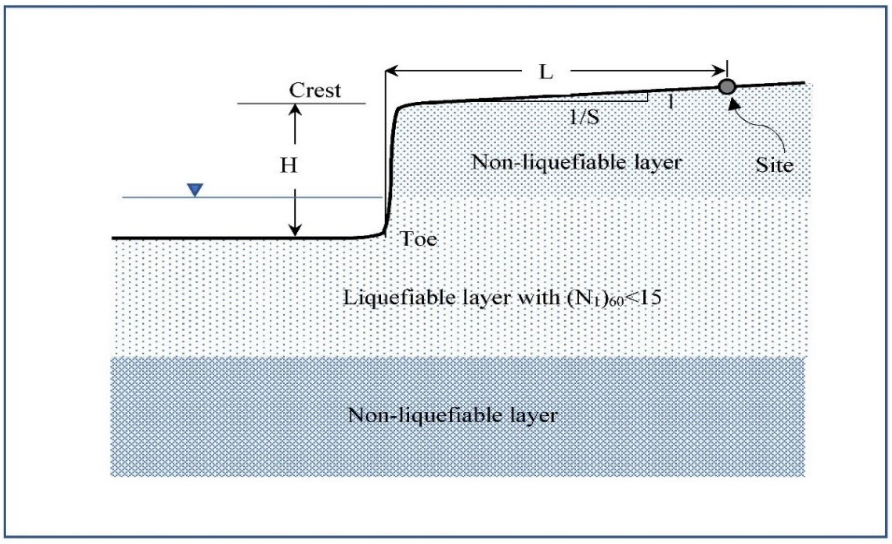


Figure 11. Model Parameters (Modified after Bartlett and Youd, 1992)

Seismic Lateral Displacement Analysis for Slopes and Earth Retaining Structures

The NDAM is used to perform lateral spreading hazards for mildly (> 6 %) to steeply sloping slopes, ERS and bridge abutments.

Externally Unrestrained Slopes and ERS

The NDAM method is used to estimate liquefaction-induced lateral displacements of moderate (> 6 %) to steeply sloping ground and ERS not restrained or stabilized by means of externally founded structural elements such as piles, tiebacks and ground anchors. These ERS include gravity walls and conventional, semi-gravity type reinforced concrete walls founded on shallow foundations, and internally stabilized walls, such as soil nail and mechanically stabilized earth walls and geosynthetic-reinforced walls.

For these ERS, two types of lateral stability and displacement mechanism need to be considered: (1) Overall or global stability using GLE based pseudo-static slope stability method as discussed earlier, and (2) Lateral sliding along the base using pseudo-static limit-equilibrium analysis of the forces and resistances in the lateral or horizontal direction. In this analysis, the seismic inertial force acting on the ERS or ERS-backfill composite sliding mass is calculated using the seismic coefficient (k_h), and the yield acceleration capacity $(k_h)_y$ is calculated based on a FS=1.0 against lateral sliding, in a similar manner discussed above for slopes.

The lower of the two $(k_h)y$ values, one based on the overall or global stability and second based on the lateral sliding stability along the base, will control the lateral sliding failure of the wall, and thus is yield capacity of the ERS in term of seismic coefficient. This lower value of $(k_h)_y$ is used to calculate lateral displacement using NDAM in the similar manner discussed above for a slope, for which there were only one $(k_h)_y$ value.

Externally Restrained Slopes and ERS

Liquefaction-induced lateral ground displacement analysis for externally restained ground and ERS is of major significance for several transportation systems. These systems include bridge supports founded on piles and ERS supported vertically on and/or laterally supported by deep foundations and anchored walls.

In concept, NDAM for these systems is very similar to the laterally unrestrained systems or slopes as presented above. The main difference is that lateral stability and lateral spreading displacement analyses need to incorporate the lateral nominal resistance offered by the external restraining elements on lateral stability and lateral spreading displacements. The total lateral nominal resistance is the sum of the lateral nominal resistances of the soil and the externally restained elements.

Once the lateral nominal resistance per unit width of the slope or ERS is determined, it can be applied as an additional lateral resisting force on the potential sliding soil-mass. One major complexity is that the two lateral nominal resistances are not moblized at the same lateral displacement. As discussed the soil mass is considered rigid and the sliding material is characterized as perfectly plastic. This means that no lateral displacement occurs prior to the mobization of the soil lateral nominal resistance or yield condition. This assumption is considered reasonable for soils but not for external restraining elements such piles. Significant lateral deformation of the pile is necessary before its lateral nominal resistance is mobilized.

The piles in these cases derive their lateral resisting capacities on the soils below the potential sliding surface, which in turn depends on the magnitude of the lateral resistance offered by the piles. The restraining element and the soil mass interact in a complex manner.

Additional difficulties arises due to nature and timing of ground motion and soil liquefaction, and interaction with other elements, for example the superstucture and the abutment wall in the case of a bridge abutment founded on piles.

In summary, lateral spread hazard analyses for these sytems are very difficult due to the complex nature of the interactions between soil and the external restaints, and the phenomena of earthquake motions and soil liquefaction, and other factors.

Due to the above complexities, the state of knowledge and practical analysis procedures are limited and evolving rapidly. At this time, simplifying assumptions must be made to be able to develop practical lateral spreading displacement analyses for these types of structures. However, to be acceptably reliable, even the simplest of the methods require some modeling and numerical soil-structure-interaction analyses.

The Department's currently recommended procedure for estimating liquefaction-induced lateral spreading displacements at bridge supports with deep foundations is in MTD 20-15 (2017). An example lateral spreading analysis using the procedure in MTD 20-15 is included herein.

Until ERS specific analysis procedures are available, the procedure may be used to perform liquefcation-induced lateral spreading hazard for other types of externally restained ERS.

Other Analysis Methods for Estimating Lateral Spread Displacements

Researchers have been using physical modelling and numerical methods to study the phenomenon of liquefaction-induce lateral spreading. While the results of such studies are useful to understand the basic mechanisms and identify the factors, due to the complexities involved, their applicability as well as reliability at this time are limited for use in the design of routine projects. Use of numerical methods requires special expertise, skills and tools, and may be considered for large and complex projects in consultation with the structure designer.

Hazard Mitigation Strategies

As discussed earlier, deep foundations are considered the most appropriate for supporting bridges at sites susceptible to liquefaction-induced lateral spreading hazards during earthquakes. One of the main reasons for this is that a pile foundation not only to transfers loads to deeper, usually more competent soils, it also provides additional lateral nominal resistances to the potentially laterally unstable soil mass. In some cases, use of pile foundations needed to support other loads may be adequate to limit or even prevent any lateral spreading displacements. In all cases, provided the deep foundation is embedded into the foundation soils below the moist critical slip surface by length not less than the critical length (L_c), as defined in the MTD 3-1, will provide some mitigation. The

critical length (L_c) in this case must be evaluated by ignoring lateral resistances from the soils above the most critical slip surface.

Ground improvements, with or without deep foundations, may be used to reduce or even eliminate lateral spreading hazards. Ground improvements may be targeted to reduced or eliminate liquefaction hazards and/or provide additional lateral resistance to the soil mass susceptible to lateral spreading. Ground improvement methods may include: removal and replacement of liquefiable soils with compacted fills, dynamic compaction, stone columns, soil mixing and grouting.

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Lateral Spreading Analysis Example

Memo-To-Designers 20-15, dated May, 2017 (MTD 20-15) presents the procedure for liquefaction-induced lateral spreading analysis for new and existing bridges. This procedure includes both geotechnical and structural analyses to be performed by Geotechnical Services (GS) and Structure Design (SD), respectively, in an interactive and, when necessary, iterative manner. The following is a step-by-step example of geotechnical liquefaction-induced lateral spreading analysis using the MTD 20-15 procedure.

Background

Due to relatively low strength and high compressibility, soils susceptible to liquefaction are not generally considered suitable for supporting bridges on shallow or spread-footing type foundations. In California, most existing bridges located at such sites are likely to be supported on deep or pile foundations. The Department's current practice is to use deep foundations when soil liquefaction is predicted to occur at a structure support location.

Bridge support locations predicted to experience soil liquefaction due to the design ground motion may be susceptible to liquefaction-induced lateral spreading hazards. MTD 20-15 includes a procedure for liquefaction-induced lateral spreading hazard analysis for Seismic Design Criteria (SDC) compliant new bridges and, existing bridges for which the performance criteria are described in MTD 20-1.

These bridges are designed to have large displacement capacity against collapse when subjected to the Safety Level Evaluation (SLE) "Design Earthquake" as defined in MTD 20-1. That is, seismic analysis and design of these bridges are concerned with the stability or collapse, not serviceability, when subjected to the earthquake effects not excceding those corresponding to the SLE "Design Earthquake." In other words, seismic design of these bridges are concerned only with large deformations. With regard to the ground, such large deformations are generally associated with soil-liquefaction. Thus, for seismic lateral stablity of pile foundations, large lateral ground deformations generally associated with liquefcation-induced lateral spreading are of concern.

The scope of this example analysis is limited to the case of a typical pile-supported bridge abutment. It is assumed that the reader is familiar with the contents of MTD 20-15.

The lateral spreading analysis procedure included in MTD 20-1 utilizes the Newmark's Displacement Analysis Method (NDAM) to estimate liquefaction-induced lateral spreading displacements at bridge abutments. This method was developed by assuming a rigid block soil resting on the surface of a rigid base with the interface characterized by a perfectly plastic load (shear)-displacement (sliding) behavior. During a seismic shaking event, it is assumed that such rigid body when resting on the surface of an inclined rigid base can slide downward only. This sliding displacement occurs incrementally during those cycles of acceleration-time history with the peak acceleration greater than a threshold values. The threshold acceleration, termed as the yield acceleration, represents the magnitude of the input ground acceleration at which the rigid body yields in shear along the perfectly plastic interface. Due to the assumed perfectly plastic shear behavior, the NDAM cannot predict the pre-yield shear displacement of a flexible soil-mass, in particular those supported by flexible deep foundation, experience during a seismic shaking event.

The base or input motion is represented by the design acceleration-time history, which is designated herein as $[a_{hg}(t)]g$, where a_{hg} is the coefficient of horizontal ground (base) acceleration, t is the time and g is the acceleration due to gravity. For SLE design, the coefficient of the design horizontal peak ground acceleration, HPGA, is the absolute peak value of the base acceleration, $|(a_{hg})_{max}|$, referred to hereafter in this document simply as $(k_h)_{max}$. The parameter $(k_h)_{max}$ is often referred to as a_{max} or k_{max} in the literature. It represents the design peak or maximum demand in a pseudo-static slope stability analysis or in the Newmark type rigid body displacement analysis.

The NDAM consists of two parts or steps:

1) *Limit Equilibrium Based Pseudo-Static Analysis for Lateral Sliding Stability:* A limit-equilibrium based pseudo-static lateral sliding stability analysis in which the seismic load effects on the soil mass are represented by a lateral inertial force, k_hW, acting in the direction (s) of the potential sliding. Here, k_h ($=a_{hg}$) is the seismic horizontal acceleration coefficient, often referred to simply as the "Seismic Coefficient," and W is the weight of the soil mass being analyzed for lateral stability. The purpose of this analysis is to evaluate the value of k_h that results in a minimum factor safety against lateral sliding, FS_{min} = 1.0 and the corresponding slip surface. For soil slopes, a FS_{min}=1.0 implies full mobilization of the available shear strength or incipient yielding (in shear) of the soils along a certain slip surface. The corresponding k_h value is termed as the coefficient of (horizontal) yield acceleration, designated herein as $(k_h)_y$. It is often designated as k_y or k_{yield} in the literature.

2) *Permanent Sliding Displacement Analysis:* The NDAM assumed that any time during the ground shaking period the magnitude of the base acceleration $[a_{hg}(t)]g$ acting in the opposite direction of potential sliding exceed the yield acceleration $[(k_h)_y]g$ some finite amount of relative displacement between the soil-mass and the base occurs by sliding. Sliding stops soon after the magnitude of the base acceleration $[a_{hg}(t)]g$ drops back to less than the yield acceleration $[(k_h)_y]g$. No relative sliding displacement occurs until the next cycle during which base acceleration $[a_{hg}(t)]g$ exceeds the yield acceleration again $[(k_h)_y]g$. After each of these cycles, the soil-mass comes to a stop with respect to the base.

The total sliding displacement is the sum of the sliding displacements that occurs during the individual acceleration cycles. At the end of the ground shaking, the soil mass will come to a full stop. Therefore, the NDAM cannot predict any relative lateral displacement of the soil mass that occurs, if any, after the cessation of ground shaking.

As per MTD 20-15, using the coefficient of the yield acceleration, $(k_h)_y$, the design ground motions HPGA and the moment magnitude of the associated design earthquake M_w, use the method of Bray and Tavasarou (2007) for Newmark's rigid body type sliding displacement to evaluate the liquefaction-induced lateral spreading displacement (Δ) at bridge abutments:

$$\begin{aligned}\Delta\,(\text{inches}) \;=\; & 0.3937\,\text{Exp}[-0.22 - 2.83\,\text{Ln}(k_h)_y - 0.333\,\{\text{Ln}(k_h)_y\}^2 + \\ & 0.566\,\text{Ln}(k_h)_y\,\text{Ln}(\text{HPGA}) + 3.04\,\text{Ln}(\text{HPGA}) - 0.244\,\{\text{Ln}(\text{HGPA})\}^2 + \\ & 0.278(M_w - 7)] \qquad (1)\end{aligned}$$

Note that the symbols k_y and PGA used in Bray and Tavasarou (2007) are replaced with the symbols $(k_h)_y$ and (HPGA), respectively.

A great deal of interaction between GS and SD is necessary to determine the need for and to perform the lateral spreading analysis. GS needs detailed information from SD on the proposed bridge and foundation design in order to develop a repesentative geometric model (profiles and cross sections) of the abutment ground to be analyzed for lateral spreading and ground displacements. GS needs the bridge design information and related data that SD is required to include with the request for a Foundation Report (FR) as per MTD 1-35 (Caltrans, 2008) and MTD 3-1 (Caltrans, 2014). For existing bridges, As-built plans that include the required abutment geometrics, foundation and other input parameters may be

useful, and sometimes sufficent, to perform an initial or preliminary liquefaction-induced lateral spreading hazard analysis.

Step-By-Step Procedure

For this example, the coordinates of the bridge are assumed to be Latitude = 34.065177^{o} and Longitude = -117.302619^{o}. The design ground motion parameters were evaluated based on a probabilistic seismic hazard analysis (PSHA) and correspond to a return period of approximately 1,000 years. The time-averaged shear velocity V_{s30} for the upper 100 feet soils at the site was estimated to be about 886 feet/sec (270 m/sec). Based on the procedure specified in the SDC and using the Caltrans ARS Online tool (Caltrans, 2018), the following SLE design ground motion parameters apply:

- Coefficient of Horizontal Peak Ground Acceleration, HPGA or $(k_h)_{max}$ = 0.8
- Mean Earthquake Moment Magnitude for the design HPGA, M_w or M = 7.7

Step 1: Develop Geometric Model

The geotechnical lateral spreading hazard analysis starts with the collection and review of the available site and bridge foundation design information needed to develop a geometric model of the abutment soil-wall-pile foundation system (Figure 1).

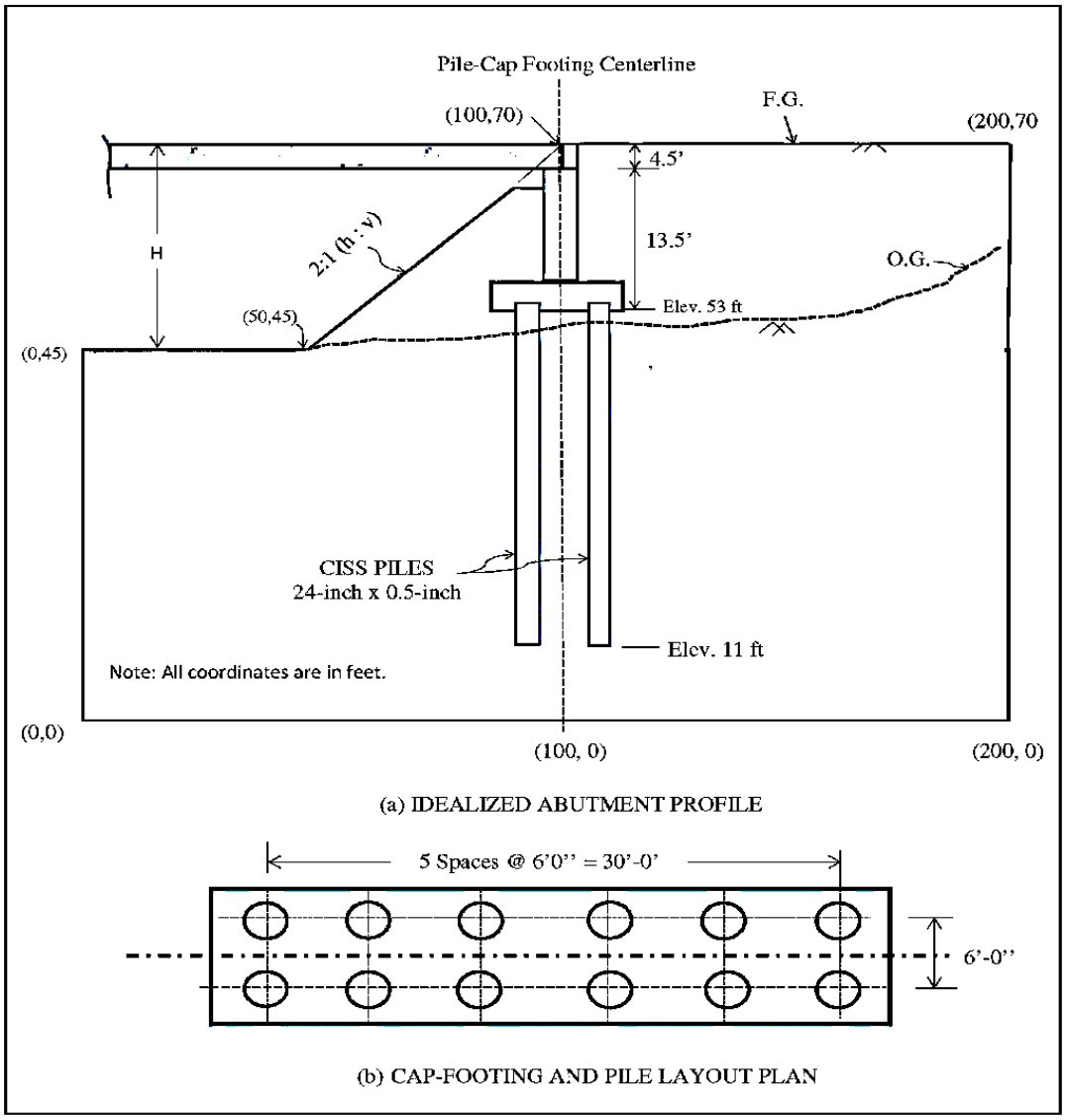


Figure 1. Idealized Geometric Model for Abutment 4

The bridge is a three-span structure with two seat-type abutments (Abutment 1 and 4) and two bent supports (Bents 2 and 3). For this example, liquefaction-induced lateral spreading analysis is performed for Abutment 4. Due to the presence of liquefiable soils, pile foundations are proposed to support the abutment.

Figure 1 shows the original grade (OG) and finish grade (FG) profiles, the proposed 2:1 (Horizontal:Vertical) abutment embankment end slope, the seat-type abutment supported by two rows of 24-inch diameter Cast-in-Steel Shell (CISS) piles. It also shows the proposed layout of the abutment piles.

With reference to Figure 1, the geometric model should extend laterally to the left and right a distance of at least 2H and 4H from the toe and top of the abutment end slope, respectively. Here H is the height of the abutment/embankment as shown in Figure 1. The soil profile should extend to a depth of at least 2H below the toe elevation of the abutment end slope or 3H from the top of the abutment embankment.

Step 2: Develop Idealized Soil Profile with Design Soil Parameters

Figure 2 presents an idealized soil profile at Abutment 4.

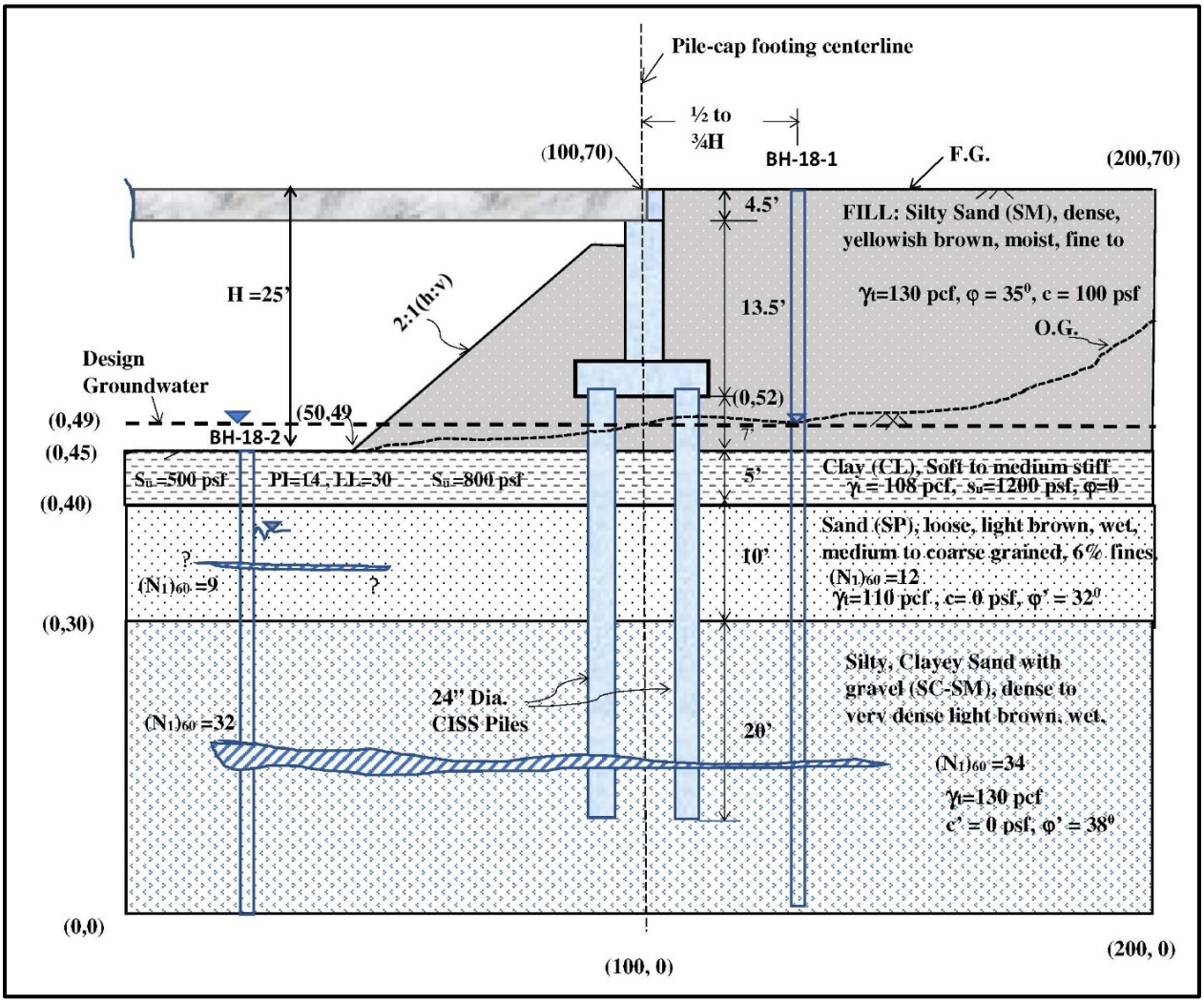


Figure 2. Idealized Geometric and Material Models for Analysis at Abut 4

Research conducted during the early phases of the project development process indicated the likelihood of the presence of potentially liquefiable soils and groundwater, both at relatively shallow depths at the proposed abutment locations. Based on this information, two borings were drilled near each abutment, as shown in Figure 2 for Abutment 4. These boring locations were selected to optimize the subsurface information necessary to develop subsurface soil models for a detailed seismic hazard evaluation, including pseudo-static slope stability analysis. For slope stabilty analysis, borings needed to extend to a depth of at least 2H below the bottom of the approach embankment fill. For the example case, deeper borings were needed for analysis and design of the foundation piles.

Since the presence of potentially liquefiable soil was anticipated, groundwater was measured and SPT blow counts were measured and interpreted in accordance with the ASTM Standard Test Method D6066. The upper portions of the borings were drilled using a hollow-stem auger until groundwater was encountered. Groundwater was allowed to stablize prior to the depth measurements. Below groundwater, the borings were drilled using mud rotary methods.

Respresentative soil samples were tested in the laboratory to determine the soil types and parameters, in particular for the liquefiable soil layer(s). Design soil parameters were selected based on the field exploration and laboratory test results. The design soil parameters for each identified soil layer developed for all applicable loading and/or drainage conditions are presented in Figure 2. The majority of the soil profile and design soil parameters included in Figure 2 were obtained as part of the routine site characterization.

The following sections present brief discussions on the soil parameters needed specificially for the liquefcation-induced lateral spreading analysis.

The undrained residual shear strengths (S_r) for the liquefied soils were based on the empirical correlation developed by Kramer and Wang (2015) and recommended in MTD 20-15 to evaluate S_r for use in the lateral spreading analysis:

$$S_r(\text{psf}) = 2116 \ \exp\left\{-8.444 + 0.109(N_1)_{60} \ + 5.379\left(\frac{\sigma'_{vo}}{2116}\right)^{0.1}\right\} \qquad (2)$$

Where,

$(N_1)_{60}$ = Energy corrected and overburden pressure normalized SPT blow count. See “Liquefaction Module” of the Geotechnical Manual for the procedure to calculate $(N_1)_{60}$.

σ'_{vo} = Initial in-situ effective overburden stress based on the ground surface and groundwater elevations applicable to seismic design (Extreme Event Limit State I).

The correction factor (C_N) used in the evaluation of $(N_1)_{60}$ was based on the effective overburden stress (σ'_{vo}) corresponding to the ground surface and groundwater elevations

existing at the time SPT blow counts (N_m) were recorded. Unlike liquefaction analysis, the SPT blow counts used in Equation (2) are not corrected for fines content.

Unless the piles are not capable of providing any nominal lateral resistance, the potential for significant lateral spreading displacement at piled bridge supports is considered low where the liquefiable soil layers within the depth of significance for lateral stability (i.e., 2H below the top of the abutment embankment) are characterized by $(N_1)_{60}>15$. Soils with $(N_1)_{60}>15$ start to dilate (i.e., provide increased shear resistance) immediately after the onset of limited liquefaction, if any, and do not experience significant lateral displacements during earthquakes.

The undrained residual shear strength (S_r) of the liquefied soils in Equation (2) is a function of both $(N_1)_{60}$ and σ'_{vo}. Due to the variations in the ground surface profile, σ'_{vo} at a given elevation below the ground surface will vary laterally. There may also be lateral variations in the $(N_1)_{60}$ values along the same elevation even for the same soil layer. In other words, within a single liquefied soil layer, the undrained residual shear strength (S_r) may vary both vertically and laterally. This will also be true with regard to the undrained shear strength (S_u) for cohesive soil layers. Therefore, such soil layers present within the depth of influence for lateral stablity, should be divided into sufficient numbers of sublayers (verticially) and subzones (laterally) so that each subzone can be represented by an average value of the undrained shear strength parameter S_r or S_u.

For the example case, the clay as well as the liquefied soil layers in Figure 3 are divided laterally into three subzones and each zone is assigned a single value of the shear strength parameters S_u and S_r. The undrained residual shear strength (S_r) values for three subzones of the liquefied soil are evaluated as follows:

(a) S_r for Zone L1-1:

This zone is located to the left of the toe of the abutment end slope with a flat ground surface at elevation 45 feet. The S_r value at the mid elevation (elevation 35 feet) of this zone is:

$$\sigma'_{vo} = (5.0\text{ ft})(108-62.4)\,\frac{lb}{ft^3} + (5.0\text{ ft})(110-62.4)\,\frac{lb}{ft^3} = 466.0\text{ psf}$$

$$(N_1)_{60} = 9$$

$$(S_r)_{L1-1} = 2116\,exp\left\{-8.444 + 0.109\,(9) + 5.379\left(\frac{466}{2116}\right)^{0.1}\right\}\text{psf} \cong 125\text{ psf}$$

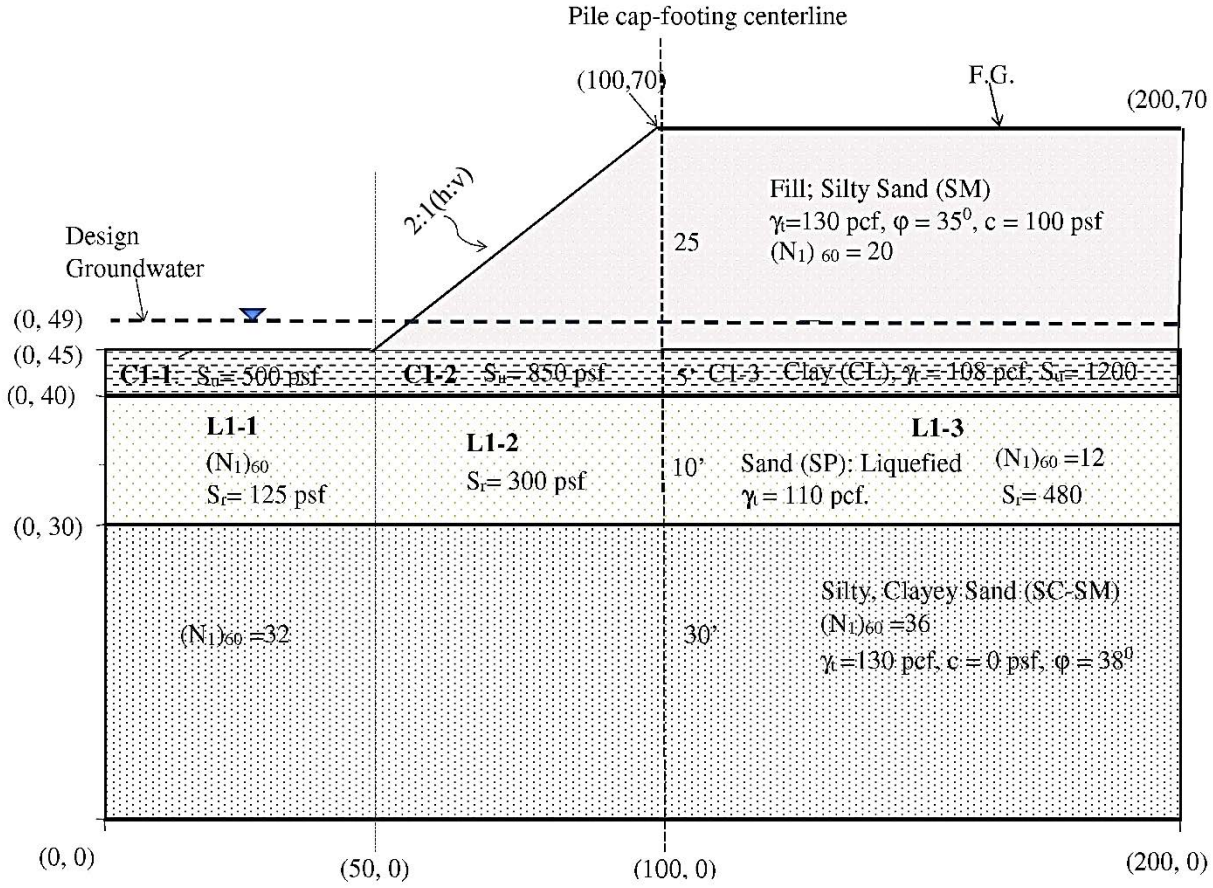


Figure 3. Soil Engineering Parameters for Liquefaction-Induced Lateral Spreading Analysis

(b) S_r for Zone L1-2

This liquefied soil zone is located underneath the steeply sloping ground surface between zone L1-1 and L1-3 (Figure 3). In the absence of any direct SPT blow count measurement (N_m) within this zone, an assumed SPT $(N_1)_{60}$ value equal to the average SPT values for the Zones L1-1 and L1-3 was used in the S_r value. That is,

$(N_1)_{60} = 20$

$$\sigma'_{vo} = (130\ \frac{lb}{ft^3})(12.5 - 4.0)\text{ft} + (130 - 62.4)\frac{lb}{ft^3}\ (4.0\text{ ft}) + (108 - 62.4)\frac{lb}{ft^3}(5\text{ ft}) + (110 - 62.4)\ \frac{lb}{ft^3}\ (5.0\text{ ft}) = 1841.1\ psf$$

Average $(N_1)_{60} = (\frac{9+12}{2}) = 11$

$$(S_r)_{L1-2} = 2116\ exp\left\{-8.444 + 0.109\ (11) + 5.379\left(\frac{1841.1}{2116}\right)^{0.1}\right\}\ \text{psf} \cong 300\ \text{psf}$$

(b) Sr for Zone L1-3

This zone is located to the right of the top of the embankment end slope (i.e., below the full height embankment zone). It is assumed that the abutment end slope surface extends upward to the intersection of the roadway Finish Grade (F.G.) surface and the transverse centerline of the abutment pile cap-footing (Figure 3). The S_r value at mid elevation (Elevation 35 feet) of this zone is:

σ'_{vo} = 130 $\frac{lb}{ft^3}$ (21.0 ft) + (130-62.4) $\frac{lb}{ft^3}$ (4.0 ft) + (108-62.4) $\frac{lb}{ft^3}$ (5.0 ft) + (110-62.4) $\frac{lb}{ft^3}$ (5.0 ft) = 3466.4 psf

$(N_1)_{60}$ = 12

$$(S_r)_{L1-3} = 2116\ exp\left\{-8.444 + 0.109\ (12)\ + 5.379\left(\frac{3466.4}{2116}\right)^{0.1}\right\} \text{ psf } \cong \text{ 480 psf}$$

The undrained shear strengths (S_u) for the three subzones of the clay layer are:

The laboratory measured S_u values for two represenative samples, one each retrieved from boring BH-18-1 and BH-18-2 (Figure 2) are 1200 psf and 500 psf, respectively. The average undrained shear strength for the Zone C1-2 is the average of the undrained shear strengths for the Zones C1-1 and C1-3.

- $(S_u)_{C1-2} = (\frac{500+1200}{2})$ psf = 850 psf

The soil engineering parameters necessary for this analysis are summarized in Table 1.

Table 1. Idealized Soil Profile with Assigned Soil Parameters for Lateral Spreading Analysis and p-y Curves

Soil Layer No.	Top and Bottom Elevations (ft)	Soil Description	Zone[1]	$(N_1)_{60}$	Total Unit Wt (pcf)	Shear Strength Parameters		p-y Parameters	
						Cohesion or Undrained Shear Strength,c, S_u, or S_r(psf)	Friction Angle, ϕ (degrees)	k_s and/or k_c (pci)	ε_{50}
1	70-45	FILL: Silty Sand (SM), Medium dense, from fine-to coarse sand, trace clay, yellowish brown, from moist to wet.	-	20	130	c=100	35	k_s=120/60 (above/below water table) k_c=66	-
2	45-40	Silty Clay (CL), Soft to medium stiff, dark gray, wet (LL=30, PI =10)	C1-1	-	108	S_u=500	ϕ_u=0.0	k_c=280 (Based on avg. S_u=850 psf)	0.01
			C1-2	-		S_u=850[2]			
			C1-3	-		S_u=1200			
3	40-30	Sand (SP), fine and medium, from loose to medium dense, 8-10 % fines, non-plastic, light brown, wet, (Liquefied Layer)	L1-1	9	110	S_r = 125 psf	ϕ_u=0.0	k_c=100	0.02
			L1-2	11[2]		S_r = 300 psf			
			L1-3	12		S_r = 480 psf			
4	30 to 0 [3]	Silty, Clayey Sand (SC-SM)with gravel (SM), dense, from fine to coarse sand, fine gravel, trace clay, grayish brown, wet	-	32-36	130	c=0	38	k_s=125	-

Notes: (1) See Figure 3 for locations of soil layer zones, (2) Estimated, and (3) Soil profile for lateral spreading analysis should extend to a depth of at least 2H below the bottom of the slope. Here, H= Height of the embankment.

Step 3: Perform GLE-Based Pseudo-Static Slope Stability Analysis

The step involves a series of General Limit Equilibrium (GLE) based pseudo-static slope stability analyses performed using the models in Figures 1 through 3.

Step 3.1: Select Lateral Spreading (Sliding) Mechanism

Caltrans MTD 20-15 specifies a 2D wedge type lateral spreading mechanism. For a given value of the pile lateral resistance (R_{Tot}) and seismic inertial loading (k_h), a set of pseudo-static analyses (runs) are performed varying the slip surface to evaluate the minimum factor of safety (FS_{min}).

MTD 20-15 imposes a number of important constraints on the horizontal as well as the vertical limits of the potential sliding mass when analying lateral spreading hazards (See Figure 4 of MTD 20-15 for details).

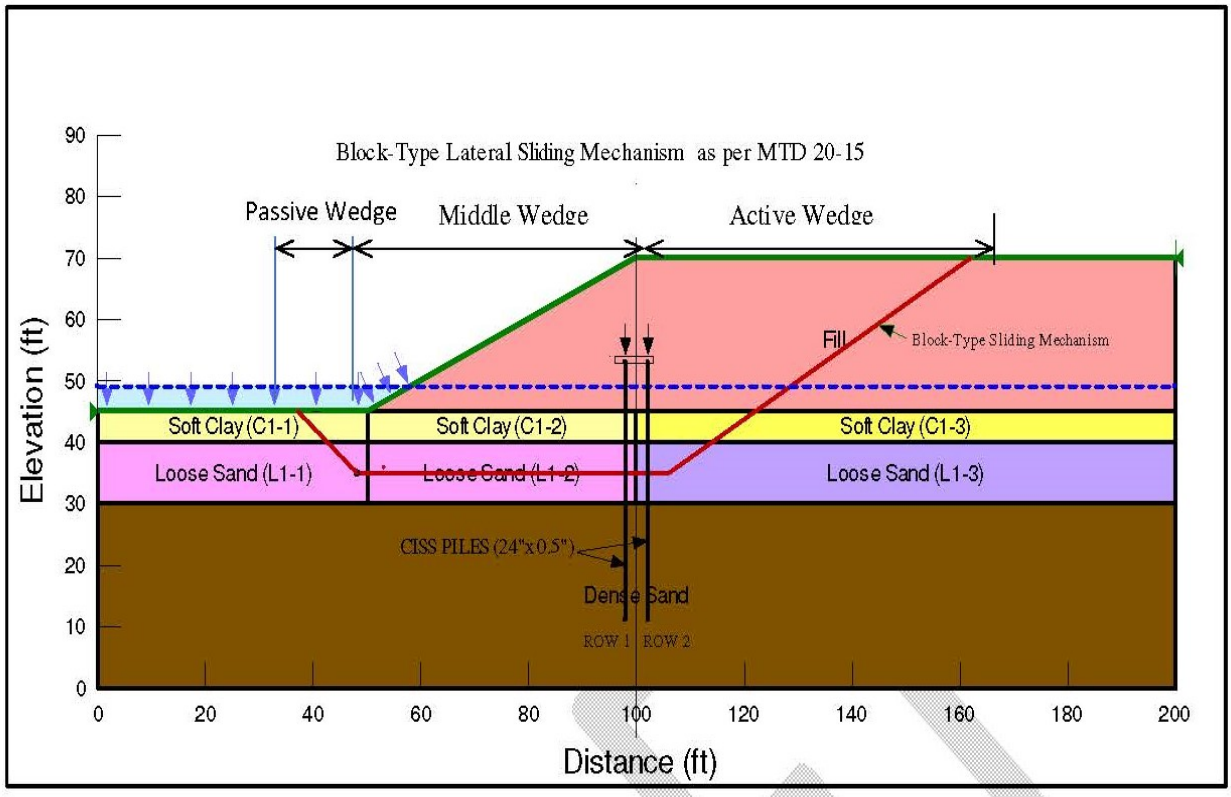


Figure 4. Digital SLOPE/W Model for the Example with Typical Wedge Type Sliding Mechanism

Figure 4 shows a typical geometry of the wedge-type sliding mass used in this example. The potential sliding consists of three wedges: (1) an Active Wedge, (2) a Middle Wedge, and (3) a Passive Wedge.

Step 3.2: Select Pseudo-Static Stability Analysis Method and Computer Software

A GLE-based slope stability analysis procedure should be used for the evaluation of pseudo-static slope stability analyses of wedge-type and composite abutment soil-foundation sliding masses. Geo-professionals performing lateral spreading analysis should be thoroughly familiar with the computer software used to perform pseudo-static slope stability calculations. The results of such analyses must be verified as reasonable.

For this example, the GLE-based Morgenstern-Price procedure for slope stablity analysis was used to perform all pseudo-static slope stability analyses using SLOPE/W (GEO-SLOPE, 2014). The Spencer procedure may also used for this analysis.

Step 3.3: Develop Digital Slope Stability Model Including Pile Lateral Resisting Forces

Develop a digital computer software model based on the idealized geometric, soil and foundation models shown in Figures 1 through 3. Figure 4 presents the digital model developed in SLOPE/W. To use pile lateral resistances as shown in Figure 3 of MTD 20-15, the abutment pile foundations were modeled in SLOPE/W as built-in 1-D vertical “reinforcement elements”.

As per MTD 20-15, the Structure Designer (SD) performs a soil-foundation interaction analysis at the abutment to determine the mobilized values of the total pile lateral resistance ($R_{Tot)}$ as a function of lateral ground displacement (Δ). See Figure 1(b) of MTD 20-15 for the definition and distribution pattern of the displacement parameter (Δ) along the pile. This information is sent to GS in the form of Curves 1 and 2 shown in Figure 5 of the MTD 20-15. GS uses this information to select the the appropriate range (R_{Tot}) of values to perform the geotechnical lateral spreading analysis in order to obtain Curve 3 shown on Figure 5 of the MTD 20-15.

In SLOPE/W, for each of the pile “reinforcement” element as shown in Figure 4, the input parameters for pile lateral resistances include:

- Pile length (L), in unit of length
- Direction (θ), in degrees counterclockwise from the (horizontal) slip direction
- Shear force (V_c) in unit of force per pile
- Shear force reduction factor (f_g), dimensionless factor
- Pile spacing parameter, S (in unit of length measured in the direction normal to horizontal/slip direction), and
- Direction of the shear force (either parallel to slip or perpendicular to the reinforcement)

For N rows of piles (normal to the slip direction), N individual pile reinforcements, each representing a single row of piles in the normal direction to sliding, should be used in SLOPE/W. In a SLOPE/W analysis, the total pile lateral resistance R_{Tot} (force per unit width of abutment) may be equally divided between the N pile reinforecements or vice versa. That is, for each vertical pile reinforcement element shown in Figure 4:

$$V_c = \frac{(R_{Tot})\,S}{N} \qquad (3)$$

For the example, there are two (2) rows of piles (N=2). The input parameters for each pile in Figure 4, for a given value of R_{Tot} in kip/ft width of abutment, are as follows:

- Pile length (L) = 47 feet
- Direction (θ) = 90°
- Pile spacing, S = 6.0 feet
- Shear force $V_c = (\frac{R_{Tot} \text{ x } 6.0}{2})$ = 3.0 x R_{Tot} (kips per pile)
- Shear force reduction factor (f_g) =1.0
- Direction of the shear force = parallel to slip

Step 3.4: Perform Pseudo-Static Slope Stability Analysis for R_{Tot} = 0.0 and k_h=0.0

Initial or preliminary forms of this analysis will usually be performed early in the project development, e.g., during the planning and preliminary engineering phases. The purpose of this analysis is to assess if the potential for liquefaction-induced lateral spreading, including flow type failure, exists at the site, and most significantly to determine the location of the potential sliding surface.

Perform a pseudo-static slope stability analysis for the abutment wall-soil-foundation system based on the digital model developed in Step 3.3 using:

- R_{Tot} = 0.0 kips/ft (or V_c=0.0 and S=1.0 as the input parameters)
- k_h = 0.0

The FS_{min} values obtained from this and the following step will provide a qualitative indication of soil lateral capacity available for the liquefied conditions to resist lateral sliding and valuable information on the potential sliding mass, including the locations of the critical slip surface and its intersection with the piles, and the extent of additional lateral support that may be needed from existing and/or new piles.

For example, a FS_{min}<1.0 indicates that, in the absence of any additional lateral support (e.g., from piles), the soil-mass analyzed is susceptible to liquefaction-induced lateral flow. Lateral flow involves large movements as stated in MTD 20-15. It can occur during ground shaking after soil liquefaction starts, and also immediately after the cessation of shaking. In the case of an existing pile-supported abutment, it also indicates that a re-evaluation of the lateral flow hazards will be necessary by including the additional lateral

capacity obtainable from the pile(s). In the case of new construction, it provides qualitative information on the need for, and the extent of, additional lateral supports such as piles or ground improvements. The lower the value of FS_{min} compared to 1.0, the higher is the likelihood for lateral spreading and larger ground displacement, requiring more extensive countermeasures.

For this example, a range of wedge-type slip surfaces were analysed using SLOPE/W by specifying a left and a right block of grid points (grid-block) to locate the intersection points of the slip planes associated with the three wedges as shown in Figures 4 and 5. The lower boundary lines for these grid-blocks were specified to be at the same elevation as the mid-point of the liquefied soil. This is to force the critical sliding surface within the middle wedge to be located at the mid-depth of the liquefiable soil layer as specified in MTD 20-15. Results of this analysis are presented in Figure 5.

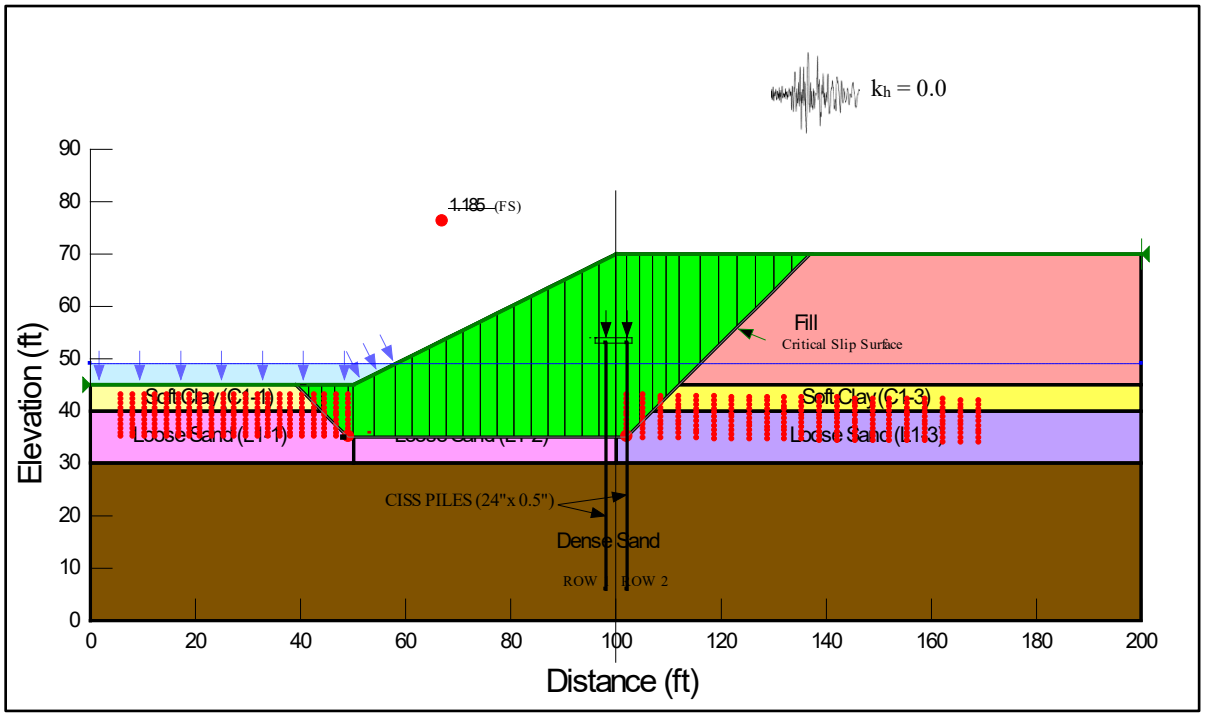


Figure 5. Results of Slope Stabilty Analysis for R_{Tot} = 0.0 lbs/ft width and k_h=0.0

As seen in Figure 5, FS_{min} = 1.18 for these conditions. Therefore, this abutment is not considered susceptible to liquefaction-induced flow failure.

Step 3.5: Perform Pseudo-Static Slope Stability for k_h= HPGA or $(k_h)_{max}$

Determine if the available soil lateral capacity alone is sufficient to support the abutment wall-soil-foundation system. This step consists of performing a pseudo-static slope stability analysis for the following conditions:

- R_{Tot} = 0.0 kips/ft (or V_c=0.0 and S=1.0 as the input parameters)
- k_h=HPGA =0.8

Results of this analysis are presented in Figure 6, which indicates a very low FS_{min} =0.2.

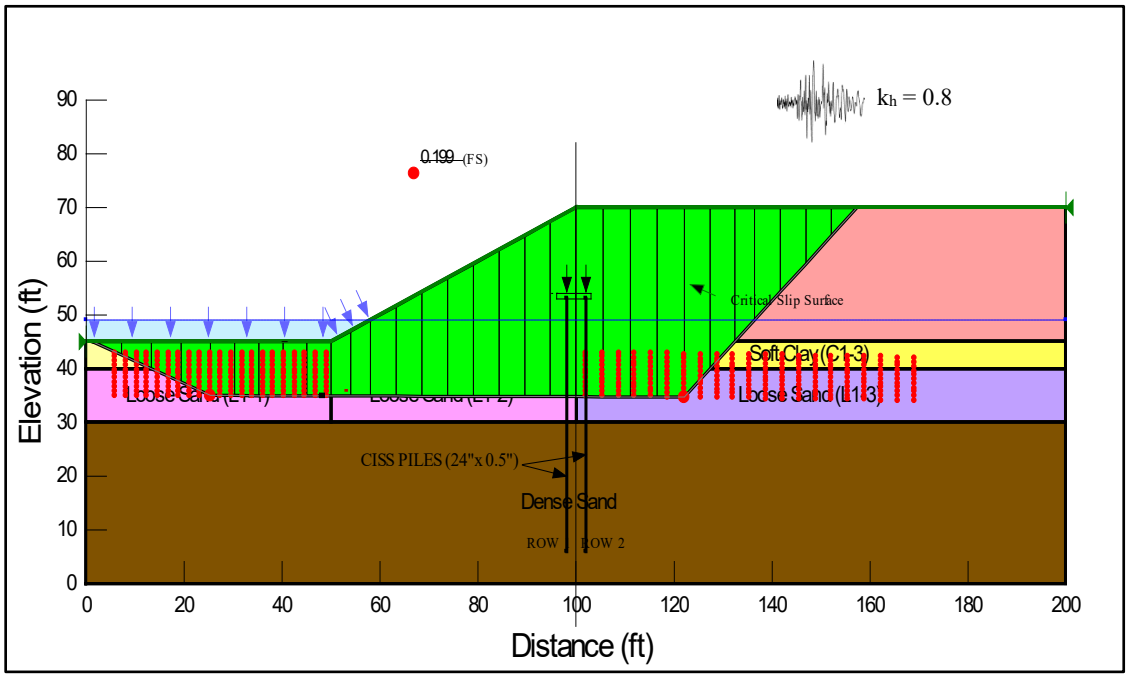


Figure 6. Pseudo-Static Slope Stabilty Analysis for R_{Tot} = 0.0 lbs/ft and k_h=$(k_h)_{max}$

If this analysis indicates FS_{min} ≥1.0, the subject abutment will not be considered susceptible to lateral spreading hazards. No further lateral spreading analysis is necessary. Communicate this information to SD, and present the results and findings in the Foundation Report (FR).

Otherwise, continue with the lateral spreading hazard analysis.

Step 3.6: Evaluate Coefficient of Yield Accelerations $(k_h)_y$ for R_{total} =0.0

Evaluate the coeffcient $(k_h)_y$ for the abutment without considering any contribution from piles to the lateral resistance, and determine the critical slip surface for the above conditions.

This step consists of performing a series of pseudo-static slope stability analyses with input R_{Tot} (or V_c) = 0.0 into the digital model in Figure 4, and varying the input value for k_h. All other input parameters, except the limits of the specified left and right search grid blocks, remain the same. The lateral positions of the failure plane intersection are likely to vary with the input k_h values as well as the input R_{Tot} values. Therefore, it may be necessary to extend outward the lateral limits of the left and right search grid blocks.

As per MTD 20-15, the outer limits of the intersectipn of the active wedge sliding plane and the ground surface does not need to extend beyond a distance of 4H from the back of the abutment wall, where H is the height of the abutment.

Results of two typical slope stability analysis runs, for R_{Tot} = 0.0 kips/ft and k_h=0.05 and 0.125 are presented in Figures 7(a) and 7(b), respectively. Notice the differences in the size and limits of the critical slip surfaces for the two different cases.

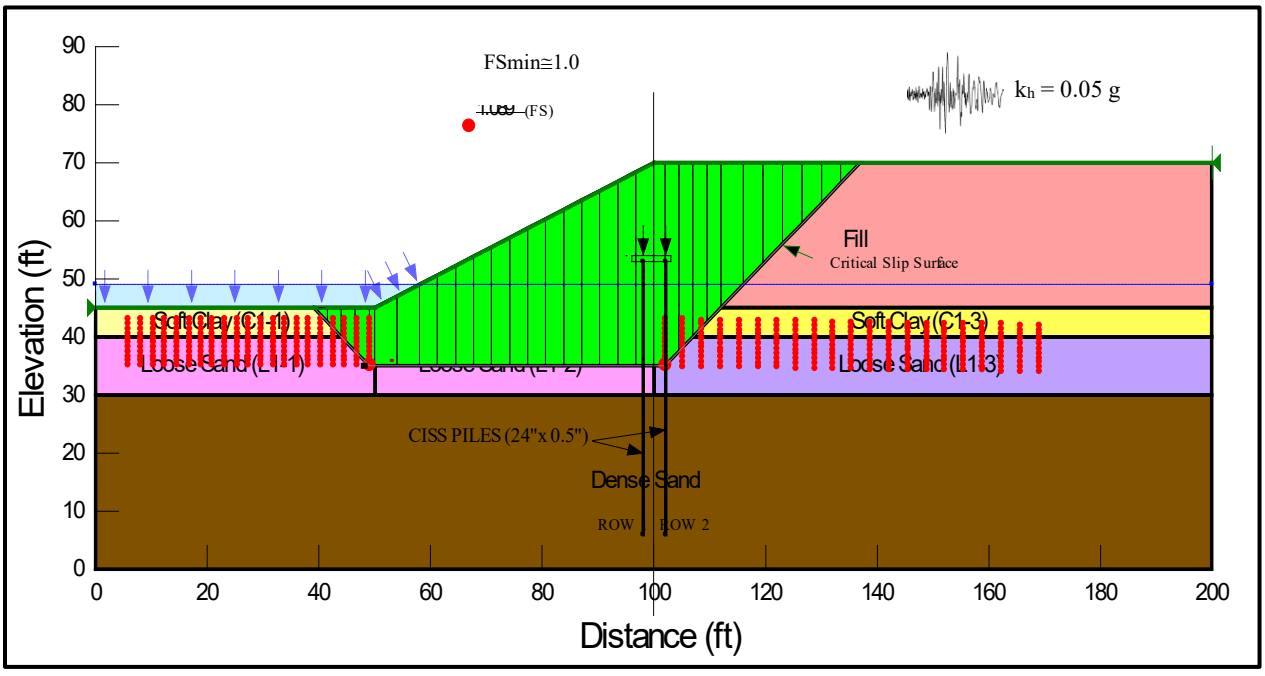


Figure 7(a). Results of Pseudo-Static Stability Analyses for R_{Tot}=0.0 and k_h=0.05

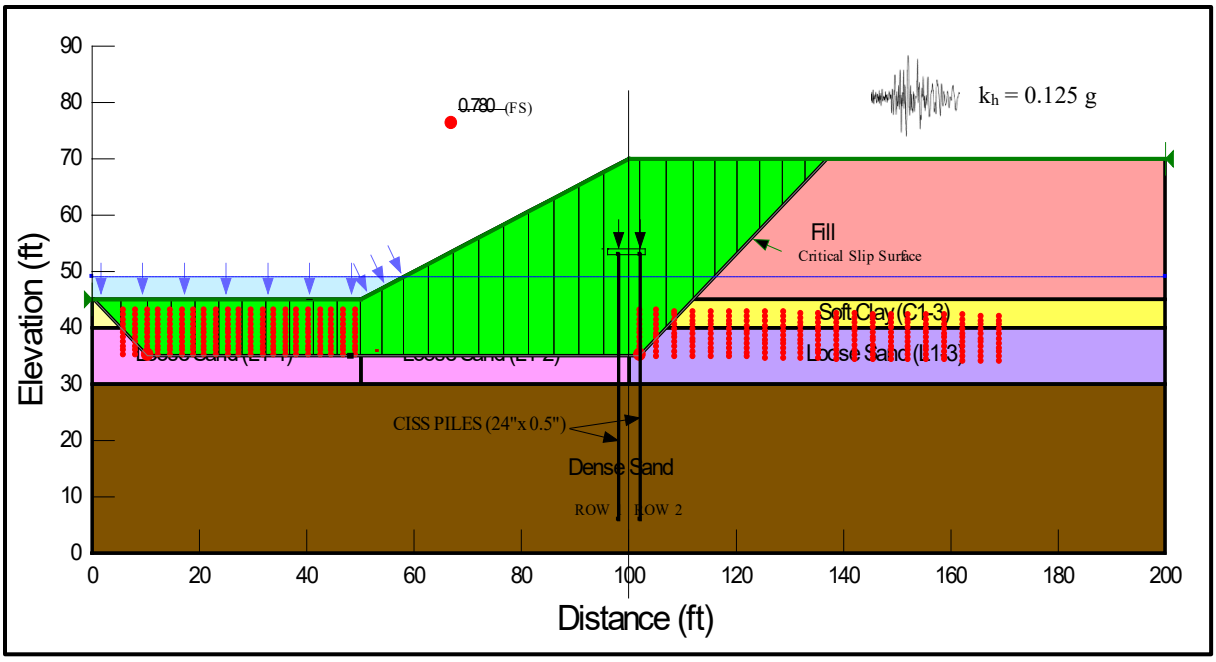


Figure 7(b). Results of Pseudo-Static Stability Analyses for R_{Tot}=0.0 and k_h=0.125

For each input value of k_h, the outcome of the pseudo-static slope analysis is a corresponding FS_{min} value. The FS_{min} values were obtained by repeating this analysis, varying the input k_h value while keeping the input R_{Tot} value the same (= 0.0, in this case). Results of this analysis are in Table 2 and plotted in Figure 8.

Table 2. FS_{min} Values for R_{Tot} =0.0 kips/ft and Different k_h Values

R_{Tot} (or V_c) =0.0 kips/ft	
k_h	FS_{min}
0.000	1.184
0.025	1.088
0.050	1.003
0.075	0.924
0.10	0.848
0.125	0.780
0.150	0.601

The $(k_h)_y$ value for the current case (e.g., R_{Tot}=0.0) is evaluated from Figure 8 as equal to the k_h corresponding to a minimum FS_{min}=1.0. As shown by the arrows in Figure 8, for FS_{min}=1.0, k_h=0.05. Thus, $(k_h)_y$ =0.05 for R_{Tot}=0.0.

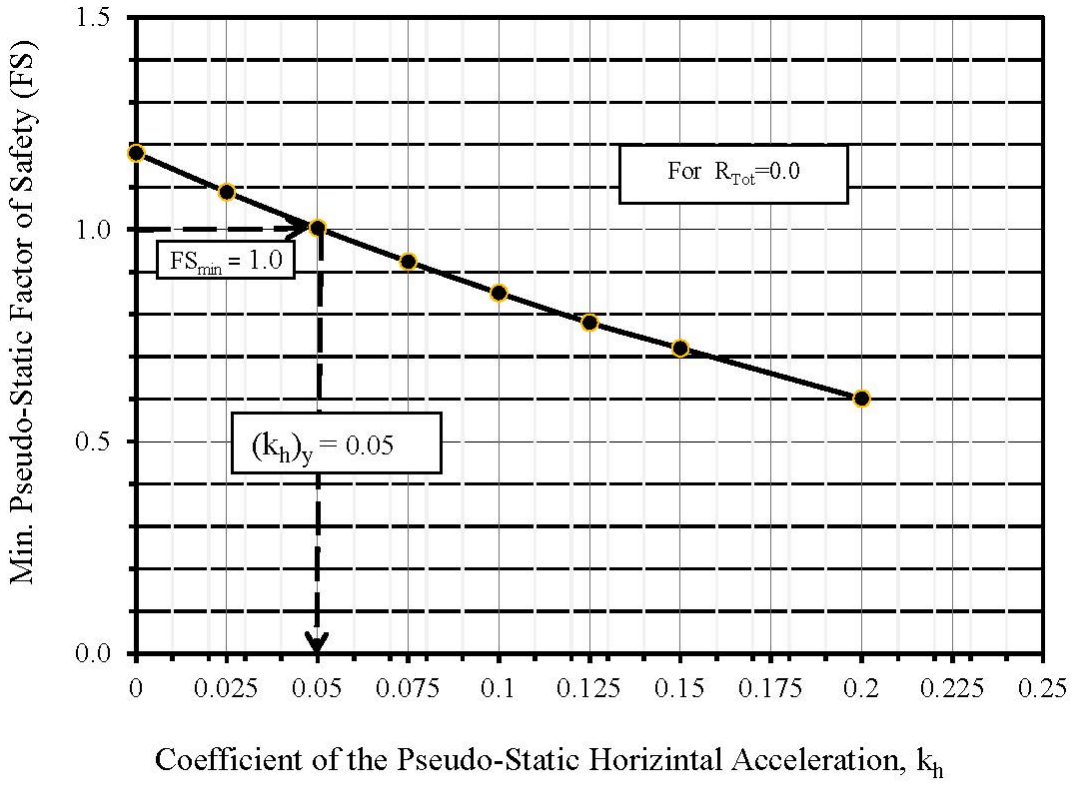


Figure 8. Coeffcient of Horizontal Yield Acceleratаion $(k_h)_y$ for R_{Tot}= 0.0 kips/ft

Step 4: Interim Communication with the Structure Designer

Send SD the following information, and continue with the lateral spreading analysis:

- Table 1, including (if not provided already) the recommended liquefied soil profile and soil parameters necessary to perform p-y type laterally loaded pile analysis.
- Figure 7(a) which shows the locations where the piles intersect the critical sliding surface for $k_h=(k_h)_y$. SD will utilize this information in the soil-foundation interaction analysis as per MTD 20-15 to determine R_{Tot} as a function of the displacement (Δ).

Step 5: Obtain/Select an Input Range and the Discrete R_{Total} Values

Based on the geotechnical information provided in Step 4, SD will perform a soil-foundation interaction analysis for the proposed pile type, size, length and pile group layout shown in Figure 1. Based on this analyis, SD evaluates R_{Tot} values for a range of assumed displacement (Δ) values. Results are presented in Figure 9.

Based on Figure 9, the total ultimate or nominal pile lateral resistance per unit width of the abutment, designated here as $(R_{Tot})_N$, is about 28 kips/ft. A range of input R_{Tot} =0.0 to 60 kips/ft is considered adequate for further pseudo-static slope stablity evaluation. The maximum value for input R_{Tot}= 60 kips/ft is selected to be about two (2) times the $(R_{Tot})_N$ in Figure 9.

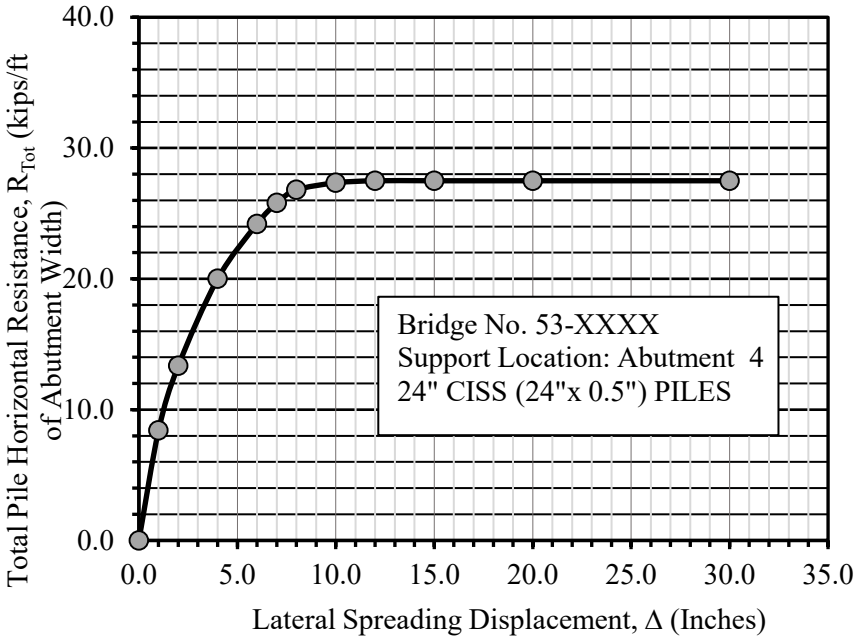


Figure 9. Plot of R_{Tot} Versus Lateral Spreading Displacement (Δ)

Based on the above range of input (R_{Tot}), a set of discrete R_{Tot} values included in Table 3 were selected for additional pseudo-static slope stability analyses. These numbers were selected to obtain a sufficient number of equally spaced discrete R_{Tot} values for the range of 0.0 to 60 kips/ft. The input V_c values in the SLOPE/W model are presented in Table 3.

Table 3. Selected Input R_{Tot} Values and the Corresponding V_c Values

R_{Tot} (kips/ft)	Vc (kips /Pile)[1]
0.0	0.0
5.0	15.0
10.0	30.0
20.0	60.0
30.0	90.0
40.0	120.0
50.0	150.0
60.0	180.0

Note 1: For two rows of piles and pile spacing parameter S=6.0 ft.

Step 6: Evaluate Coefficient of Horizontal Yield Accelerations $(k_h)_y$ Each R_{Tot} Value

Conduct pseudo-static slope stability analyses (similar to Step 3.6) for each R_{Tot} value in Table 3 to evaluate the corresponding yield acceleration. Inherent in this analysis for pile-pinning effects is the assumption that, for the soil-pile system, there exists a yield acceleration for each mobilized R_{Tot} value. Based on these analyses, plot a series of curves for each value of R_{Tot} , as shown in Figure 1, each similar to the one shown in Figure 8 for R_{Tot}=0.0.

The $(k_h)_y$ value for each R_{Tot} are then determined from the corresponding plot in Figure 10 as equal to the k_h for FS_{min}=1.0. The $(k_h)_y$ values are presented in Table 4.

Table 4. Yield Coefficient $(k_h)_y$ for Various R_{Tot} Values

R_{Tot} (kips/ft)	$(k_h)_y$
0	0.05
5	0.08
10	0.10
20	0.13
30	0.15
40	0.17
50	0.19
60	0.21

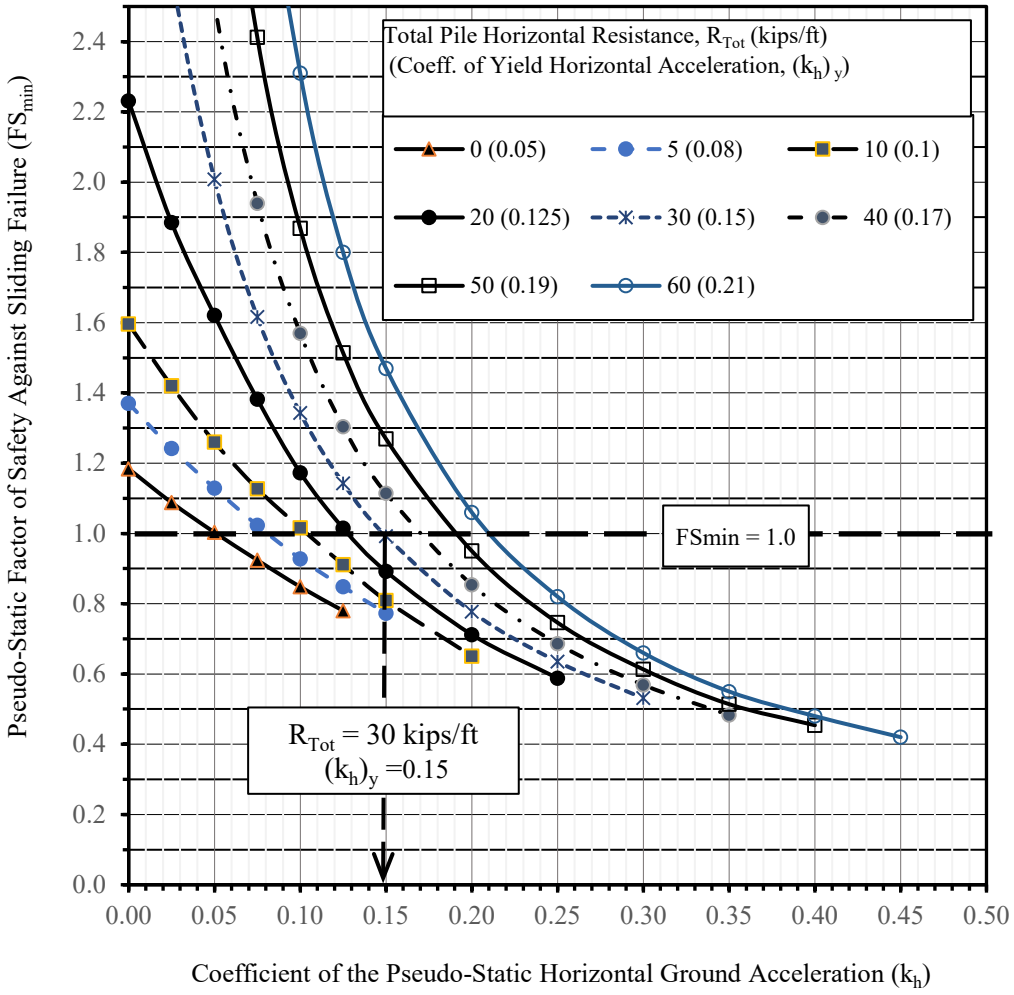


Figure 10. Plots of FS_{min} as a Function of k_h for Various R_{Tot} values

Step 7: Determine Newmark Displacements as a Function of $(k_h)_y$

Calculate the median Newmark's rigid body type sliding displacement of the soil-mass due to the design ground motion for each $(k_h)_y$ value in Table 4 using the empirical correlation (Eq. 1) by Bray and Tavasarou (2007), HPGA=0.8 and M_w=7.7.

Results are presented in Table 5.

Table 5. Lateral Spreading Displacement (Δ) as a Function of R_{Tot}

R_{Tot} (kips/ft)	$(k_h)_y$	Displacement, Δ (inches)
0	0.05	172.8
5	0.08	102.2
10	0.10	75.6
20	0.13	50.9
30	0.15	40.2
40	0.17	32.4
50	0.19	10.4
60	0.21	8.6

Step 8. Plot R_{Tot} versus Displacement (Δ)

Each $(k_h)_y$ value in Table 5 and the corresponding value of the estimated median ground displacement (Δ), corresponds to a specific value of R_{Tot} (See Table 4). Plot the R_{Tot} versus Displacement (Δ) as shown in Figures 11 and 12.

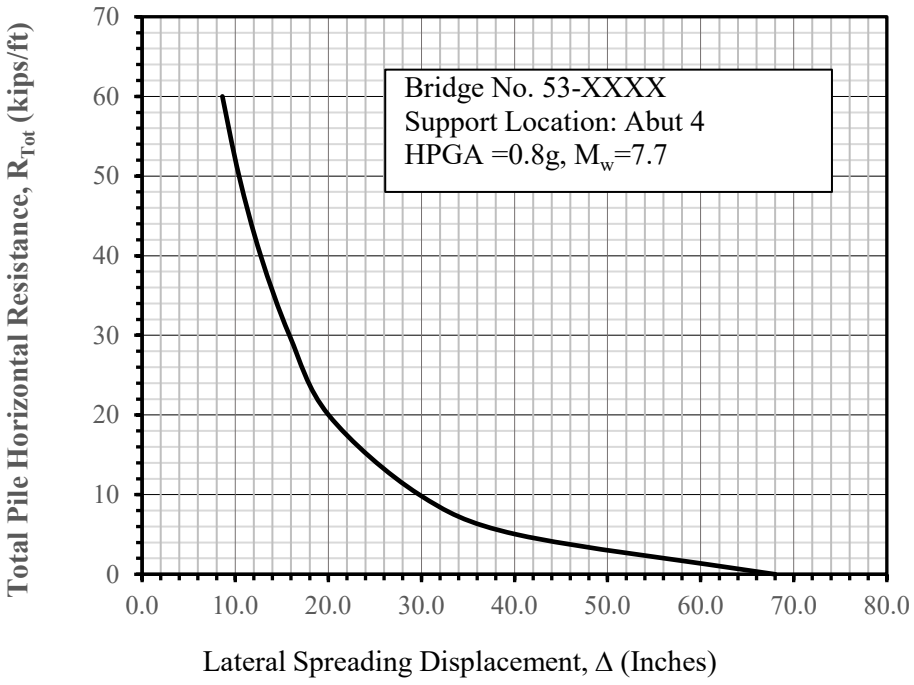


Figure 11 . R_{Tot} vs. Newmark Rigid Body Displacement (Δ)

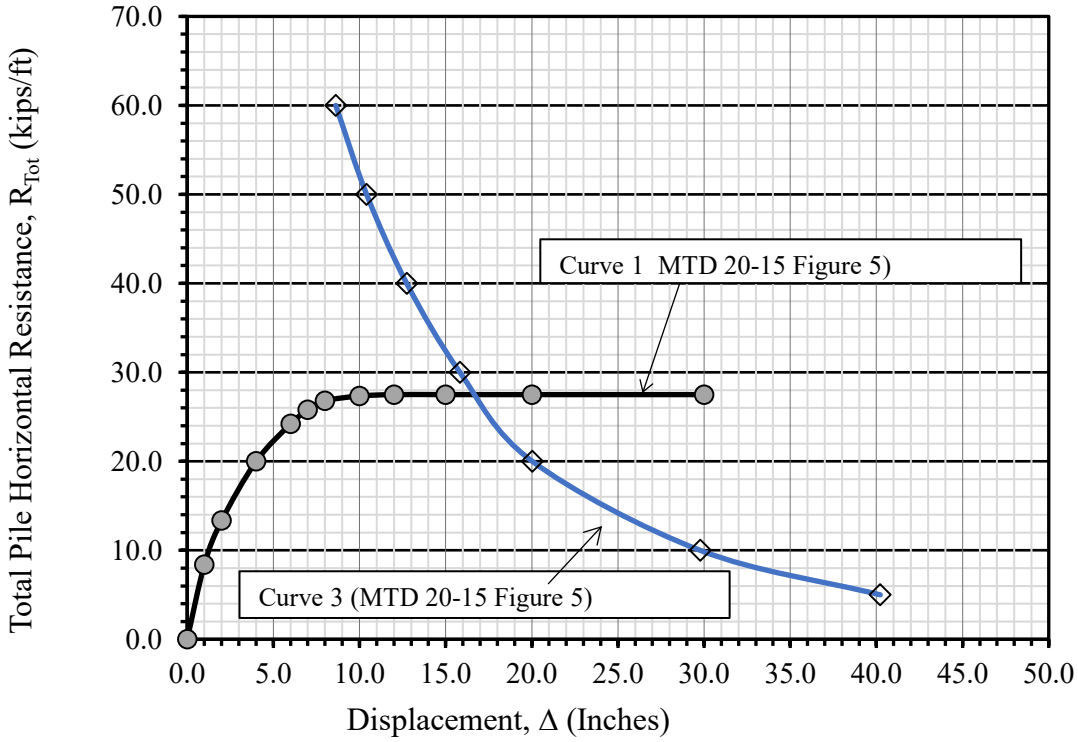


Figure 12. Interaction Curves (Corresponding to Fig. 5 of MTD 20-15)

Step 9: Send Results of Lateral Speading Analysis to Structure Designer

Send SD the results of the geotechnical lateral spreading analysis presented in Table 5 and Figure 11 or Figure 12. The SD will use this information to plot Curve 3 in Figure 5 of MTD 20-15 with Curves 1 and 2. SD will determine the "compatible" liquefaction-induced lateral ground displacement (Δ) at the abutment from the intersection of Curve 2 and Curve 3 in Figure 5 of the MTD 20-15. SD will evaluate these findings and inform GS if the predicted compatible lateral spreading displacement is acceptable or design modifications and additional analysis will be needed.

Step 10: Prepare and Submit Report

Once SD confirms that no further geotechnical lateral spread analysis is necessary, report the findings to SD either by incorporating the above analysis into the Foundation Report or preparing a separate technical memorandum. A draft report may be submitted to SD for review and comments. Prepare and submit a final report by addressing the review comments, if any, from SD.

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Conventional Retaining Walls

This module presents the Caltrans standard practice for the geotechnical investigation, design, and reporting for conventional retaining walls. Conventional retaining walls are rigid gravity and semi-gravity retaining walls as defined in AASHTO LRFD BDS Section 11.2. Conventional retaining walls are reinforced concrete walls in the shape of an "L" or inverted "T". Conventional retaining walls commonly used by Caltrans are Retaining Wall Types 1, 5, and 6 as detailed in the Caltrans Standard Plans and Caltrans Revised Standard Plans, and Retaining Wall Type 7 as detailed in the Caltrans Bridge Standard Detail Sheets. Conventional retaining walls are also any variation or modification of these retaining walls that may include piles, sound walls, barriers, or enlarged foundations.

In addition to this module, the documents that guide or govern the investigation, design, and reporting for conventional retaining walls include:

- AASHTO LRFD Bridge Design Specifications, (Eighth Edition) with California Amendments
- Caltrans Standard Plans and Revised Standard Plans
- Geotechnical Manual, *Soil Correlations*
- Geotechnical Manual, *Foundation Reports for Earth Retaining Systems*
- Geotechnical Manual, *Geotechnical Design Reports*
- Geotechnical Manual, *Geotechnical Design Using standard Plan and Bridge Standard Detail Sheets*
- Caltrans Memo to Designers (MTD) 5-19, *Earth Retaining Systems Communication*
- Caltrans Memo to Designers (MTD) 5-5, *Design Criteria of Standard Earth Retaining Systems*
- Caltrans Bridge Design Details 1-16*, Use of Bridge Standard Detail Sheets (XS-Sheets)*
- Caltrans Bridge Design Aids (BDA) 3-7, *Pile Layouts for Standard Plan Retaining Wall*s
- Caltrans Bridge Design Aids (BDA) 12-2, *Permissible Horizontal Load for Standard Plan and Steel HP Piles*
- Caltrans Highway Design Manual, *Chapter 200 Geometric Design and Structure Standards*

Retaining wall types detailed in the Caltrans Standard Plans are commonly designed through a coordinated effort between the Geoprofessional and District Design Engineer; however, upon special arrangement, Structure Design or consultant engineers may act in place of the District Design Engineer. Retaining Wall Type 7 and varied or modified Standard Plan retaining walls are designed through a coordinated effort between the Geoprofessional and Structure Design or consultant engineers.

The Geoprofessional assists in all phases of project development as requested by District or Structure Design. This assistance may include research, preliminary retaining wall design, and type selection during the project planning or early design phase; field investigation, analyses, external retaining wall design, and design support during the project design phase; and construction support and possible retaining wall alterations due to project changes or unforeseen conditions discovered during the construction phase. The geotechnical effort must be documented and communicated in appropriate reports and memorandum.

Conventional retaining walls are typically type-selected during project planning or early design phases through the coordinated effort between the Geoprofessional and project development staff. A more formal type selection process may be conducted for some retaining walls, as may occur for complex projects or wall sites. In such cases, the type selection should be based on preliminary geotechnical reports addressing the retaining walls. A thorough discussion of retaining wall type selection may be found in Chapter 10 of the Federal Highway Administration Publication No. FHWA-NHI-07-071, *Earth Retaining Structures Reference Manual*.

Investigations

A geotechnical investigation must be conducted for all retaining walls. The goal of the geotechnical investigation for conventional retaining walls is to determine the distribution, properties, and behavior of the soil and rock that will affect retaining wall design and construction; the groundwater condition that will affect retaining wall design and construction; the distribution of unsuitable or weak materials requiring remedial measures; and the suitability of excavated soil to be used as embankment fill or structure backfill. In addition to permanent features, the investigation should gather data useful in the evaluation of temporary construction features such as excavations and shoring.

The geotechnical investigation must provide data to determine the:

- strength and settlement characteristics of foundation soils
- strength and weight of soils to be retained
- strength and unit weight of soils affecting slope stability
- corrosion potential of soils in contact with the retaining wall
- groundwater location
- qualitative assessment of groundwater seepage

All foundation soils, including fills, must be investigated. Refer to the *Geotechnical Investigations* module for direction on performing a literature search. In some instances, the information obtained through the literature search and field mapping may be sufficient for retaining wall design. Examples of such instances are walls built in “layer cake” sedimentary strata where nearby borings exist for the same sedimentary

units, or walls founded on rock with abundant rock exposures and where previous testing is adequate to sufficiently characterize the rock.

The Geoprofessional should develop an exploration plan considering site constraints and available resources, and consider the uncertainty and risk of not drilling at a particular location. The Geoprofessional should:

- Obtain retaining wall layout and configuration as accurately as possible. Final wall layout and height may not be determined until late in the design phase.
- Perform a literature search. Gather all relevant information related to site geology, geologic hazards, subsurface conditions, and soil and rock engineering parameters.
- Perform geologic field mapping of the wall site. The mapping should be sufficient to generate geologic cross sections along the retaining wall alignment when combined with other terrain data.
- Develop a subsurface exploration and laboratory testing plan to augment information gathered through archive research and field mapping. Space exploratory borings, Cone Penetration Test (CPT) soundings, and/or drive holes at maximum intervals of 100 to 200 feet along the proposed wall alignment, with borings strategically positioned in front, behind, and directly on the retaining wall layout line. The number of borings necessary to delineate site conditions may be reduced or increased due to the value of pre-existing data, uniformity of site geology, and the quality of site specific geologic mapping.
- Where shallow foundations are anticipated, advance the subsurface exploration to an appropriate depth, which should generally extend below the foundation to the deepest of:
    - 15 feet,
    - twice the height of the retaining wall,
    - 4 times the estimated footing width, or
    - to the full depth of soft, loose, weak soils upon which wall stability, bearing resistance, and settlement is dependent
- Conduct Standard Penetration Test (SPT) at maximum depth intervals of 5 feet. Closer intervals of SPT testing should be considered within a depth of 2 times the footing width below the proposed bottom of footing (the zone of greatest bearing pressure), and where soil strength properties are anticipated to be soft or loose.
- Conduct consolidation testing of clay soils wherever settlement magnitude and rate are significant project considerations.
- Where deep foundations are anticipated, refer to either the *Driven Pile Foundations* module or the *CIDH Pile Foundations* module.
- Gather data to evaluate the stability of permanent and temporary excavations and cut slopes that will influence design and construction of the retaining wall.

All material within the active wedge (i.e., retained zone) must meet the minimum strength assumed for structure backfill (friction angle, Φ = 34 degrees).

- Estimate soil strengths based on index properties established through SPT, pocket penetrometer, torvane, and CPT (see Soil Correlations Module). For retaining walls founded on Intermediate Geo Materials (IGM) or rock, strengths may be sufficiently estimated by reviewing data developed for similar rock on nearby projects. Perform laboratory strength tests only when correlation-based strengths result in borderline acceptable or unacceptable calculation results.
- Conduct corrosion testing on representative samples of the soil that will contact the retaining wall. Evaluate and interpret the collected data to arrive at reasonable assessments of corrosion potential. For example, if some samples gathered from a single sedimentary stratum or formational unit are found to be corrosive and other samples are found to be non-corrosive, the entire sedimentary stratum or formational unit should be deemed corrosive. If only a small zone of a formational unit appears to be corrosive, attempt to ascertain why only that zone should be regarded as corrosive.
- Sample and test mandatory borrow sites to determine if the material satisfies corrosion and gradation criteria for structure backfill. Sample and test project cut excavations to determine if material generated on-site will meet structure backfill requirements and should be designated for use as such. Conventional retaining walls may be constructed atop fills that do not exist at the time of the investigation but will be placed during the project. If the material borrow site is known the site should be investigated to determine soil properties useful in further evaluations.

Design

The design of conventional retaining walls must follow the Geotechnical Manual and AASHTO LRFD BDS. The design must address strength, service, and extreme event limit states.

The walls in the Standard Plan and Bridge Standard Detail sheets (known as XS sheets) have been designed for sliding, deflection, eccentricity and internal structural stability requirements for the specific retained soil strength listed on the plan sheets. The Geoprofessional must evaluate the site soil to determine if the conditions meet the minimum strength and stability criteria provided on the Standard Plan and the XS sheets.

The geotechnical design of a conventional retaining wall must include consideration of:

a. The design soil/rock profile
b. Factored Gross Nominal Bearing Resistances (Strength and Extreme Event Limit States)
c. Frictional resistance of foundation material for sliding analysis

d. Total and differential settlement
e. Overall slope stability (Service and Extreme Event Limit States)
f. Erosion susceptibility and mitigation
g. Seismic stability
h. Surface and subsurface drainage systems
i. Foundation improvement requirements
j. The minimum unbonded ground anchor length for Type 7 walls that incorporate ground anchors

Design Soil Profile

Use the geologic information to develop initial design soil profile(s) along the wall layout line, which may be revised to reflect ground improvement in the case that the design parameters are not met. Use of a table is recommended to summarize the design soil profile.

Example: Design Analysis Soil Parameters

Layer No.	Layer Boundaries	Group Name	Engineering Parameters
1	Finished grade to elev. 300	Silty Sand (fill)	ϕ' = 34 degrees, γ = 120 pcf
2	Elev. 285 to 300	Silty Sand	ϕ' = 33 degrees, γ = 113 pcf
3	Elev. 272 to 285	Poorly-graded Sand	ϕ' = 34 degrees, γ = 120 pcf
4	Elev. 250 to 272	Silty Sand	ϕ' = 34 degrees, γ = 114 pcf

Minimum Design Parameters for Standard Plan Walls and XS Sheet Walls with Sound Walls

The following tables present the minimum foundation soil effective friction angle (phi) for all standard plan and XS sheet wall types, cases, heights, and limit states for bearing, sliding, and settlement (items b, c, and d above). The values represent the minimum required effective friction angle for all foundation materials in the bearing and settlement zone per the following:

For a given wall type, case, and height:

- The bearing influence zone is the foundation material that lies between the bottom of the footing and the depth of 1.5 times the effective footing width below the footing base.

- The settlement influence zone is the foundation material that lies between the bottom of the footing and the depth of 3 times the effective footing width below the footing base.
- Calculated settlement is 2 inches.
- All requirements of the Standard Plans and XS sheets (e.g., level ground in front of wall) must be met.

For example, the foundation material in the bearing stratum as defined in the preceding bullets must have Φ' = 34 degrees (or greater) for a 12-foot high Standard Type 1 (Case 2) retaining wall. If the requirement is met then the Geoprofessional must verify the design for items e, f, g, and h above prior to approving the wall design. If the requirement is not met, the wall does not meet the required standard plan design and cannot be used without ground improvement.

Minimum Effective Friction Angle for Standard Plan Walls

	Standard Type 1			Standard Type 1A		Standard Type 5		
Height (feet)	**Case 1**	**Case 2**	**Case 3**	**Case 1**	**Case 2**	**Case 1**	**Case 2**	**Case 3**
4	28	30	28	28	30	31	31	30
6	29	33	28	29	32	32	32	32
8	31	33	30	31	33	33	33	33
10	31	33	32	31	33	34	35	35
12	32	34	33	32	34	35	36	35
14	33	35	33					
16	34	36	35					
18	35	37	35					
20	35	38	36					
22	36	39	36					
24	36	39	38					
26	37	39	38					
28	38	40	38					
30	38	41	38					
32	38	41	38					
34	39	41	39					
36	39	41	40					

Minimum Effective Friction Angle for Standard Plan Walls (continued)

	Standard Type 6A		Standard Type 6B	
Height (feet)	**Case 1**	**Case 2**	**Case 1**	**Case 2**
3.33	28	30	28	30
4	28	32	28	31
4.67	28	33	28	32
5.33	28	33	28	32
6	28	33	28	33

Minimum Effective Friction Angle for XS Sheet Walls with Sound Walls

Height (feet)	Standard Wall Type 1 SW	Standard Wall Type 1 SWB	Standard Wall Type 5 SW	Standard Wall Type 5 SWB
6	29	29	32	32
8	31	31	33	33
10	32	32	34	34
12	34	34	34	34
14	36	36	35	35
16	34	34	36	36
18	35	35	36	36
20	36	36	38	38
22	36	36	38	38
24	36	36	38	38
26	37	37		
28	38	38		
30	38	38		
32	39	39		

Factored Gross Nominal Bearing Resistance (Strength and Extreme Event Limit States)

The Factored Gross Nominal Bearing Resistance must be calculated for soil foundations for both the Strength Limit State and Extreme Limit State. The bearing resistance is affected when groundwater is at a depth less than 1.5 times the footing width below the footing base. Determine the bearing resistance using the highest anticipated groundwater level at the footing location according to AASHTO LRFD BDS Section 10.6.3.

For bearing resistance on rock follow the design procedures in AASHTO LRFD BDS Section 10.6.3.2.

Walls on Slopes

The proximity of a retaining wall footing to a descending slope must be considered in the bearing resistance calculations. If the finished ground in front of the retaining wall slopes downward within a distance that is twice the width of the footing measured from the retaining wall toe, then the bearing resistance will differ from the level ground configuration.

The Modified Bearing Capacity Factors for Footing Adjacent to Sloping Ground (AASHTO LRFD BDS 10.6.3.1.2c) developed by Meyerhof (1957), or similar, must be used. Table C11.10.2.2.1 in the AASHTO LRFD Bridge Design Specifications provides guidelines for minimum foundation embedment when a wall is located on a slope.

Additionally, a sliding analysis using the retaining wall configuration, the foundation soil characteristics, and the slope geometry must be conducted. Because the site geometry does not conform to the Standard Plans and Standard Details, the PS&E must be prepared by Structure Design. The sliding analysis will be performed by the Structure Designer. The Structure Designer may request assistance from Geotechnical Services with calculating the factored sliding resistance. See AASHTO LRFD BDS section 10.6.3.4.

Settlement Evaluation (Service Limit State)

Calculate settlement using the net bearing stress shown in the Standard Plans, Revised Standard Plans, XS Sheets, or provided by the Structure Designer. If the calculated settlement is less than or equal to the specified permissible settlement, the retaining wall geometry and configuration meet the Service Limit State settlement criteria. If the calculated settlement exceeds the permissible settlement, then the retaining wall must be redesigned or the foundation conditions improved.

Settlement must be calculated per AASHTO LRFD BDS Section 10.6.2.4. The total settlement may include elastic, consolidation and secondary components. Settlements must be within the tolerable criteria for the type of retaining wall selected. The settlement must be calculated for the Service Limit State stress. The settlement evaluation must include settlement that occurs during and after wall construction.

Tolerable total and differential settlement criteria are as follows:

Wall Type	Tolerable Total Settlement	Differential Settlement over a Distance of 100 feet
Conventional Retaining Wall	$\leq$ 2”	$\leq$ 0.75”

More stringent tolerances may be necessary to meet aesthetic requirements for the walls.

Settlement evaluation for foundations on rock must follow the recommendations in AASHTO LRFD BDS Section 10.6.2.4.4.

Foundation stress distribution that may affect known underground utilities or adjacent structures must be evaluated. Results should be shared with the client so that the appropriate stakeholder is consulted.

Sliding Evaluation

This analysis is the responsibility of the Structure Designer. The Structure Designer may request assistance calculating the factored sliding resistance.

Overall Slope Stability (Service and Extreme Event Limit States)

The overall stability of the wall must be calculated using Service I Loads and AASHTO LRFD BDS Section 11.6.2.3.

Erosion Susceptibility and Mitigation

Embedment of the retaining wall foundation must account for anticipated scour, erosion or undermining. AASHTO LRFD BDS Sections 2.6.4.4.2 and 10.6.1.2 must be followed. Considerations for embedment should include slope geometry, erosional potential in front of the wall, frost heave protection, future construction activities, and external and global wall stability.

When the foundation material is subject to erosion or scour, measures must be taken to avoid undermining. In this instance the embedment may exceed the minimum embedment depth requirements, and additional countermeasures such as erosion control and hard facing should be considered.

The minimum cover for a conventional retaining wall footing is 1.0 or 2 feet.

Seismic Stability

Standard Plan Retaining Wall site seismic criteria threshold must be analyzed to confirm that the Coefficient of Horizontal Acceleration, k_h does not exceed 0.2. The k_h is calculated as 1/3 Horizontal Peak Ground Acceleration (HPGA). Therefore, at sites where the HPGA is equal to or less than 0.6g the Standard Plans are applicable. At sites where the HPGA is greater than 0.6g the wall will require seismic displacement analyses in accordance with the *Geotechnical Seismic Design of Earth Retaining Systems* module.

Seismic recommendations must also address seismic hazards such as liquefaction impacts to bearing resistance, overall stability and lateral deflection. The maximum allowable displacements are governed by wall performance and potentially-impacted facilities.

Surface and Subsurface Drainage Systems

Additional drainage measures should be implemented if the wall backfill cannot be depended on to be fully drained or if the groundwater conditions at the project site will affect the integrity of the wall. These additional groundwater control measures may include standard or deep underdrains and horizontal drains in addition to geocomposite drains, and drainage blankets at the wall or at the back of the backfill. To the greatest extent possible these groundwater control measures should prevent the infiltration of groundwater into the structure backfill. If wall drainage cannot be relied upon, the wall must be designed for hydrostatic pressure.

Surface drainage should be directed away from the wall. If this is not possible, surface drainage appurtenances such as impervious drainage inlets, lined ditches, curbs and gutters should be recommended.

Infiltration basins should not be positioned to introduce water into the retained earth zone or into the foundation material. The effects of bioswales on earth retention systems should be carefully considered. Depending upon the details of their construction, bioswales may lead to increased infiltration of water, and the possibility of increased hydrostatic pressure and pore pressures.

Modified Design of Conventional Retaining Walls

The Geoprofessional must review the proposed retaining wall configuration for conformance with the Standard Plans and XS Sheets. Among the important configuration elements to check are the lateral distance to a descending slope in front of the retaining wall, the vertical footing cover, the ground slope in the retained zone, additional surcharges that deviate from the live traffic loading, and the HPGA. For configurations other than those shown in the Standard Plans, special design walls by Structure Design are required.

For the modified design of Standard Plan and XS Sheet retaining walls the Structure Designer will evaluate the lateral sliding and deflection, eccentricity of the resultant foundation load, and internal structural stability requirements.

When the lateral earth pressures resulting from surcharge loads do not conform to those assumed for the Standard Plans and XS Sheets, project specific lateral earth pressures must be developed based on Section 3.11.6 of the AASHTO LRFD Specifications.

Retaining Walls Supported on Piles (Type 1, Type 5 and Type 7)

Pile tip elevations for the Strength, Service and Extreme Load demand must be determined according to the applicable deep foundations module. Factored pile load demands will be provided by the Structure Designer and determined on a project specific basis.

Bridge Design Detail Sheets are available for Type 1 SWP, Type 1 SWBP, Type 5 SWP, Type 5 SWBP, Type 7 SWP and Type 7 SWBP. These sheets show the use of driven Class 90 battered piles. Where battered piles are used, a portion of the lateral demand on the foundation piles will be resisted by compression of the battered piles.

When vertical piles are required, site specific lateral pile analyses are required. The Geoprofessional may be requested to provide the Structure Designer with foundation soil parameters to perform the analysis.

Type 7 Retaining Walls with Ground Anchors

Type 7 Retaining Walls use ground anchors to resist overturning, sliding, and/or uplift. The anchor bond zone must be developed below the theoretical shear failure zone for bearing resistance. There is typically at least 5 feet between the bonded zone and the theoretical shear failure surface in the foundation soil or rock. Use a minimum ground anchor unbonded length of 15 feet. The minimum horizontal spacing of anchors should be greater than 3 times drilled hold diameter to minimize group effect between adjacent ground anchors.

Ground Improvement

Where existing foundation materials do not provide adequate bearing resistance or result in excessive settlement, consider improving the foundation conditions by removing some or all of the unsuitable material, and replacing it with compacted fill. Material meeting the specification for structure backfill or aggregate base is often used as the replacement material. Consideration should be given to whether a geotechnical fabric is required to separate the backfill from the native soils or to enhance the subgrade behavior. Standard Specification Section 19-5.03B discusses a typical configuration that could be used for removal and replacement. Recommend that a typical section for the “remove and replace” be provided in the plans.

Where “remove and replace” is not feasible, consider ground improvement options as an alternative to deep foundations (see *Ground Improvement* module).

Although lightweight fill such as cellular concrete, expanded polystyrene (EPS) blocks and volcanic materials (Scoria) may not be considered as ground improvement, they can be used as backfill material behind the conventional retaining wall to reduce the active pressure and to decrease the foundation stresses.

Reporting

- Conventional retaining wall recommendations prepared for the District must comply with the *Geotechnical Design Report* module.

- Conventional retaining wall recommendations prepared for Structures Design must comply with the *Foundation Reports for Earth Retaining Systems* module.

    - If the conventional retaining wall is within 150 feet of a bridge then include the retaining wall recommendations in the foundation report for the bridge.

    - All other conventional retaining walls prepared for Structure Design must be reported in its own foundation report.

References

1. AASHTO LRFD Bridge Design Specifications, 2012 (Sixth Edition) with California Amendments
2. Caltrans Standard Plans and Revised Standard Plans
3. Caltrans Memo to Designers (MTD) 5-19, *Earth Retaining Systems Communication*
4. Caltrans Geotechnical Manual, *Geotechnical Investigations*
5. Caltrans Geotechnical Manual, *Soil Correlations*
6. Caltrans Geotechnical Manual, *Foundation Reports for Earth Retaining Systems*
7. Caltrans Geotechnical Manual, *Geotechnical Design Reports*
8. Caltrans Geotechnical Manual, *Ground Improvement*
9. Federal Highway Administration Publication No. FHWA-NHI-07-071, *Earth Retaining Structures Reference Manual,* June 2008.
10. Federal Highway Administration Publication No. FHWA-IF-99-015, Geotechnical Engineering Circular No. 4, *Ground Anchors and Anchored Systems*, June 1999.

Mechanically Stabilized Embankment (Caltrans Pre-Designed)

This module presents Caltrans practice for the geotechnical investigation, design, and reporting for Mechanically Stabilized Embankments (MSE). The AASHTO LRFD Bridge Design Specifications describe MSE as:

> *"Mechanically Stabilized Earth (MSE) systems, whose elements may be proprietary, employ either metallic (strip or grid type) or geosynthetic (geotextile, strip, or geogrid) tensile reinforcements in the soil mass, and a facing element which is vertical or near vertical."*

MSE are defined as having a face inclination of 70 degrees to vertical, whereas GRE are flatter than 70 degrees. MSE have a maximum vertical height of 50 feet.

This module addresses Caltrans pre-designed MSE, which must meet minimum requirements presented in the Design section of this module. It does not address MSE with geosynthetic reinforcement or Geosynthetic Reinforced Embankments (GRE). Refer to the MSE (Non-Standard) module for design of MSE that do not meet the criteria of this module.

When the Caltrans pre-designed MSE is shown on the Contract Plans, the Contractor has the option to construct a Proprietary Earth Retaining System (Caltrans Pre-approved Alternative Earth Retaining Systems).

The following terms are defined:

- Reinforced Soil: The material that contains the reinforcement.
- Retained Material: The material located behind or above the reinforced soil block.
- Foundation Material: The material located below the reinforced soil block.

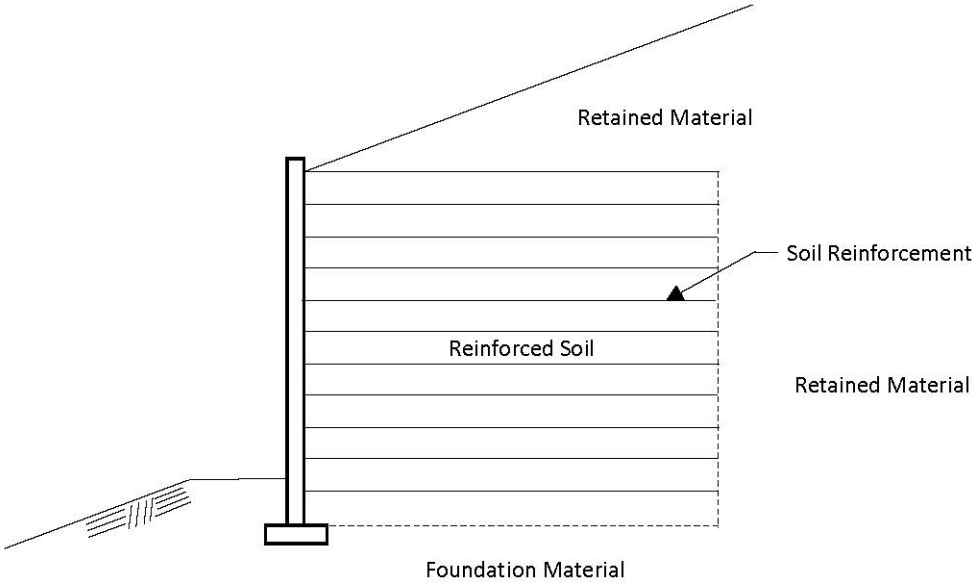


Figure 1: MSE

In addition to this module, the documents that govern or guide the investigation, design, and reporting for MSE include:

- AASHTO LRFD Bridge Design Specifications with California Amendments
- Caltrans Standard Specifications 47-2, *Mechanically Stabilized Embankment*
- Caltrans Bridge Standard Detail Sheets (XS-Sheets) Section 13
- Caltrans Bridge Design Aids (BDA) 3-8, *Mechanically Stabilized Embankments*
- Caltrans Memo to Designers (MTD) 5-19, *Earth Retaining Systems Communication*
- Caltrans Geotechnical Manual, *Foundation Reports for Earth Retaining Systems*

The Geoprofessional's role is to assist in all phases of project development as requested by the Structure Designer. This assistance may include: literature search, preliminary design, and type selection during the project planning or early design phase; field investigation, analyses, external MSE design, and design support during the project design phase; and construction support and possible retaining wall alterations due to project changes or unforeseen conditions discovered during the construction phase. The geotechnical effort must be documented and communicated in appropriate reports and memorandum.

Investigations

A geotechnical investigation must be conducted for all MSE. The goal of the investigation is to determine the distribution, properties, and behavior of the soil and rock that will affect MSE design and construction; the groundwater condition that will affect MSE design and construction; the distribution of unsuitable or weak materials requiring planned remedial measures; and the suitability of excavated soil to be used as reinforced soil, retained material, and/or foundation material.

The geotechnical investigation should provide information to evaluate the stability and performance of the MSE, which should include:

- Strength and unit weight of the reinforced soil, retained material, and foundation material
- Settlement characteristics of the foundation material
- Strength and unit weight of materials affecting slope stability
- Corrosion potential of materials in contact with the MSE
- Groundwater location and quantity of seepage

In some instances, the information obtained through a literature search (see *Geotechnical Investigations Module*) and field mapping will be sufficient for MSE design. Examples of such instances are MSE built in a sequence of sedimentary strata where nearby borings exist for the same sequence of sedimentary units, or walls founded on rock with abundant rock exposures and previous testing to sufficiently characterize the rock.

The Geoprofessional should develop a prudent exploration plan considering site constraints and available resources, and consider uncertainty and risk of not adequately investigating a location. The Geoprofessional should:

- Establish MSE layout and configuration as accurately as possible. Final layout and height may not be determined until late in the design phase.
- Perform a literature research to gather all relevant information related to site geology, strength of soils, and geologic hazards.
- Perform geologic field mapping of the site. The mapping should be sufficient to generate geologic cross sections at the MSE when combined with other terrain data, and to identify geologic hazards.
- Develop a subsurface exploration and laboratory testing plan to obtain information gathered through the literature research and field mapping. Locate exploratory borings or Cone Penetration Tests spaced at intervals of 100 to 200 feet along the proposed alignment, with borings strategically positioned in front, behind, and directly on the layout line. The number of borings necessary to delineate site conditions may be greatly reduced or increased due to the value of pre-existing data, uniformity of site geology, and the quality of site specific geologic mapping.
- Advance the subsurface exploration to an appropriate depth. The depth of exploration should generally extend below the foundation to the deepest of:
    - 15 feet
    - 2 times the height of the MSE
    - 4 times the base width
    - To the full depth of soft, loose, weak soils upon which wall stability and settlement is dependent
    - To a depth below where material strength and strain characteristics are acceptable
- Conduct Standard Penetration Tests (SPT) at maximum intervals of 5 feet. Closer intervals of SPT testing should be considered within a depth of 2 times the base width below the proposed reinforced zone (the zone of greatest bearing loads), and where soil strength properties are projected to be low and/or highly varied, such as poorly compacted fill or soft/loose alluvial soils.
- Gather all information necessary to evaluate the stability of permanent and temporary excavations and cut slopes that will influence design and construction of the MSE.
- Estimate soil strengths based on index properties established through SPT correlations, pocket penetrometer, torvane, and CPT (see *Correlations Module*). Perform laboratory strength tests when correlation-based strengths are borderline acceptable or unacceptable, or otherwise questionable.
- For MSE founded on rock, strengths may be sufficiently estimated by reviewing data developed for similar rock on nearby projects. Perform laboratory strength testing as necessary to confirm that the rock meets the minimum strength requirements, or if rock excavation is anticipated.
- Conduct consolidation testing of clay soils wherever settlement magnitude and rate are project considerations.

- Sample and test potential borrow sites and cut locations to determine if the material satisfies gradation and/or corrosion criteria for the reinforced soil (SS47-2.02C), retained material, and foundation material.

Design Procedures

MSE are designed through a coordinated effort between the Geoprofessional and the Structure Designer. Refer to *Caltrans Memo to Designers 5-19, Earth Retaining Systems Communication* for the communication protocol between the Structure Designer and Geoprofessional. MTD 5-19 provides tables to facilitate communication between the Structure Designer and Geoprofessional.

Design of an MSE involves evaluation of:

(i) Service Limit State: Overall (Global) and Compound Stability, Settlement
(ii) Strength Limit State:
    a. External Stability: Bearing Resistance, Sliding, Limiting Eccentricity
    b. Internal Stability: Tensile and pullout resistance of reinforcement, structural resistance of face elements and connections
(iii) Extreme Limit State:
    a. External Stability: Overall (Global) and Compound Stability, Bearing Resistance, Sliding, Limiting Eccentricity
    b. Internal Stability: Tensile and pullout resistance of reinforcement, structural resistance of face elements and connections
(iv) Additional considerations, such as corrosion, drainage, traffic barriers, and facing elements.
(v) Geologic Hazards

For a Caltrans pre-designed MSE the Geoprofessional is responsible for:

(i) Service Limit State: Overall (Global) and Compound Stability, Settlement
(ii) Strength Limit State: Bearing Resistance
(iii) Extreme Limit State: Overall (Global) and Compound Stability, Bearing Resistance
(iv) Drainage, Corrosion sampling and testing
(v) Geologic Hazards

For a Caltrans pre-designed MSE the Structure Designer is responsible for:

(i) Service Limit State: Total Permissible Settlement
(ii) Strength Limit State:
    a. External Stability: Sliding, Limiting Eccentricity
    b. Internal Stability: Tensile and pullout resistance of reinforcement, structural resistance of face elements and connections
(iii) Extreme Limit State:
    a. External Stability: Sliding, Limiting Eccentricity
    b. Internal Stability: Tensile and pullout resistance of reinforcement, structural resistance of face elements and connections

(iv) Additional considerations, such as corrosion, drainage, traffic barriers, and facing elements.
(v) Scour elevation

The geotechnical parameters shown in the Caltrans Bridge Standard Detail Sheets (XS-Sheets) are as follows:

- Internal Design: Friction Angle (ϕ) = 34°, Unit Weight (γ) = 120 pcf
- External Design: ϕ (retained backfill) = 30°, γ = 120 pcf
- ϕ (foundation) = 30°
- Horizontal Seismic Coefficient (k_h) = 0.2

Typical factors that would disqualify use of the Caltrans pre-designed MSE include:

- Use of reinforced soil that does not meet the requirements of either SS47-2.02C or those shown on the XS-Sheets.
- External loading that exceeds those used in the Caltrans pre-designed MSE.
- Bearing resistance and/or overall stability requirements cannot be met.
- Where the k_h is greater than 0.2. Caltrans practice is to calculate k_h as 1/3 PGA (PGA is the horizontal peak ground acceleration).

Overall (Global) and Compound Stability

Perform static stability analyses (Service Limit State) and pseudo-static stability analyses (Extreme Limit State) using the parameters of the foundation material, reinforced soil, reinforcement, and retained material, and the anticipated groundwater surface. The analysis should include evaluation of failure surfaces behind the MSE, through the MSE (compound), and beneath the MSE (AASHTO Fig. 11.10.2-1, Fig. 11.10.4.3-1). For overall stability, the reinforced soil mass can be modeled as a block, using a high cohesion value to force the failure surfaces behind the MSE.

Perform the pseudo-static slope stability analysis using a k_h = 1/3 PGA.

The calculated resistance factor (□), the inverse of the calculated factor of safety, must be less than the required resistance factor in AASHTO 11.6.2.3.

If overall stability cannot be satisfied, consider:

(i) Increasing the reinforcement length
(ii) Lowering the foundation elevation
(iii) Ground improvement

Settlement

Calculate settlement using the vertical bearing stress and effective footing width for the Service Limit State (BDA 3-8, Attachment 2). Use a LRFD Resistance Factor (□) of 1.0 for all settlement calculations (AASHTO 10.5.5.1). Settlement magnitudes greater than the total permissible settlement should be discussed with the Structure Designer (MTD 5-19, Attachment 3).

- Cohesionless Soils: Use the Hough method or the elastic half-space method (AASHTO 10.6.2.4.2). As both methods can be overly conservative, calculate settlement using both methods prior to considering ground improvement or alternative wall types.
- Cohesive Soils: Use AASHTO (10.6.2.4.3).
- Rock: Use AASHTO (10.6.2.4.4). For foundations on sloping ground, make appropriate reductions in overburden stresses.

Bearing Resistance

Calculate bearing resistance of soil using the bearing capacity equation (AASHTO 10.6.3.1.2) and the effective footing widths (BDA 3-8, Attachment 2). Use a resistance factor (ϕ) of 0.45 to 0.55 for the Strength Limit State, and 1.0 for the Extreme Limit State. Adjust the bearing capacity equation as necessary to account for sloping ground conditions (AASHTO 10.6.3.1.2c). The factored bearing resistance must be greater than or equal to the vertical bearing stress found in BDA 3-8, Attachment 2. The methods discussed above may also be used for design in Intermediate Geomaterial or weak rock that behaves like a very dense or hard soil. For more competent rock, calculate bearing resistance on rock using AASHTO 10.6.3.2.

In cases where unacceptable settlements are predicted, or low bearing resistance results from the presence of near-surface loose, soft, or non-uniform materials, consider removing the inadequate material and recompacting it, or replacing it with structure backfill.

Drainage

Drainage details and considerations are presented in BDA 3-8 and the Bridge Standard Detail Sheets (XS-Sheets). Evaluate the surface water drainage and provide recommendations to direct water away from the MSE.

Evaluate the potential for groundwater flow into the reinforced soil mass and retained backfill, and select either the 8-inch corrugated perforated plastic pipe or the 4-inch (or larger) smooth-walled perforated rigid plastic pipe. Select the flexible pipe when differential settlement is anticipated. Specify chimney drains or drainage blankets as necessary to supplement the underdrain system (AASHTO 11.10.8).

Where a potential for inundation of the MSE system exists specify a rapidly-draining reinforced soil by modifying the gradation requirements for Structure Backfill (SS47-2.02C).

Do not allow drainage systems from temporary shoring or surface drainage to be combined with the MSE underdrain system.

Geologic Hazards

The design must also account for geologic hazards such as:

- Liquefaction, Lateral Spreading: The estimated seismic settlement and/or displacement should be communicated to the Structure Designer and Project Engineer to determine acceptability. Unacceptable displacement might be reduced using remove and replace or ground improvement methods.
- Landslides: Lateral thrust forces from landslides should be calculated and provided to the Structure Designer for analysis of adequacy of the Caltrans pre-designed system.
- Scour: MSE may be constructed adjacent to a watercourse provided that the top of footing is located at least two feet below the total scour elevation (AASHTO 11.10.2.2).

Reporting

Caltrans pre-designed MSE recommendations must be reported in accordance with *Foundation Reports for Earth Retaining Systems* and the requirements herein.

Geotechnical recommendations Caltrans pre-designed MSE must include the following:

- Soil and/or rock parameters (unit weight, friction angle and cohesion for soil, and unit weight and shear strength of the rock mass). This applies to the foundation material, reinforced soil, and retained materials.
- Suitability of on-site material for use as foundation material, reinforced soil, and retained materials.
- Factored bearing resistances (strength and extreme event limit states)
- Total and differential settlement as a result of application of the vertical bearing stress (service limit state). Differential settlement should be examined both along the alignment of the MSE and between the front and back of the MSE.
- Effects of MSE construction on adjacent ground and/or existing structures, utilities and other structures, both above and below ground.
- Calculated resistance factor for overall (global) and local stability (service and extreme event limit states). Provide the method of analysis.
- Susceptibility of foundation material to erosion and recommended mitigation.
- Seismic stability of foundation material: seismic settlement, liquefaction impacts to overall stability (including estimated permanent lateral displacement) and bearing resistance. Provide recommended mitigation measures.
- Discuss the groundwater condition anticipated over the design life of the MSE, and requirements for a blanket drain located behind the reinforced soil block. Describe the drainage system location and configuration. Reference the Standard Specifications for permeable material and geosynthetic filter fabric type.
- Corrosiveness of foundation material, retained material, and water sources.
- Foundation improvements required to meet geotechnical design objectives, such as sub-excavation, foundation preloading, and surcharge delay periods.
- The minimum embedment depth to account for erosion, overall stability, bearing resistance, scour, and settlement.
- Minimum base width to meet overall stability requirements.

Provide the following tables:

Design Data for MSE XX

MSE Station (feet)	Design Height (H) (feet)	Bottom of Leveling Pad Elevation (feet)	Base Width (B) (feet)	Minimum Embedment Depth (feet)

Foundation Data for MSE XX

MSE Station	Service Limit State			Strength Limit State			Extreme Limit State		
	Service Limit State, Effective Base Width[1] (feet)	Service Limit State, Vertical Bearing Stress[1] (psf)	Service Limit State, Calculated Settlement at Vertical Bearing Stress2 (inch)	Strength Limit State, Effective Base Width[1] (feet)	Strength Limit State, Vertical Bearing Stress[1] (psf)	Strength Limit State, Factored Bearing Resistance (ϕ = __) (psf)	Extreme Limit State, Effective Base Width1 (feet)	Extreme Limit State, Vertical Bearing Stress[1] (psf)	Extreme Limit State, Factored Bearing Resistance (ϕ = __) (psf)

1. BDA 3-8, Attachment 2
2. Total Permissible Settlement = YY (provided by Structure Designer)

References

1. AASHTO LRFD Bridge Design Specifications with California Amendments
2. Caltrans Standard Specifications 47-2, *Mechanically Stabilized Embankment*
3. Caltrans Bridge Standard Detail Sheets (XS Sheets), Section 13
4. Caltrans Bridge Design Aids (BDA) 3-8, *Mechanically Stabilized Embankments*
5. Caltrans Memo to Designers (MTD)
    - MTD 5-19, *Earth Retaining Systems Communication*
    - MTD 5-5, *Design Criteria of Standard Earth Retaining System*s
6. Caltrans Geotechnical Manual
    - *Foundation Reports for Earth Retaining Systems*
    - *Geotechnical Investigations*
7. FHWA-NHI-10-024 and 025, *Geotechnical Engineering Circular No. 11 - Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes*
    - Volume I
    - Volume II
8. FHWA-NHI-09-087, *Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Slopes*

Mechanically Stabilized Embankments (Non-Standard)

A mechanically stabilized embankments (MSE) consists of facings, tensile reinforcements, and reinforced soils (Figure 1). The facings can be precast concrete panels, modular blocks, wire mesh etc., and tensile reinforcements can be either metallic (strip, grid or wire mesh mat) or geosynthetic (strip, grid or sheet). Caltrans standard MSE use wire mesh as the tensile reinforcements and standardized precast concrete panel as the facings.

An MSE is considered non-standard when:

- Design parameters, load cases, and wall geometry are outside of the design parameters presented in Section 13 of *Bridge Standard Details (XS Sheets)*, and Section 3.8 of *Bridge Design Aids*
- Tensile reinforcements other than welded wire mat are used such as steel strips, and geosynthetic
- Facing elements other than standardized precast concrete facing panels are used such as modular blocks, wire meshes etc.

Design and performance advantages of MSE include:

- Capable of tolerating greater total and differential settlements compared to conventional gravity retaining walls
- Cost effective in fill, especially for design wall heights greater than 10 feet

MSEs are not favorable when there are:

- Utilities or highway drainage located within the reinforced mass
- Flood plains or scour which can undermine the reinforced soil mass
- Underlying soft highly plastic clays; organic soils; collapsible soils; expansive soils

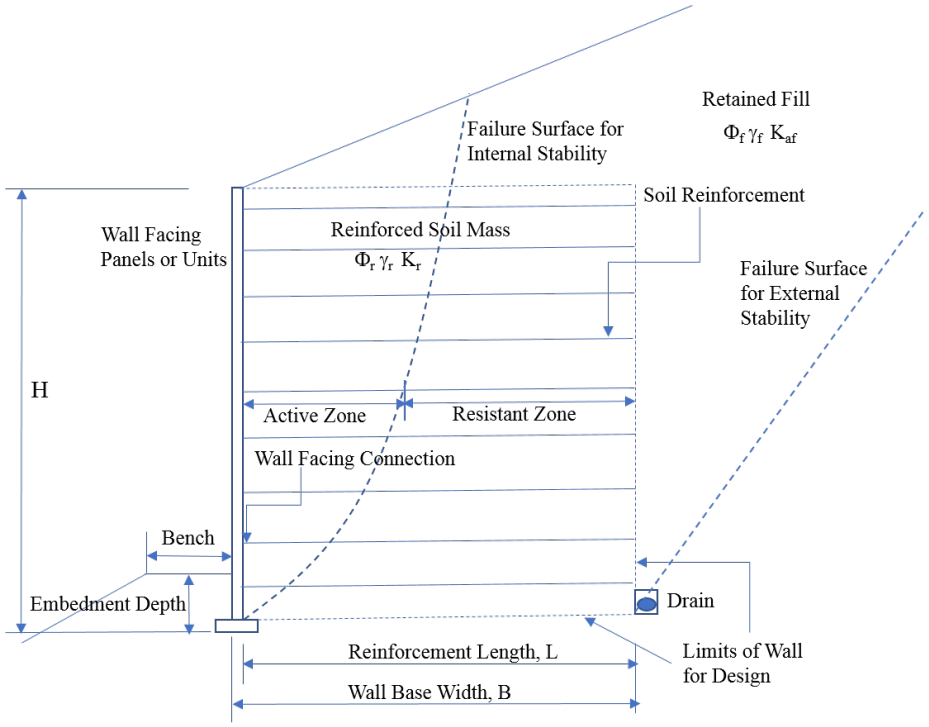


Figure 1: MSE Components and Dimensions (After AASHTO LRFD BDS, 2012)

References for Design and Reporting

Non-standard MSE design uses the same geotechnical analysis and design methodology of the standard MSE (i.e., settlement, bearing resistance and global stability). This is because the entire MSE, including all components, is modeled as a rigid block. Therefore, the "Mechanically Stabilized Embankment (Caltrans Pre-Designed)" module is applicable for the geotechnical design and analysis of non-standard MSE.

For MSE design, use this module, and:

- *AASHTO LRFD Bridge Design Specifications (BDS) with California Amendments*, hereafter AASHTO.
- *Geotechnical Manual,* "Mechanically Stabilized Embankment (Caltrans Pre-Design)"
- *Geotechnical Manual, "*Seismic Design of ERS"
- *Bridge Design Aids*, Section 3-8 "Mechanically Stabilized Embankments"
- *Memos to Designers 5-19*, "Earth Retaining Systems Communication"
- *Geotechnical Manual*, "Foundation Reports for Earth Retaining Systems"

For design cases where the guidance provided in the above documents is not applicable, refer to other FHWA reference manuals including *FHWA NHI-05-094*, "LRFD for Highway Substructures and Earth Retaining Structure" *or FHWA NHI-10-024* "Design and

Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volumes 1 and 2".

Responsibilities for Design

Geotechnical Services' responsibilities in the design of non-standard MSE are:

- Develop interpreted subsurface cross sections. For a long wall, several subsurface cross sections along the alignment may be needed
- Determine engineering properties such as unit weight, cohesion, friction angle, and associated lateral earth pressure coefficients
- Analyze the magnitude and distribution of lateral earth pressure for complex wall geometries when conventional earth pressure theories are not applicable or when requested by the structure designer
- Analyze the minimum horizonal reinforcement length (base width) based on global stability requirements and bearing capacity requirements at the wall base
- Determine the bearing capacity and settlement at the wall base

Information that should be provided by the structure designer over the course of non-standard MSE investigation and design are:

- Plans showing the location of wall (begin and end, length and alignment)
- Elevation view of wall (maximum and minimum design height)
- Cross sections of wall (for example, every 10 to 50 feet)
- Bottom of MSE leveling pad elevation
- Base width
- Effective base width for service, strength and extreme limit states
- Vertical bearing stress for service, strength and extreme limit states

Investigations

The geotechnical investigation for a non-standard MSE should follow the *Investigations* section of the *Mechanically Stabilized Embankment (Caltrans Pre-Designed)* module.

Design Procedures

The geotechnical design of MSE must meet displacement and stability requirements for following limit states:

- Service Limit State – Movement and Global Stability (AASHTO 11.10.4).
- Strength Limit State – Bearing Resistance under Safety against Soil Failure (External Stability) (AASHTO 11.10.5.4).
- Extreme Limit State – Bearing Resistance and Global Stability (AASHTO 11.10.7).

For each of the limit states, load and resistance factors should be applied in accordance with AASHTO 3.4.1 (Table 3.4.1-1) and 11.5.6 and *California Amendments* (Tables 3.4.1-1 and 11.5.7-1).

The Geoprofessional should assist the structure designer in estimating all applicable lateral pressures including static and seismic earth pressure, surcharge load induced earth pressure, and hydrostatic pressure. For the estimation of lateral pressures, refer to AASHTO 11.10.5.2, 11.10.10, 11.6.5 and 11.10.7.

For the seismic design of MSE, use 1/3 horizontal peak ground acceleration (HPGA) for the horizontal acceleration coefficient (k_h) if they can tolerate the expected mean seismic displacement of 5.0 inches during seismic event. The HPGA is the peak ground acceleration (PGA) calculated using Caltrans ARS online (v.2.3.09), which is the acceleration at zero period (T=0 second). If the MSE cannot tolerate the expected mean seismic displacement of 5.0 inches or sliding stability at wall base is not satisfied by using the 1/3 HPGA, consult the structure designer for tolerable seismic displacement of the MSE, and assist the structure designer in calculating k_h. The k_h based on the tolerable permanent seismic displacement should be used for the seismic design of MSE.

Service Limit State

Displacement

The design of the MSE must ensure that the vertical and lateral displacement does not affect the performance of the wall. As the MSE is a flexible system, it can accommodate greater settlement than a typical retaining wall. As a rule of thumb, a total vertical settlement of about 6 inches is considered acceptable. For the calculation of settlement, refer to AASHTO 10.6.2.4.2, 10.6.2.4.3 and 11.10.4.1. For the tolerable limits of differential settlement, refer to AASHTO 11.10.4.1, and consult with the structure designer.

Global Stability

The global stability is evaluated using limit equilibrium (LE) slope stability analysis such as Morgenstern-Price, Modified Bishop, Janbu, or Spencer methods. For the global stability analysis and resistance factors, refer to AASHTO 11.10.4.3 and 11.6.2.3, and Figure 11.10.4.3-1.

Strength Limit State

Bearing Resistance

For the bearing resistance of MSE, refer to AASHTO 11.10.5.4, 10.6.3.1 and 10.6.3.2. When calculating the bearing resistance, use the effective footing width, and a resistance factor of 0.65 (Table 11.5.7-1 of *California Amendments*). If there is downward sloping ground near or adjacent to the MSE, adjust the bearing capacity equation as necessary to account for sloping ground conditions according to AASHTO 10.6.3.1.2c.

Extreme Limit State

Bearing Resistance

For the seismic bearing resistance of MSE, refer to AASHTO 11.5.8, 11.10.7.1, 11.10.5.4, 10.6.3.1 and 10.6.3.2. When calculating the bearing resistance, use the effective footing width, and a resistance factor of 0.9 (AASHTO 11.5.8). If there is downward sloping ground near or adjacent to the MSE, adjust the bearing capacity equation as necessary to account for sloping ground conditions according to AASHTO 10.6.3.1.2c).

Seismic Global Stability

For the seismic global stability, use AASHTO 11.10.4.3 with a resistance factor of 1.0 and a k_h of 1/3 HPGA. If the seismic global stability is not satisfied with the k_h of 1/3 HPGA or the expected mean seismic displacement of 5.0 inches is not acceptable for MSE, use the following steps:

1. Find the horizontal yield acceleration coefficient (k_y) using iterative LE slope stability analyses.
2. Perform seismic displacement analysis using simplified seismic displacement method according to AASHTO A11.
3. Consult the structure designer for calculated and tolerable seismic displacement.

Reporting

Prepare the report in accordance with *Foundation Reports for Earth Retaining Systems.* Include geotechnical design data provided by the SD and foundation recommendations as listed below.

When construction specifications related to project-specific geotechnical design and/or recommendations for non-standard MSE are not addressed in the Standard Specifications, provide the information and instructions in the *Notes for Specifications* section of the foundation report. For the information and guidance on the geotechnical related Standard and Non-Standard Special Provisions, refer to the *Geotechnical Notes for Specifications* module.

Recommendations

- Soil and rock soil properties (unit weight, cohesion and friction angle for soil and unit weight and shear strength of rock mass). This applies to foundation soils and retained soils behind the reinforced soil mass. Foundation soil/rock properties can be used for sliding stability check and soil properties for the retained soils behind the reinforced soil mass can be used for the calculation of lateral earth pressure.
- Magnitude and distribution of static/seismic lateral earth pressure behind the reinforced soil mass if needed or requested
- Factored bearing resistance for strength and extreme limit states
- Permissible net bearing stress corresponding to tolerable settlement (service limit state) or total settlement
- Differential settlements under service limit state along alignment of the MSE and facing elements, and between the front and back of the MSE
- Minimum horizonal reinforcement length (base width) to meet global stability requirement; The minimum base width should not be less than 8 feet for uniform compaction and constructability and shall be greater than 70 percent of the wall height measured from the leveling pad (refer to AASHTO 11.10.2.1)
- Calculated resistance factor or factor of safety for global stability (service and extreme limit states)
- Minimum MSE facing embedment below finish grade to meet the requirement for erosion, future excavation, local stability and global stability (refer to AASHTO 11.10.2.2); according to *Caltrans Bridge Design Aid 3-8*, the embedment depth shall not be less than 10 percent of the design wall height with a minimum of 2 feet. A minimum horizonal bench width of 4 feet in front of wall is also recommended for walls founded on sloping ground
- Liquefaction potential and associated lateral spreading, and recommended mitigation measures if needed
- Horizontal acceleration coefficient (k_h) and associated seismic displacement
- Drainage system details and specifications, if drain systems other than the standard are needed to intercept the flow

Use the following tables to summarize geotechnical design data provided by the SD and foundation design recommendations:

Design Data for MSE XX

MSE Station (feet)	Design Height (H) (feet)	Bottom of Leveling Pad Elevation (feet)	Base Width (B) (feet)	Minimum Embedment Depth (feet)

Foundation Design Recommendations for MSE XX

MSE Station	Service Limit State			Strength Limit State			Extreme Limit State		
	Effective Base Width$_1$ (feet)	Vertical Bearing Stress$_1$ (psf)	Calculated Settlement at Vertical Bearing Stress$_2$ (inch)	Effective Base Width$_1$ (feet)	Vertical Bearing Stress$_1$ (psf)	Factored Bearing Resistance (ϕ=__) (psf)	Effective Base Width1 (feet)	Vertical Bearing Stress1 (psf)	Factored Bearing Resistance (ϕ = __) (psf)

1 SOIL NAIL WALLS

The soil nail wall is an effective technique to construct an earth retaining system (ERS) that requires top-down excavation. In a soil nail wall system, soil nails function as passive reinforcing elements that are installed and grouted in sub-horizontal drilled-holes to form a composite mass.

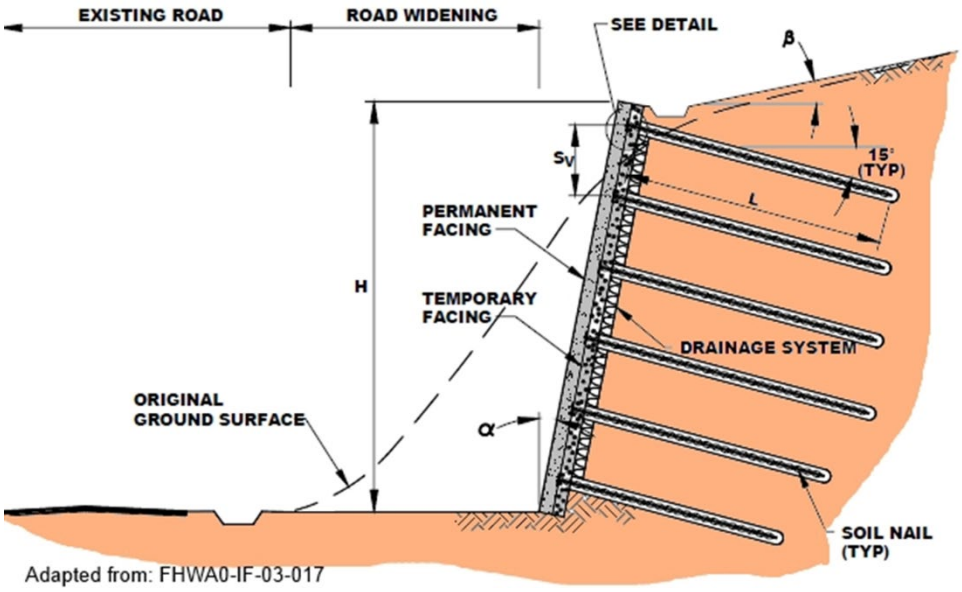


Figure 1 – Schematic Soil Nail Wall

1.1. Requirements and Guidelines

In addition to this module, refer to the following documents for the requirements and guidelines of geotechnical investigation, design, and reporting of Soil Nail Walls:

- *FHWA Geotechnical Engineering Circular (GEC) No. 7 (2015)*
- *Memos to Designers 5-19, Earth Retaining Systems Communication*
- *Geotechnical Manual, Foundation Reports for Earth Retaining Systems*
- *Bridge Design Aids 3-9, Soil Nail Wall Facing Design*
- *Structure Technical Policy 11.23 Design Criteria for Soil Nail Wall Facing*

1.2. Advantages and Characteristics of Soil Nail Walls

The advantages of soil nail walls include:

- Less right of way needed than competing systems, such as ground anchors
- Less disruptive to traffic and cause less environmental impact
- Relatively fast construction
- Cost effective at remote sites and sites with difficult access

- Effective in landslide repair above the landslide scarp
- Relatively flexible and can accommodate relatively large total and differential movements
- Perform well under seismic loading
- Have more redundancy than ground anchors because of the larger number of reinforcing elements per unit area; a passive system with relatively lower tensile stress sustained by the reinforcing elements; and have an established construction quality assurance program
- More economical than conventional earth retaining systems taller than 15 feet
- Typically, are equivalent or more cost-effective than ground anchor walls

Soil nail walls are not feasible when there are:

- Stringent requirements that limit the wall movement during construction, (e.g., the proposed wall is adjacent to and below a critical structure, such as a bridge abutment)
- Utilities behind the wall and within the soil nails reinforced zone
- Difficulty to obtain permanent easements

Favorable subsurface conditions for soil nail wall construction include:

- Excavated face can stand unsupported and stable until the facing is structurally complete
- Stiff cohesive soil or soil with sufficient apparent cohesion; weathered rock with favorable bedding planes; and well-graded and well-compacted backfill
- Drilled-holes can remain open and stable without casing until the nails are installed and the drilled-hole is grouted
- The toe of wall is above groundwater table

Unfavorable subsurface conditions for soil nail wall construction include:

- Poorly graded loose sand; soft highly plastic clay; organic soil; collapsible soil; expansive soil; cobbles and boulders; weathered rock with unfavorable bedding planes
- Groundwater table is above the toe of wall
- Corrosive soil and groundwater

2 GEOTECHNICAL DESIGN PRACTICE

The geotechnical tasks for the design of soil nail walls include:

1. Work with other project development team (PDT) members to evaluate and select the appropriate wall type
2. Evaluate of available information
3. Perform subsurface exploration
4. Perform design and analysis
5. Issue the geotechnical report
6. Assist in developing and reviewing the PS&E package

7. Provide construction support

The following are main task to be carried out during geotechnical design.

3 PROJECT INITIATION

During project initiation, the need for a wall will be identified by the District. Upon request, the geotechnical designer should perform preliminary assessment of the site and issue a preliminary geotechnical report that provides an evaluation of feasible wall types and a recommended wall type.

During Type Selection process, communicate and discuss with District Project Engineer and structure designer to select the preferred wall type. Constructability, constraints, and cost should be discussed and evaluated to arrive at the preferred option.

4 LITERATURE REVIEW

Refer to *Geotechnical Investigations* for guidelines on performing literature searches and evaluate available information for applicability to the soil nail wall design. Obtain and evaluate the general plans and cross sections of proposed walls from District or Structure Design (SD).

5 GEOTECHNICAL INVESTIGATION

Geotechnical investigation for soil nail wall design should obtain enough information for design, including:

- Soil and rock stratigraphy
- Soil and rock engineering properties, including unit weight, shear strength, orientation and spacing of bedding, and estimated nominal pullout resistance
- Groundwater elevation
- Information that may assist in deducing and evaluating potential construction issues, such as difficulties of excavating the wall face and drilling the holes should also be retrieved as much as practically possible

To plan for and carry out geotechnical investigation, including laboratory test, refer to the applicable modules in the Geotechnical Manual and:

- *FHWA Geotechnical Engineering Circular (GEC) No. 7 Soil Nail Walls*
- *FHWA Geotechnical Engineering Circular (GEC) No. 5 Geotechnical Site Characterization*
- *NCHRP Web-Only Document 258 Manual on Subsurface Investigations (2019)*

Designing and constructing a soil nail wall along a highway often require excavating the lower portion of a native slope that extends far above the highway. Access to the steep slope behind the wall layout line to perform geotechnical investigation can be challenging and often impracticable. An alternative option is to perform subsurface exploration in front of the proposed wall by means of trench excavation, geotechnical and geological mapping, and horizontal drilling.

Perform at least one, if feasible, horizontal boring into the slope that is to be excavated and drilled for soil nail wall construction to obtain soil and rock specimens and evaluate the cave-in potential of drilled-holes during construction.

6 Design and Analysis

To perform geotechnical design and analysis of soil nail walls use *Snail*, which is a soil nail wall design and analysis software developed, owned and maintained by Caltrans. *Snail* implements the design methodologies of *FHWA GEC No. 7*, and includes features for the design and analysis of the soil nails and structural components of the wall face.

Before using *Snail*, read the *Snail User Guide* and practice the *example files*. Select representative and critical cross sections by reviewing the layout and elevation views of the soil nail wall. Consider applicable excavation heights, geometry, soil and rock profiles, subsurface conditions, and design efficiency when selecting the representative cross sections.

The procedures and issues discussed in this module and *FHWA GEC No. 7* cover only the internal and external (sliding and overturning) stability of soil nails. Global stability of the soil nail wall system is not addressed in this module or *FHWA GEC No. 7*. To make the geotechnical analysis complete, global stability analysis of a soil nail wall must be performed.

7 Typical Design Configuration and Parameters

The following are recommended parameters to start the soil nail wall design.

- Drilled-hole Diameter: 6 inches; increase to 8 inches or larger if necessary, however, drilled-hole diameter greater than 6 inches is rare in soil nail wall construction.

  The drilled-hole diameter entered into the Snail input is only used for calculation and must not be presented in the geotechnical report or the contract plans. According to Caltrans contracting practice, selection of drilled-hole diameter is the contractor's responsibility, and the contractor must demonstrate the selected drilled-hole diameter and associated drilling and grouting methods can provide required nominal pullout resistance, Q_b, via verification and proof tests. To implement this contracting practice, Snail output only shows nominal pullout resistance, Q_b, calculated from the drilled-hole diameter and nominal bond strength. Hence, the Snail output can be included as an attachment to the geotechnical report.

- Soil Nail Length: At least 15 feet and typically 0.7 to 1.0 times designed excavation height, increase as necessary. To facilitate ease of construction and inspection, use a uniform nail length throughout a cross section.

- Soil Nail Inclination: 10° to 15° from horizontal.

  According to *CIRIA C637 Soil Nailing - Best Practice Guidance*, a soil nail installed at 15° below the horizontal has an efficiency of 64% of the nail installed at the optimum angle (35° above horizontal). However, it has nearly twice the length in the resistant

zone and more than four times the average overburden. Therefore, a soil nail inclined slightly downwards is more effective. As a soil nail inclines steeper than 15° the efficiency decreases rapidly without any increase in pullout length or significant increase in overburden. Therefore, the optimum soil nail inclination angle should be between 10° and 15°.

Nail inclination angles less than 10° should be avoided to prevent voids in the grout and an extended "bird's beak" at the nail head. Voids can reduce the pullout resistance of soil nails.

- Wall Face Batter: 1(H):12(V) or any batter angle to account for lateral displacement of the wall face during construction. As a passive reinforcing system, soil nails are expected to be strained during construction. A soil nail wall face that rotates outward from vertical may appear unstable even though the wall is still sound and stable.
- 1st Soil Nail Row: 2.5 feet from the top of excavated face.
- Soil Nail Spacing: 5 feet for both horizontal and vertical spacing; with columnar layout to facilitate the placement of geocomposite drains.
- Nail Bar Diameter and Grade: Use No. 8 and Grade 75 bar;
- Nominal Bond Strength, q_u: Refer to *FHWA GEC No. 7* (Tables 4.4a, 4.4b, 4.5, and 4.6) for suggested nominal bond strength ranges for different soil, rock, and conditions.

  The nominal bond strength entered into the Snail input is only used for calculation. Do not present the nominal bond strength values in the geotechnical report or contract plans. Instead, the nominal pullout resistance, Q_b, which is derived from the nominal bond strength and drilled-hole diameter, should be presented in the geotechnical report and contract plans.
- Horizontal Seismic Coefficient: Follow the *FHWA GEC No. 7* procedure. Use *Caltrans ARS Online* with V_{S30} to determine the Peak Ground Acceleration (PGA) and 1-second response acceleration (S_{D1}). Obtain acceptable wall displacement due to seismic events from the owner of the project or wall, so that the corresponding horizontal seismic coefficient can be determined for the seismic analysis and design of the wall.
- Surcharge: Include live and dead loads, such as traffic and structure loads; consult with Structure Design or District Design.

8 Design Considerations

There are many issues should be considered during the design of a soil nail wall. Some of these issues have been addressed in *FHWA GEC No. 7*. The following are additional issues should be considered.

- For a soil nail wall with a steep slope above the wall, potential rock fall and mud flow issues should be addressed (see Rockfall module).
- In limit equilibrium analysis, the resulting most critical surface must be bracketed by the set search limits to ensure that the search has yield the most critical surface, and there are no other surfaces outside of the search limits have a lower FoS than the

most critical surface found within the set search limits. When the most critical surface is found lay on the edge of the set search limits, extend or move the search limits to capture the actual most critical surface.

When analyzing soil nails with sloping ground above the wall, sometimes the upper point of the most critical surface may persistently lay on the point that defines the upper search limit. As a result, the upper search limit may need to be continually moved further up-slope while the most critical surface cannot be found. When this happened, limit the search to 3 times the excavation height horizontally away from the wall face. The reason for limiting the search to 3 times the excavation height horizontally is that the analysis of soil nails only addresses internal and external stability of the soil nail wall system. A search beyond this limit is considered a global slope stability analysis and should be carried out using a slope stability analysis tool, such as Slide or Slope/W.

- For global slope stability analysis, use a horizontal seismic coefficient equal to 1/3 PGA.
- When a soil nail wall is to be constructed under and in front of structures or facilities that may be sensitive or affected by the lateral displacement of the wall or the settlement of the ground above the soil nail wall, numerical analysis using the software, such as Plaxis and FLAC, should be performed. This working stress analysis is needed to evaluate the expected lateral displacement and settlement of the ground above the wall, and to analyze possible excavation and construction options to limit the displacement and settlement.

9 COMMUNICATION WITH STRUCTURE DESIGNER

Communicate with the structure designer via emails or *geotechnical report (Draft)*, and provide the following information as the first step of design iterations:

1. Elevation-view plan sheets with delineation of wall zones
2. Instruction for placement of soil nails, including
   - Nail array pattern – use columnar layout to facilitate the placement of geocomposite drains
   - Maximum horizontal and vertical soil nail spacing
   - Vertical distance of the soil nails from the top of the excavated face – 2.5 feet (typical), and the bottom of the wall – 2.5 feet (typical)
   - Horizontal distance from the ends of the wall – 2.5 feet (typical)
   - Minimum spacing between soil nails – 2.5 feet (typical), when adjusting soil nail spacing near the bottom and ends of the wall
   - Minimum clearance between soil nails and utilities or obstructions – 2.5 feet (typical)
   - Maximum allowable horizontal and vertical rotation of soil nails from design orientation – 20° (typical), to provide clearance for utilities or obstructions
3. Schedule of soil nail lengths
4. Inclination of the soil nails measured from horizontal
5. Wall face batter measured from vertical

6. Calculated Factor of Safety for internal and global stability (according to Table 5-1, *FHWA GEC No. 7*)
7. Estimated static lateral displacement, if applicable. If the soil nail wall design satisfies the stability requirements of soil nails, then the lateral displacement may be assumed acceptable
8. Nominal pullout resistance Q_b of soil nails in force/unit length, which should be shown on the Plans as the value for Q_b
9. Soil nail bar ASTM designation and grade that arrived at the bar yield strength, the bar diameter entered into Snail, and the required Factor of Safety provided by the structure designer
10. Allowable facing resistance ($F_{unfactored}$) used in Snail calculation and T_o (Note: unfactored facing resistance should always be greater than T_o; unfactored facing resistance = allowable facing resistance from geotechnical ASD calculation).
11. Layout of proof test nails, on the provided elevation-view plan sheets of the wall that are 8 percent of the total number of production soil nails for each wall zone; be aware of the typical location of the geocomposite drains

Work with the structure designer to arrive at an agreed upon unfactored facing resistance ($F_{unfactored}$) used for both geotechnical and structure design.

Obtain plan sheets from the structure designer, and review delineation of wall zones, soil nails and proof test nails layout to ensure the geotechnical design information and recommendations are implemented on the plan sheets.

The following flowcharts present Caltrans design processes of soil nail walls in various conditions according to *BDA 3-9*. The typical conditions that cover most of the soil nail wall design scenarios are present in Figure 2.

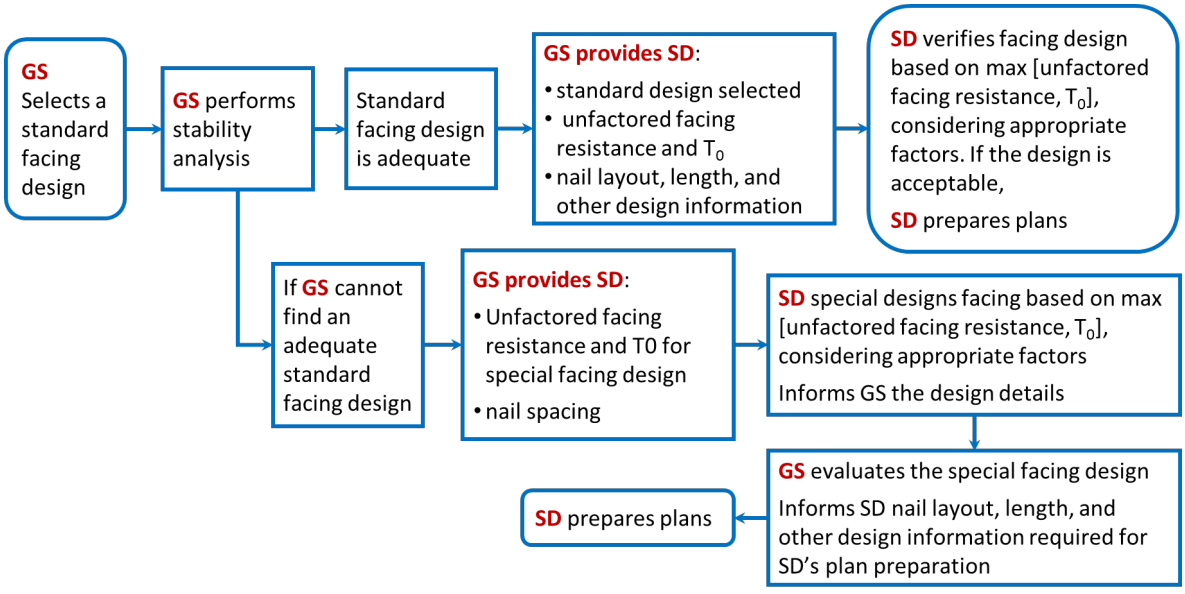


Figure 2 – Design Process for Typical Conditions

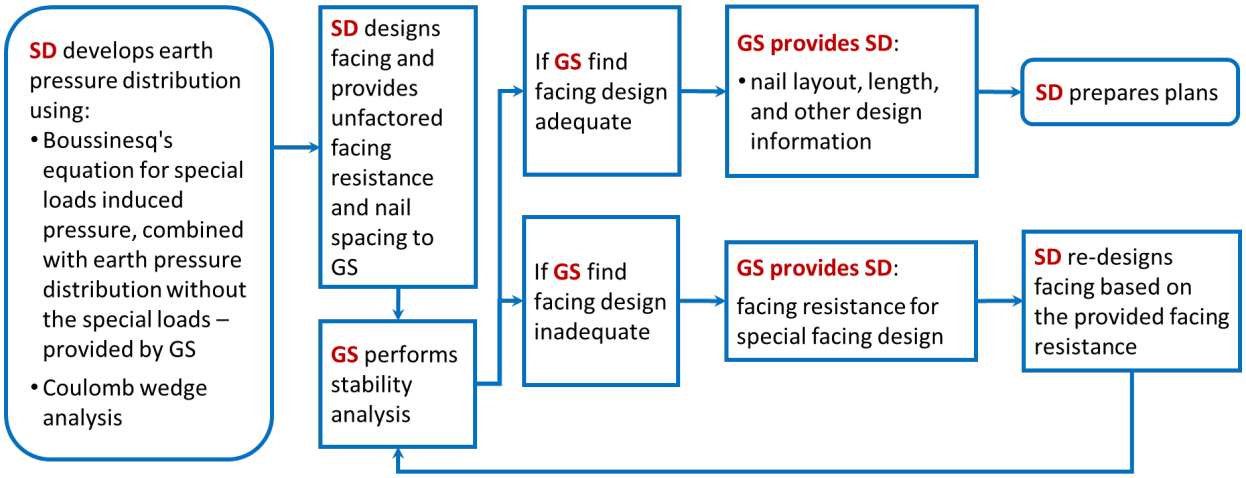


Figure 3 – Design Process for Complex Conditions 1

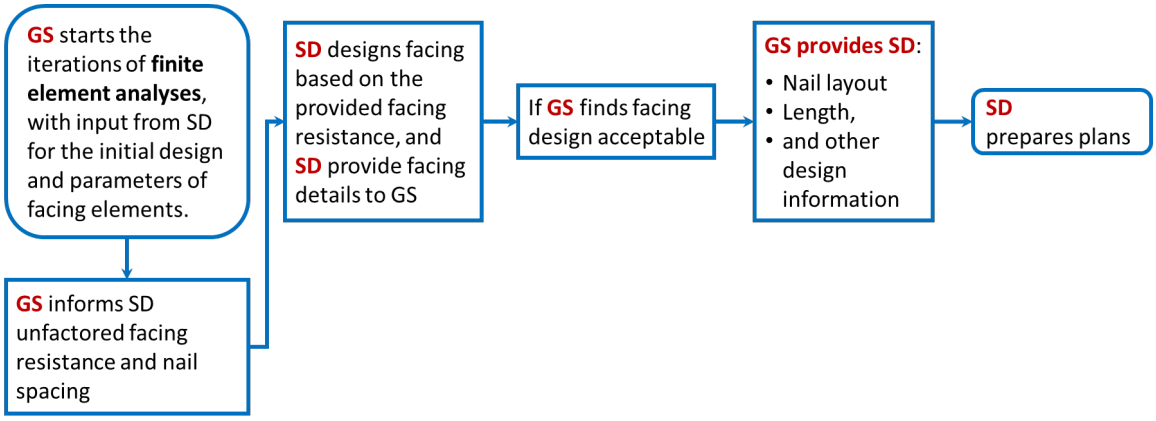


Figure 4 – Design Process for Complex Conditions 2

10 Reporting

Produce geotechnical reports per *Foundation Reports for Earth Retaining Systems* and the requirements herein.

The *Geotechnical Recommendations* section conveys design information to the structure designer, whereas the *Notes for Specifications* conveys information and instruction to the Specifications Engineers for the edits and compilation of Special Provisions.

Provide the following information in the *Geotechnical Recommendations* and *Notes for Specifications* sections of the Foundation Report.

11 GEOTECHNICAL RECOMMENDATIONS

1. Description or schematic design cross sections showing the structure elements, loads, and interpreted subsurface profiles with soil types and layers. For each soil layer, include interpreted soil total unit weights, shear strength parameters, and groundwater conditions
2. Layout and limits of wall zones, or refer to the Plans that show the wall zones
3. Instruction for placement of soil nails, including
   - nail array pattern
   - the maximum horizontal and vertical soil nail spacing
   - the maximum vertical distance of the soil nails from the top of the wall, and the bottom of the wall
   - the maximum horizontal distance from the ends of the wall
   - the minimum spacing between soil nails when adjusting soil nail spacing near the bottom and ends of the wall
   - The minimum clearance between soil nails and utilities or obstructions; and the maximum allowable horizontal and vertical rotation of soil nails from design orientation to provide clearance for utilities or obstructions
4. Schedule of soil nail lengths – recommended
5. Inclination of the soil nails measured from horizontal
6. Wall face batter measured from vertical
7. Calculated Factor of Safety for internal and global stability
8. Estimated lateral displacement, if calculated
9. Nominal pullout resistance Q_b of soil nails in force/unit length, which should be shown on the Plans as the value for Q_b
10. Nail bar yield strength, the bar diameter entered into Snail, and the required Factor of Safety provided by the structure designer
11. Required minimum unfactored facing resistance ($F_{unfactored}$) used to satisfy geotechnical design requirements and T_o. (Note: unfactored facing resistance should always be greater than T_o; unfactored facing resistance = allowable facing resistance from geotechnical ASD calculation). The structure designer must ensure that structural facing design meets or exceeds $F_{unfactored}$.
12. Layout of proof test nails, on the provided elevation-view sheets of the wall that are 8 percent of the total number of production soil nails

Do not layout the verification test nails on the Plans. The locations of verification test nails in each wall zone are to be determined by the contractor, because it is the contractor who determines which location and direction to start constructing each wall zone.

Following examples show the recommended design information in the geotechnical report.

Example 1

General Design Information:

1. Excavation height is the vertical distance from the original grade behind the wall to the bottom of excavation for the wall.
2. Use columnar nail layout pattern.
3. Set soil nail inclination angle at 15 degree from horizontal.
4. Set wall batter at 1(H):12(V).
5. Place the first row of soil nails no more than 2.5 feet below the original grade behind the wall for nail spacing of 5 feet. Place the first row of soil nails no more than 2.0 feet below the original grade behind the wall for nail spacing of 4 feet.
6. Place the bottom row of soil nails no more than 2.5 feet above the bottom of excavation of the wall.
7. For structural wall facing design, apply appropriate structural resistance factor to the required minimum unfactored facing resistance provided in the following table.

Table 1: Section Specific Design Information

Design Excavation Height (ft)	Min. Nail Length (ft)	Max. Vertical Nail Spacing (ft)	Max. Horizontal Nail Spacing (ft)	Nail Bar		Min. Unfactored Facing Resistance (kips)	
				Yield Strength (ksi)	Diameter (inch)	Static	Seismic
Up to 10	15	5	5	75	1.0	27	34
10 to 16	18	5	5	75	1.0	27	34

Table 2 – Nominal Pullout Resistance for Wall Zones

Wall Zone	Station (M Line)	Nominal Pullout Resistance Q_b (lbf/ft)
1	634+12.09 to 635+57.04	2720
2	638+37.28 to 639+82.69	2720

Example 2

General Design Information:

1. Excavation height is the vertical distance from the original grade behind the wall to the bottom of excavation for the wall.
2. Use columnar nail layout pattern.
3. Set soil nail inclination angle at 15 degree from horizontal.
4. Set wall batter at 1(H):12(V).
5. Place the first row of soil nails no more than 2.5 feet below the original grade behind the wall for nail spacing of 5 feet. Place the first row of soil nails no more than 2.0 feet below the original grade behind the wall for nail spacing of 4 feet.
6. Place the bottom row of soil nails no more than 2.5 feet above the bottom of excavation of the wall.

7. For structural wall facing design, apply appropriate structural resistance factor to the required minimum unfactored facing resistance provided in the following table.

Table 3: Section Specific Design Information

Wall Zone	Station ("B" Line)	Max. Excavation Height (ft)	Min. Nail Length (ft)	Max. Vertical Nail Spacing (ft)	Max. Horizontal Nail Spacing (ft)	Nail Bar		Nominal Pullout Resistance Q_b (lbf/ft)	Min. Unfactored Facing Resistance (kips)	
						Yield Strength (ksi)	Diameter (inch)		Static	Seismic
1	2093+75 to 2109+00	8.5	15	5	5	75	1.0	2,720	30	38
	2109+00 to 2121+00	11.7	20	5	5	75	1.0	2,720	30	38
2	2121+00 to 2127+50	34.5	35	5	5	75	1.0	3,200	30	38
	2127+50 to 2127+96	34.4	30	5	5	75	1.0	3,200	30	38
3	2127+96 to 2129+50	30.5	25	4	4	75	1.0	3,200	30	38
	2129+50 to 2130+50	26.6	20	4	4	75	1.0	3,200	30	38
4	2130+50 to 2134+73	23.1	15	4	4	75	1.0	3,200	30	38

12 Notes for Specifications

Follow the instructions provided in the Geotechnical Notes for Specifications module in the Geotechnical Manual for the information and recommendations should be provided in this section.

Send the Soil Nail Verification Test nSSP (section 46-3), available on the NSSP for Geotechnical Design intranet page, to the Specification Engineer so it can be included in the contract Special Provisions of the project. This nSSP implements the maximum pullout load of 3 × Estimated Nominal Pullout Resistance (Test Load), with the goal of pulling the verification test nails to failure. The objective is to obtain actual nominal pullout resistance of soil nails in the soil/rock conditions at the site and with the construction methods provided.

13 Geotechnical Review for Final Structure PS&E (Expedite Notice)

According to *Structure Design Expedite Notice* process, a Geotechnical Review must take place after the issuance of Draft Structure PS&E and before the start of Final Structure PS&E (Expedite Notice) process.

During Geotechnical Review, review and verify that draft Plans and Special Provisions have implemented the recommendations of the Foundation Report. Geotechnical reports

are part of the contract. Any inconsistency between the geotechnical reports, Plans, and Special Provisions can cause serious problems during construction.

14 Geotechnical Tasks during Construction

15 Pre-Construction Meeting

Contact the Resident Engineer (RE) and Structure Representative to attend pre-construction meeting with or without the Contractor. Be prepared to discuss and answer questions related to the design and construction of the soil nail walls, including Standard Specifications and Special Provisions Sections 19 and 46.

16 Shop Drawing Review

Before construction, Shop Drawings will be submitted by the contractor for review, as described in Sections 19-3.01C(4) and 46-1.01C(2) of *Standard Specifications*. Review the following items in the Shop Drawings and provide comments to the Structure Representative. Do not direct the means and methods of construction as these are the responsibility of the contractor.

Under Section 19 Earthwork:

- Soil parameters used for stability analysis,
- Stability analysis of proposed excavation lifts, and
- Proposed *stability test* locations.

Under Section 46 Ground Anchor and Soil Nail Wall:

Refer to the Contract Plans while reviewing the proposed test nail details in the Shop Drawings, including:

- Proposed drilled-hole diameter,
- Estimated nominal bond strength,
- Converted nominal pullout resistance,
- Verification and proof test details,
- Quantity of test nails, and
- Soil nail bar grade and diameter.

17 Test Nail Results Review and Archive

Request the Structure Representative to forward all soil nail test data, irrespective of whether the test nails passed or failed. Reviewing and collecting test nail data is the only means to evaluate the estimate of pullout resistance during design, and to calibrate for future design.

When a soil nail wall is completed, the contractor need to send an email with the soil nail test results as a tabulated spreadsheet to the Engineer and Geotechnical.Data@dot.ca.gov (Section 46-1.01C(3) of *Standard Specifications*). Obtain the test results from Geotechnical.Data@dot.ca.gov and archive the test results in GeoDOG.

18 TYPICAL CONSTRUCTION ISSUES

Site conditions, diligence of the design deliberation, and competency of the contractor dictate whether construction difficulties or issues will arise during construction. Typical issues encountered during soil nail wall construction include:

- Sloughing of excavated face due to dry sandy materials
- Desiccation, weathering, and instability of the excavated face due to prolonged exposure after excavation
- Creeping and continual movement of the wall face due to high-plastic clay
- Difficult drilling through cobbles and boulders
- Caving in of the drilled-holes
- Excessive groundwater seepage and poor control of surface water and groundwater
- Construction mistakes, poor workmanship, and incorrect installation
- The original grade (OG) line as shown on the Plans does not match and is above the top of the excavated face at the site
- Failed verification and proof tests

Work closely with the Structure Representative and be prepared to promptly answer calls and address Request for Information (RFI) to resolve construction issues in a timely manner.

19 TECHNICAL NOTES

This section provides addition design insights to be considered during design of soil nail walls.

20 CORROSION PROTECTION OF SOIL NAILS

The long-term stability of a soil nail wall or slope primarily depends on the corrosion protection of the soil nails that protect the structural integrity of the soil nail bars.

The grout surrounding the soil nails should not be relied on as a water barrier as the grout column will develop cracks under tensile stress. To develop pullout resistance, the inherent function of soil nails, the grout, as a medium between the soil/grout interface and soil nail bar, will sustain tensile stress and ultimately develop cracks to transfer the stress. Subsequently, surrounding water and moisture will infiltrate through these cracks and reach the soil nail bars.

The following three-volume articles published by *Belgian Building Research Institute* provide excellent and comprehensive detailed pullout test data, physical measurement and photos of the exhumed ground anchors, grout columns, and developed cracks. Even though these articles are solely for ground anchors, the mechanisms of grout/ground and grout/tendon are the same for both ground anchors and soil nails, and applicable to soil nails.

- *Proceedings International Symposium – Ground Anchors, Limelette test field results, May 14, 2008, volume 1, volume 2, volume 3*

21 NOMINAL STRENGTH

Nominal strength can be best defined as: the capacity of a structure or component to resist the effects of loads, as determined by computations using specified material strengths (such as yield strength, f_y, or ultimate strength, f_u) and dimensions and formulas derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Nominal strength of a batch of construction material, such as steel and concrete, is a strength value derived from testing to failure of specimens sampled from that batch. Even though the reported nominal strength values are typically the nearest rounded-down customary value from the minimum tested strength values, the reported nominal strength is still inherently correlated to the probability density function of the material. For example, the strength distribution of an ASTM A36 steel production batch should be mostly greater than the nominal yield strength of 36,000 psi; i.e. near 100% probability that the ASTM A36 steel has a yield strength of greater than 36,000 psi, the nominal strength.

There is a much more clearly defined material strength value that is based on statistical concept, the characteristic strength. The characteristic strength is defined as: the strength

of the material below which not more than 5% of the test results are expected to fall. Sometimes, the characteristic strength is selected as the nominal strength of a material.

In any event, test-to-failure data is needed to establish the strength probability density function of a construction material or construction components. Establishing a strength probability density function for construction materials and construction components is a major and necessary step to truly implementing LRFD, and to assigning Factor of Safety under ASD. Without the strength probability density function based on test-to-failure data, the design practice can rely only on theory and combined with observed performance of prior construction.

Among geotechnical construction components, very few, if there is any, have an established and direct probability density function. This is because it is physically, financially, and contractually very difficult to test to failure a geotechnical component, such as a driven pile, let alone to test to failure a batch of these components. Therefore, the nominal strength of geotechnical construction components that based on directly measured strength or performance probability density functions is rarely available, if it is not non-existence.

Hence, almost all the nominal strengths used for geotechnical construction components are established based on theory, inferred from basic soil and rock properties, and combined with observed performance. Very few of these nominal strength values have been verified by test to failure.

22 Nominal Strength and Pullout Resistance of Soil Nails

In soil nail construction, statistically significant amount of sacrificial soil nails are required to be tested to and pass the nominal pullout resistance in order to satisfy the acceptance criteria. The implemented test regime provides relatively higher confidence for constructed soil nails than that for other geotechnical components.

However, tests that stop short of reaching failure cannot be used to establish the strength probability density function that can verify the reasonableness of the selected nominal pullout resistance.

23 Improvement in Interpreting Nominal Strength from Subsurface Exploration

The discussion in the previous section has not addressed the issue of how to interpret nominal pullout resistance based on field and laboratory tests during design. Currently, the often-quoted references on this subject are the tables (Tables 4.4a, 4.4b, 4.5, and 4.6) from *FHWA GEC No. 7*. However, the information presented in these tables need to be updated and improved.

First of all, there is a need for clarification and agreement on where the presented strength values reside in terms of the probability density function of the particular soils and rocks. Some may consider these values as the average values compiled from collected data, which is naturally the case when presenting summary of findings. However, during construction, the values selected for design, mostly referenced from these tables, are the

construction acceptance criteria – the absolute lower bound according to typical construction contract language and the de facto nominal strength values. Understanding this potential disconnect is needed when referencing these values.

Then, clearly defined nominal strength with respect to the probability density function needs to be established and agreed upon. Meanwhile, we need a concerted effort to continually accumulate engineering properties of soils and rocks from laboratory and in-situ tests and interpreted nominal pullout strength and associated design parameters of these soils and rocks. Only after we compared the interpreted nominal strengths with the nominal strengths obtained from soil nail pullout tests and other tests during construction, can we calibrate our practice. The first step taken in Caltrans is implementing the requirement of emailing all the pullout test results as a tabulated spreadsheet by the contractor to the Engineer and Geotechnical.Data@dot.ca.gov after completion of a wall construction.

This above discussion offers a general direction needed to prepare for the gradual implementation of LRFD for soil nails. It can take years, and probably decades, and requires gradual improvement to our geotechnical investigation practice for soil nail design.

Current soil nail design practices all apply various assumptions to simplify a complex composite system to comprehensible models so that workable design procedures can be implemented. Be diligent, aware of these assumptions, and be involved throughout the design, contract development, and construction phases to continually improve on the understanding of soil nail design and construction.

Non-Gravity Cantilever Earth Retaining Systems (ERS)

Non-gravity cantilever ERS have vertical structural elements embedded below design grade to establish the necessary lateral resistance through passive earth pressure to counteract the lateral earth pressures. The vertical structural elements can be either discrete piles such as soldier piles, or continuous piles such as sheet piles, tangent piles, and secant piles (Figure 1 and Figure 2). The piles can be installed by driving, vibrating, or drilling. Steel soldier piles and sheet piles are the most commonly used non-gravity cantilever ERS in Caltrans projects. Tangent and secant piles (Cast-In-Drilled Hole piles) are also used in Caltrans projects related to embankment slope or landslide mitigation, and seawalls.

Advantages of non-gravity cantilever ERS include:

- Less right of way needed than competing systems, such as soil nails
- Less disruptive to traffic than ERS using bottom-up construction methods
- Less environmental impact than ERS using bottom-up wall methods
- Relatively fast construction
- Good performance under seismic loading

Some advantages such as “less disruptive” and “relatively fast construction” may not be applicable to tangent and secant pile walls, depending on drilled-hole size and subsurface conditions.

Non-gravity cantilever ERS are not feasible when there are:

- Stringent requirements that limit the wall movement during construction (e.g., the proposed wall is adjacent to or below a critical structure, such as a bridge abutment)
- Design wall heights greater than 15 feet (for such cases, non-gravity cantilever ERS are not a cost-effective option, requiring larger piles or smaller pile spacing)

Favorable subsurface conditions for non-gravity cantilever ERS are:

- Excavated face can stand unsupported and stable until the lagging is placed for soldier pile walls
- Drilled-holes can remain open and stable without casing until the piles are installed and the concrete is placed
- Granular soils, stiff clays, and weathered bedrock with favorable bedding planes

Unfavorable subsurface conditions for non-gravity cantilever ERS include:

- Soft highly plastic clay, organic soil, collapsible soil, expansive soil, cobbles and boulders, weathered rock with unfavorable bedding planes, strong rock
- Groundwater table is above design grade for soldier pile walls

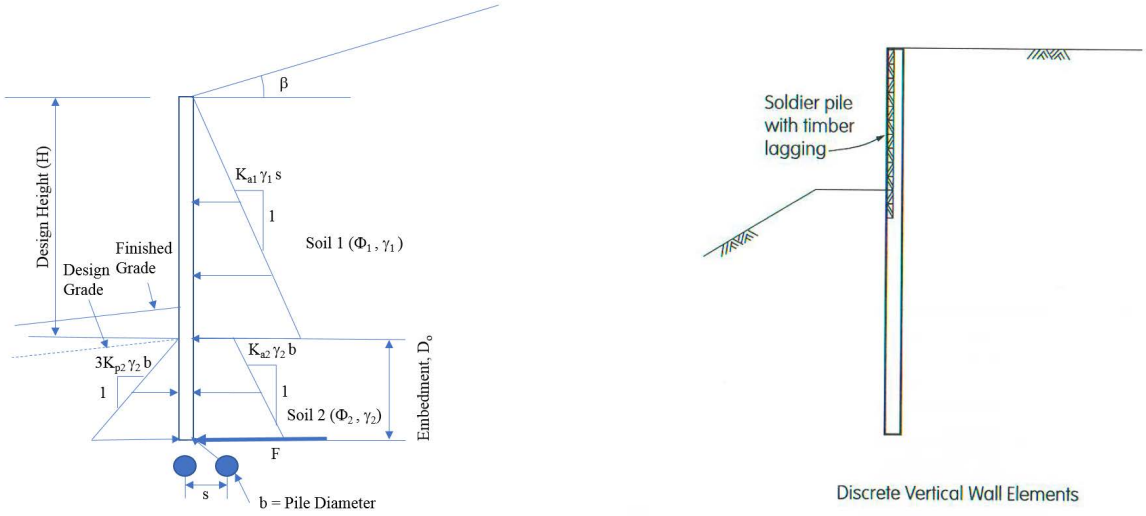


Figure 1: Typical Non-Gravity Cantilever ERS with Discrete Vertical Wall Elements Embedded in Granular Soils (Modified after AASHTO LRFD BDS, 2012 and Caltrans BDS, 2004)

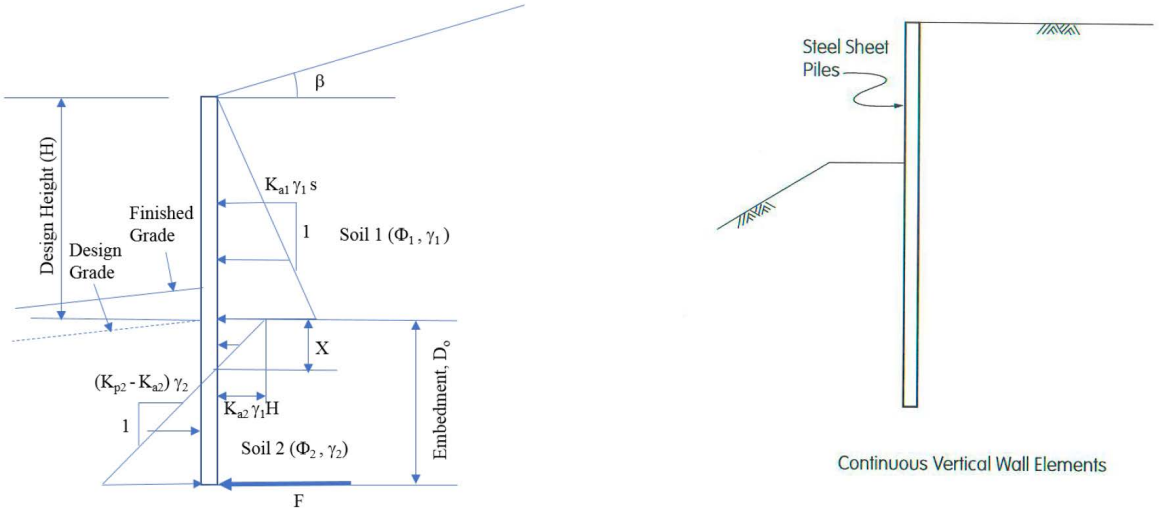


Figure 2: Typical Non-Gravity Cantilever ERS with Continuous Vertical Wall Elements Embedded in Granular Soils (Modified after AASHTO LRFD BDS, 2012 and Caltrans BDS, 2004)

Reference Manuals for Design and Communication

For non-gravity cantilever ERS design, use this module and the following design manuals and guidelines:

- AASHTO LRFD Bridge Design Specifications and California Amendments
- Memo to Designers 5-19, "Earth Retaining Systems Communication"
- Geotechnical Manual, "Foundation Reports for Earth Retaining Systems"
- Geotechnical Manual, "Seismic Design of Earth Retaining Systems"

For design cases where guidance provided in the above documents is not applicable, refer to other FHWA reference manuals including *FHWA NHI-05-094*, "LRFD for Highway Substructures and Earth Retaining Structure."

Responsibilities for Design

Geotechnical Services' responsibilities in the design of non-gravity cantilever ERS are:

- Develop interpreted subsurface cross sections. For a long wall, several subsurface cross sections along the alignment may be needed
- Estimate engineering properties such as unit weight, cohesion, friction angle, and associated lateral earth pressure coefficients
- Analyze the magnitude and distribution of lateral earth pressure for complex walls where conventional earth pressure theory does not apply, or when requested by the structure designer
- Determine the minimum pile embedment depth based on global stability requirements and required axial capacity, when pile elements are used

Information that should be provided by the structure designer for the geotechnical analysis and design are:

- Plans showing the location of the wall (beginning, end, length, and alignment)
- Elevation view of the wall (maximum and minimum design heights)
- Cross sections of the wall (for example, every 10 to 50 feet)

For the design of non-gravity cantilever ERS, the Geoprofessional (GP) should assist the structure designer in estimating all applicable lateral pressures against the wall including static and seismic earth pressures, lateral pressures induced by surcharge load, and hydrostatic pressure.

Investigations

Perform a geotechnical investigation to determine subsurface conditions that may affect the selection, design, and construction of the non-gravity cantilever ERS, including:

- Strength and deformation characteristics of foundation soil and rock
- Strength and weight of soil and rock to be retained

- Corrosion potential of soil in contact with the retaining wall
- Groundwater location
- Quantity of groundwater seepage

To plan for and carry out a geotechnical investigation refer to:

- Geotechnical Manual, “Geotechnical Investigations”
- Geotechnical Manual, “Soil and Rock Logging, Classification and Presentation Manual”
- Geotechnical Manual, “Borehole Location”
- FHWA NHI-01-031 Subsurface Investigations “Geotechnical Site Characterization”

Typical borehole spacing is about every 100 to 200 feet along the proposed wall alignment, with boreholes strategically positioned in front, behind, and directly on the retaining wall layout line. The number of boreholes should be reduced or increased based on the quality of existing data, uniformity of site geology, and the quality of site-specific geologic mapping. The depth of investigation should be more than two times the design height of the ERS or 20 feet below the ERS design grade, whichever is the greatest.

Design

The geotechnical design of a non-gravity cantilever ERS must meet displacement and stability requirements for the following limit states defined by LRFD design methodology:

- Service Limit State – movement and overall (global) stability (*AASHTO LRFD BDS* Article 11.8.3)
- Strength Limit State – overall stability - soil passive failure due to insufficient vertical element embedment (*AASHTO LRFD BDS* Article 11.8.4)
- Extreme Limit State - overall stability - soil seismic passive failure due to insufficient vertical element embedment, and overall/global stability (*AASHTO LRFD BDS* Articles 11.8.6)

For each of the limit states, the load and resistance factors should be applied in accordance with the *AASHTO LRFD BDS* Articles 3.4.1 (Table 3.4.1-1) and 11.5.6, and *California Amendments* (Table 11.5.7-1).

This module is developed for non-gravity cantilever ERS constructed by the top-down construction method.

Calculation of Lateral Pressure

The magnitude and distribution of lateral pressures depends upon the wall type, wall movement, wall geometry/stiffness, friction at the wall-soil interface, retained soil type, groundwater conditions, earth surcharge/sloping ground conditions, traffic, and

construction-related live load surcharge. These factors should be considered in the calculation of the lateral pressure.

Static Lateral Pressure

- Active Earth Pressure

  Use *AASHTO LRFD BDS* Article 3.11.5 to calculate the active earth pressure coefficient and associated earth pressure. According to Article 3.11.5, the active earth pressure coefficient and associated earth pressure can be calculated using Coulomb, Rankine, or trial wedge methods. For the design of non-gravity cantilever ERS, Rankine and Coulomb methods are typically used. When these methods cannot be used due to complex geometry or multiple soil layers, the trial wedge method should be used.

  For walls with cohesive soil conditions, both the short-term condition (undrained) and long-term (drained) condition should be evaluated. In addition, the magnitude of active earth pressure should be at least 0.25 times the effective overburden pressure at any depth, or 0.035 ksf/ft of wall height, whichever is greater (*AASHTO LRFD BDS* Article 3.11.5.6).

  For the distribution of active earth pressure in different soils and rocks, refer to Figures 3.11.5.6-1 through 3.11.5.6-7 of *AASHTO LRFD BDS* 3.11.5.6.

- Surcharge Load

  Use *AASHTO LRFD BDS* Article 3.11.6 for the lateral pressure against the wall induced by surcharge loads.

- Hydrostatic Water Pressure

  When water is present behind the wall, hydrostatic water pressure must be considered in addition to other lateral earth pressures. When the hydrostatic water pressure is applied to the wall, calculate the lateral earth pressure with effective unit weight of soils. For continuous wall facing such as sheet piles, tangent piles, and secant piles, hydrostatic pressure differentials can develop between the back and front of the wall face and can cause seepage or piping (boiling) in cohesionless soils. For such cases, perform seepage analyses using a flow net or numerical analysis to ensure base stability against seepage. Section 5.2.9 of *Geotechnical Engineering Circular No. 4*, “Ground Anchors and Anchored Systems” presents procedures to calculate porewater pressure considering the effects of seepage (Equations 15 to 17, and Figure 32), and a simplified flow net for homogeneous soil (Figure 31). For non-homogeneous soil or special drainage conditions which may alter water boundary conditions, use numerical seepage analysis.

- Passive Earth Pressure

  For the calculation of passive earth pressure, use the log-spiral method with an appropriate wall interface friction angle. When the log-spiral method can’t be used due to complex geometry or multiple soil layers, use the trial wedge method with

the wall interface friction angle no greater than 0.5□ according to *AASHTO LRFD BDS* Article 3.11.5.4, where □ is friction angle of the soil.

When Figures 3.11.5.6-1, 2, 4 and 5 in *AASHTO LRFD BDS* are used for calculation of passive earth pressure against embedded discrete vertical elements, use the Rankine passive earth pressure coefficient. The figures were developed based on Broms' Method (1964), in which the Rankine passive earth pressure coefficient was used to derive the effective width of 3 times the pile diameter. When different soil arching factors need to be used in the design, refer to *AAASHTO LRFD BDS* C11.8.6.3.

Seismic Lateral Pressure

For the calculation of a seismic lateral earth pressure, use *AASHTO LRFD BDS* Articles 11.6.5.2, 11.8.6 and 11.9.6. All the seismic loads for the design of non-gravity cantilever ERS are shown on Figure 11.8.6.2-1 of *AASHTO LRFD BDS*. As shown on Figure 11.8.6.2-1, the seismic active earth pressure resultant, P_{AE} is considered only above the design grade, while the P_A and P_{PE} are applied below the design grade. In addition, according to *AASHTO LRFD BDS* C11.8.6.2, for walls with continuous vertical elements, such as sheet pile walls, tangent pile walls, and secant pile walls, the P_{AE} will be distributed from the top of the wall to the point where the critical failure line of Limit Equilibrium (LE) analysis intersects with the vertical element below the design grade. For a wall supporting seismically critical structures, the seismic active earth pressure should be extended below the design grade to the toe of the vertical element to account for the uncertainties in the seismic distribution below the design grade.

- Seismic Active Earth Pressure

  For the calculation of the seismic active earth pressure, use *AASHTO LRFD BDS* Article 11.6.5.3. The article presents three methods to calculate the seismic active earth pressure: Mononobe-Okabe (M-O) method, trial wedge method, and Limit Equilibrium (LE) method. For wall geometry or site and soil conditions where the M-O method is not suitable, either the trial wedge method or LE method can be used.

  Because horizontal acceleration and resulting seismic earth pressure greatly depends on the magnitude of seismic displacement of a non-gravity cantilever ERS, consult the structure designer for the tolerable permanent seismic displacement of a non-gravity cantilever ERS, and estimate the horizontal acceleration coefficient (k_h). The k_h associated with the tolerable permanent seismic displacement should be used for calculating seismic active earth pressures.

  When the wall displacement analyses using a numerical method such as beam-column approach is to be performed by the structure engineer, provide soil spring parameters such as p-y curves according to *AASHTO LRFD BDS* Article C11.8.6.4.

- Seismic Passive Earth Pressure

  For the calculation of the seismic passive earth pressures, use *AASHTO LRFD BDS* Articles 11.8.6.3 and A11.4. In these articles, the log spiral or the nonlinear failure surface are recommended to be used with the wall interface friction. Do not use the M-O method for estimating the seismic passive earth pressure. According to the *AASHTO LRFD BDS* Article 11.6.5.5, a wall interface friction equal to two-thirds of the soil friction angle can be used for the calculation of the seismic passive pressure when there are no specific guidance or research results for a seismic wall interface friction. For the wall with an embedment depth of less than 5 feet, use the static method for the calculation of the seismic passive pressure according to *AASHTO LRFD BDS* Article 11.6.5.5.

Limit State Design

The design of the non-gravity cantilever ERS must meet displacement and stability requirements for each limit state below, and appropriate scour shall be considered in each limit state for the walls located in flood prone areas. For the guidance related to scour evaluation, refer to the *AASHTO LRFD BDS* Article 11.7.2.3.

Service Limit State

Displacement

The design of the wall must ensure that the vertical and lateral displacement does not affect the performance of the wall system (*AASHTO LRFD BDS* Article 11.8.3.1). For non-gravity cantilever ERS constructed by excavating a vertical face, there is no vertical stress increase in soils, and settlement due to net vertical stress does not occur.

For a wall supporting low-displacement tolerance structures, numerical analysis such as the finite element or finite different methods may be required.

Global Stability

The global stability is evaluated using LE slope stability analysis such as Morgenstern-Price, Modified Bishop, Janbu, or Spencer methods. For the resistance factors used in the global stability, refer to *AASHTO LRFD BDS* Article 11.6.2.3.

Strength Limit State

Soil passive failure due to insufficient vertical element embedment

This analysis is the structure designer’s responsibility. For soil passive failure, refer to *AASHTO LRFD BDS* Article 11.8.4.

Extreme Limit State

Seismic soil passive failure due to insufficient vertical element embedment

This analysis is the structure designer's responsibility. For seismic soil passive failure, refer to *AASHTO LRFD BDS* Article 11.8.4 and 11.8.6.

Seismic Global Stability

For the seismic global stability analysis, use *AASHTO LRFD BDS* Articles 11.8.6.1 and 11.9.3.2. There are two benchmark values for k_h, depending on tolerable seismic displacement limits, 1/2 or 1/3 horizontal peak ground acceleration (HPGA) which is the acceleration at zero period (T = 0 second) calculated from Caltrans ARS Online (v.2.3.09). Use 1/3 HPGA for the k_h if an expected mean seismic displacement of 5.0 inches is acceptable. Use 1/2 HPGA for the k_h if the 2.0-inch seismic displacement is acceptable. If the seismic global stability is not satisfied with the benchmark value of 1/2 or 1/3 HPGA, use the following steps:

1. Find the horizontal yield acceleration coefficient (k_y) using iterative LE slope stability analyses.
2. Perform seismic displacement analysis using simplified seismic displacement method according to *AASHTO LRFD BDS* Appendix A11.
3. Consult the structure designer for calculated and tolerable seismic displacement.

Reporting

Report non-gravity cantilever ERS recommendations in accordance with *Foundation Reports for Earth Retaining Systems (ERS).* Include design information, and assumptions presented in the previous sections. For geotechnical recommendations, refer to Section 3.12.2.2, *Non-Gravity Cantilever Systems*, of the *FR for ERS*.

Ground Anchor Earth Retaining Systems (ERS)

This module presents Caltrans practice for geotechnical investigation, design, and reporting of ground anchor ERS. A brief discussion of the design policy and procedures on Deadman ERS is also included at the end of the design section. In this module, ground anchors are referred to as sub-horizontal ground anchors.

A ground anchor ERS is constructed by installing rows of ground anchors to resist lateral loads acting on the wall. They are commonly used for embankment roadway re-construction and stabilization where space for re-grading is limited. Soldier pile ground anchor walls are most commonly used in Caltrans; concrete diaphragm ground anchor walls are commonly used for Caltrans roadway widening projects under the bridge abutments (see *Memo to Designers 5-12*, "Earth Retaining Systems Using Ground Anchors"). A ground anchor system consists of three components: anchorage (anchor head assembly), free stressing tendon (unbonded length), and anchor (bonded length) (Figure 1). Ground anchors provide lateral resistance by pre-stressing the tendon from the anchor to the anchorage at the wall facing. The ground anchor tendon can be either steel bars or steel strands; steel strands are more commonly used as they can provide higher tensile resistance and be easily handled during installation.

Design and performance advantages of ground anchor ERS include:

- Cost effective compared with most other ERS using top-down construction methods
- Less disruptive to traffic than ERS using bottom-up construction methods
- Less environmental impact than ERS using bottom-up construction method
- Quality assurance through performance or proof tests on each anchor
- 'Active' system to minimize lateral movement during excavation by pre-stressing of each anchor

Ground anchor ERS are unfavorable when there are:

- Permanent easement requirements
- Underground utilities or structures within the anchor zone

Favorable subsurface conditions for ground anchor ERS are:

- Excavated face can stand unsupported and stable until the facing is structurally complete
- Drilled-holes can remain open and stable without casing until the anchor is placed and grouted.

Unfavorable subsurface conditions for ground anchor ERS include:

- Soft, highly plastic clay, organic soil, collapsible soil, expansive soil, cobbles and boulders, weathered rock with unfavorable bedding planes
- Groundwater table is above design grade for soldier pile ground anchor walls
- Very weak soils behind the wall facing

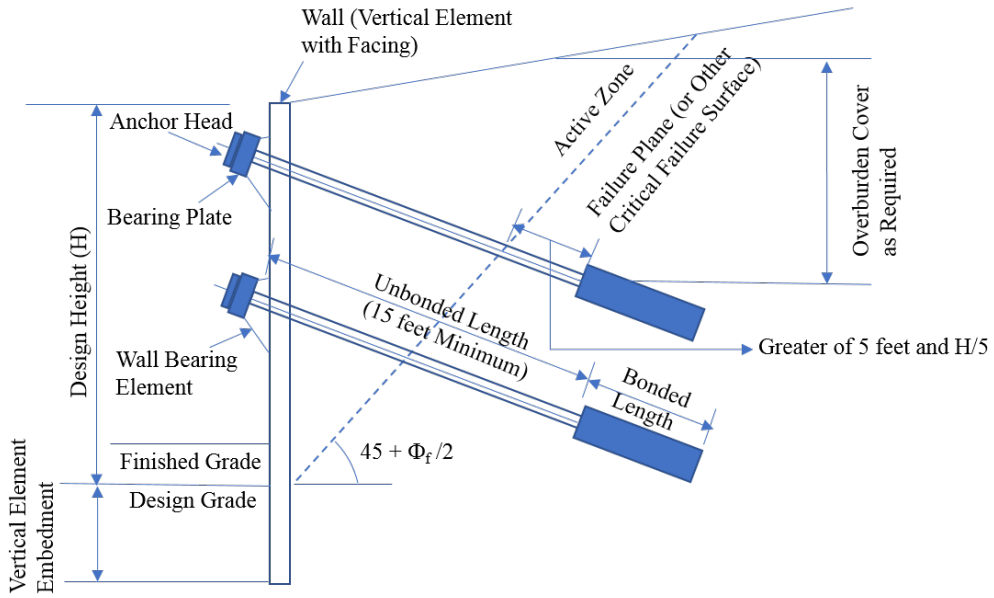


Figure 1: Ground Anchor ERS Components and Dimensions (Modified after AASHTO LRFD BDS)

Design Manuals and Guidelines for Design and Reporting

For ground anchor ERS design, use this module, and the following design manuals and guidelines:

- *2017 (8th Edition) AASHTO LRFD Bridge Design Specifications* and *2019 California Amendments* hereafter "AASHTO"
- *Memo to Designers 5-12*, "Earth Retaining Systems Using Ground Anchors"
- *Memo to Designers 5-19*, "Earth Retaining Systems Communication"
- *Geotechnical Manual*, "Foundation Reports for Earth Retaining Systems"
- *Geotechnical Manual,* "Geotechnical Seismic Design of Earth Retaining Systems"

For design cases where guidance provided in the above documents is not applicable, refer to other FHWA reference manuals including *FHWA NHI-05-094*, "LRFD for Highway Substructures and Earth Retaining Structure" *and FHWA Geotechnical Engineer Circular No. 4*, "Ground Anchors and Anchored Systems".

Geotechnical Services' Responsibilities for Design

Geotechnical Services' responsibilities in the design of ground anchor ERS are:

- Develop interpreted subsurface cross sections; for a long wall, development of more than one cross section along alignment, depending on variability of surface conditions, may be needed
- Determine engineering properties of subsurface materials such as unit weight, cohesion or undrained shear strength, friction angle, and associated lateral earth pressure coefficients
- Analyze lateral earth pressure and its distribution for complex wall geometries when conventional earth pressure theories are not applicable or when requested by the structure designer
- Analyze minimum pile embedment depth based on global stability requirement and required axial capacity, if pile elements are used.
- Analyze anchor unbonded zone length based on conventional earth pressure theory if applicable, or slope stability analysis
- Assist the structure designer or the district project engineer in determining necessary permanent easements

Information that should be provided by the structure designer for geotechnical analysis and design are:

- Plans showing the location of wall (begin and end, length and alignment)
- Elevation view of wall (maximum and minimum design height)
- Cross sections of wall (e.g., every 10 to 50 feet)
- Topography with cross sections depicting the slip-out or erosion of slope face for slope stabilization application (this information can be directly provided by District)

For the design of ground anchor ERS, the geoprofessional should assist the structure designer in providing all applicable lateral pressures against the wall including static and seismic earth pressure, lateral pressures induced by surcharge load, and hydrostatic pressure.

Geotechnical Investigation

Perform a geotechnical investigation to determine subsurface conditions that may affect the selection, design, and construction of the ground anchor ERS, including:

- Strength and deformation characteristics of foundation soils and rocks
- Strength and weight of soils and rocks to be retained
- Corrosion potential of soils in contact with the retaining wall
- Groundwater location
- Quantity of groundwater seepage

To plan for and carry out the geotechnical investigation, including laboratory testing, refer to the following:

- *Geotechnical Manual*, “Borehole Location”
- *Geotechnical Manual*, “Soil and Rock Logging, Classification and Presentation Manual”
- *Geotechnical Manual*, “Geotechnical Investigation”
- *FHWA NHI-01-031 Subsurface Investigations* “Geotechnical Site Characterization”

Typical borehole spacing is about every 100 to 200 feet along the proposed wall alignment, with boreholes strategically positioned in front, behind, and directly on the retaining wall layout line. Perform at least one horizontal boring, if feasible, into the slope that is to be excavated and drilled for ground anchor wall construction to obtain soil and rock specimens and evaluate the caving potential of drilled holes during construction. The number of boreholes may be reduced or increased based on the quality of existing data, uniformity of site geology, and the quality of site-specific geologic mapping. The depth of investigation should be more than two times the design height of the ERS or 20 feet below the ERS design grade, whichever is greater.

Design

The geotechnical design of a ground anchor ERS must meet displacement and stability requirements for the following limit states defined by LRFD design methodology:

- Service Limit State – movement and overall (global) stability (*AASHTO LRFD BDS* Article 11.9.3)
- Strength Limit State – stability against soil failures such as bearing failure (geotechnical axial capacity of vertical elements), anchor pullout failure, and passive failure due to insufficient vertical element embedment (*AASHTO LRFD BDS* Article 11.9.4)
- Extreme Limit State - stability against soil failures such as bearing failure (geotechnical axial capacity of vertical element), anchor pullout failure, and passive failure due to insufficient vertical element embedment, and overall/global stability (*AASHTO LRFD BDS* Articles 11.9.4 and 11.9.6)

For each of the limit states, the load and resistance factors should be applied in accordance with AASHTO 3.4.1 (Table 3.4.1-1) and 11.5.6 and *California Amendments* (Table 11.5.7-1).

Calculation of Lateral Pressure

The magnitude and distribution of lateral pressures depend upon wall type, wall movement, wall geometry/stiffness, friction at wall-soil interface, retained soil type, groundwater conditions, earth surcharge/sloping ground conditions, traffic, and construction related live load surcharge. These factors should be considered for the calculation of lateral pressures.

Static Lateral Pressure

- Apparent Earth Pressure (AEP)

  Calculation of AEP must follow AASHTO 3.11.5.7, and *Caltrans MTD 5-12*. The AEP was developed from top-down construction with constraining or limiting wall deformation and should be used whenever the construction method meets these conditions. The distribution of AEP should be based on construction method and sequence, stiffness of wall and anchor system, type and properties of soils, pile and anchor spacing, and expected displacement, etc.

  For simple wall geometry, and subsurface profiles in which the Rankine or Coulomb active earth pressure coefficient can be used for calculation of AEP, use AASHTO Equations 3.11.5.7.1-1 to 3.11.5.7.2b-2 and AASHTO Figures 3.11.5.7.1-1 and 3.11.5.7.2b-1. When calculating active earth pressure coefficient, k_a, the wall interface friction angle should be equal to zero for both Rankine and Coulomb. For walls with cohesive soil conditions, both short-term condition (undrained) and long-term (drained) condition should be evaluated, and AEP should be developed for the condition, whichever results in greater lateral force.

  For complex wall geometry, or subsurface profiles in which the Rankine or Coulomb earth pressure coefficient cannot be used for calculation of AEP, use Equations 5-12.1 and 5-12.2 of *Caltrans MTD 5-12*. Consider a total load, P_{total} as a total lateral load of AEP (P_{AEP}) in these equations. When calculating the P_{AEP} by the Limit Equilibrium (LE) analysis, use the following methods: Method 1 and Method 2.

  Note that a multiplier of 1.33 is introduced in Method 1 to account for the effect of construction method (ground anchor installation with pre-stressing). The multiplier was calibrated for cohesionless soil by comparing field measured earth pressure force with active earth pressure force. Therefore, do not use Method 1 with a multiplier of 1.33 for cohesive soil. Furthermore, do not use Method 2 for stiff to hard clay soils because the LE stability analysis using undrained shear strength is misleading for stiff clays. Stiff clays can be self-supporting, and no additional stabilizing force may be necessary to support the wall. When a drained analysis is performed on the stiff to hard clay soils with effective strength parameters, Method 2 may be used with caution.

  - Method 1 – for cohesionless soil

    Perform LE (slope stability) analysis for stability of the retained soil mass, and find the required stabilizing (active) force, P_A ($P_A = P_{FoS_1.0}$) resulting in a factor of safety (FoS) of 1.0. The FoS of 1.0 is equivalent to the active earth pressure condition. Then multiply the P_A by 1.33 to obtain the P_{AEP} ($P_{AEP} = 1.33\ P_A$). The multiplier of 1.33 was also used in deriving the AASHTO Equations 3.11.5.7.1-1 to 3.11.5.7.2b-2.

$$P_{AEP} = \frac{2K_a\gamma H^2}{3} = 1.33 \times \left(\frac{K_a\gamma H^2}{2}\right) = 1.33 \times P_A$$

Where,

P_{AEP} = Total force per unit length of a wall based on AEP distribution (Figures 3.11.5.7.1-1 (a), and AASHTO Equations 3.11.5.7.1-1 (b).

P_A = Total force per unit length of a wall based on triangular distribution of active earth pressure.

- Method 2

  Perform LE slope stability analysis and find the required stabilizing force per unit length of a wall, $P_{FoS_1.33 \text{ or } 1.54}$, resulting in a factor of safety (FoS) of 1.33 or 1.54. The calculated stabilizing force should be considered as the P_{AEP} ($P_{AEP} = P_{FoS_1.33 \text{ or } 1.54}$). Do not apply the multiplier of 1.33 used in Method 1 to P_{AEP}. According to AASHTO, a resistance factor of 0.75 (equivalent to FoS of 1.33) is recommended for well-defined soil parameters and a slope that is not supporting structures, while a resistance factor of 0.65 (equivalent to FoS of 1.54) is recommended for limited soil information and a slope that is supporting structures.

Method 2 is commonly used for slope stabilization or landslide mitigation. According to *FHWA Geotechnical Engineer Circular No. 4*, "Ground Anchors and Anchored Systems", $P_{FoS_1.3}$ is close to $1.3P_{FoS_1.0}$ for reasonably homogeneous sandy soils with competent soils below the bottom of excavation. For walls supporting critical structures, $P_{FoS_1.5}$, a higher value, may be used for P_{AEP} to reduce wall deformation and its effect on nearby structure. The P_{AEP} ($P_{FoS_1.33 \text{ or } 1.54}$) calculated from Method 2 should not be less than the P_{AEP} ($1.33\ P_A$) calculated from Method 1.

When soft or very loose soils exist below the bottom of excavation, and basal stability is of concern, extend LE analysis below the design grade. However, ground anchor ERS may not be suitable at sites with soft soils that extend to a significant depth below the design grade.

In addition to AEP, appropriate hydrostatic water pressure, lateral pressures induced by surcharge load, and seismic earth pressure should be included in the analysis.

- Surcharge Load

  Use AASHTO 3.11.6 for the lateral pressure against the wall induced by surcharge loads.

- Hydrostatic Water Pressures

  When water is present behind the wall, hydrostatic water pressure must be considered in the design in addition to other lateral earth pressures. When the hydrostatic water pressure is applied to the wall, calculate lateral earth pressure including apparent earth pressure with effective unit weight of soils. For ground anchor ERS with continuous wall facing such as sheet piles, tangent piles and secant piles, hydrostatic pressure differential can develop between the back and front of the wall face and can cause seepage or piping (boiling) in cohesionless soils. For such cases, perform

seepage analyses using the flow net or numerical analysis to ensure base stability against seepage. Section 5.2.9 of *FHWA Geotechnical Engineering Circular No. 4*, "Ground Anchors and Anchored Systems" presents procedures to calculate porewater pressure considering effects of seepage (Equations 15 to 17, and Figure 32), and simplified flow net for homogeneous soil (Figure 31). For non-homogeneous soil or special drainage conditions which may alter water boundary conditions, use numerical seepage analysis.

- Passive Earth Pressure

  For the calculation of passive earth pressure, use the log-spiral method with an appropriate wall interface friction angle. When the log-spiral method can't be used due to complex geometry or multiple soil layers, use the Trial Wedge Method with the wall interface friction angle of no greater than 0.5Φ as per AASHTO 3.11.5.4, where Φ is friction angle of the soil.

  When AASHTO Figures 3.11.5.6-1, 2, 4 and 5 are used for calculation of passive earth pressure against embedded discrete vertical elements, use the Rankine passive earth pressure coefficient. The figures were developed based on Broms Method (1964), in which the Rankine passive earth pressure coefficient was used to derive the effective width of 3 times the pile diameter. When different soil arching factors need to be used in the design, refer to AASHTO C11.8.6.3.

Seismic Lateral Earth Pressure

For calculation of seismic lateral earth pressures, use AASHTO 11.6.5.2, 11.8.6 and 11.9.6. All the seismic loads for the design of non-gravity cantilever ERS are shown on AASHTO Figure 11.8.6.2.1. As shown on Figure 11.8.6.2.1, a seismic active earth pressure resultant, P_{AE} is considered only above the design grade, while P_A and P_{PE} are applied below the design grade. In addition, according to AASHTO C11.8.6.2, for walls with continuous vertical elements, such as sheet pile walls, tangent pile walls, and secant pile walls, the P_{AE} will be distributed from top of wall to a point where the critical failure plane of LE analysis intersects the vertical element below design grade. For walls supporting seismic critical structure, seismic active earth pressure should be extended below the design grade to the toe of the vertical element to account for uncertainties in the seismic distribution below design grade. For more information on how to determine the design horizontal seismic coefficient to be used in the calculation of seismic earth pressures, refer to *Geotechnical Seismic Design of Earth Retaining Systems*.

- Seismic Active Earth Pressures

  For the calculation of seismic active earth pressures, use AASHTO 11.6.5.3. These standards present three methods to calculate the seismic active earth pressure: Mononobe-Okabe (M-O) Method, Trial Wedge method, and Limit Equilibrium (LE) Method. For wall geometry or site and soil conditions for which the M-O Method is not suitable, either Trial Wedge method or LE method can be used.

Because seismic horizontal acceleration and resulting seismic earth pressure greatly depends on the magnitude of seismic displacement of a ground anchor ERS, consult the structure designer for tolerable permanent seismic displacement of a ground anchor ERS, and assist the structure designer in estimating the design horizontal seismic coefficient (k_h). The k_h associated with the tolerable permanent seismic displacement should be used for calculating seismic active earth pressures.

When wall displacement analyses using numerical methods such as finite element or finite difference methods, and the beam-column approach are to be performed by the structure engineer, soil spring parameters such as p-y curves should be provided by the geoprofessional according to AASHTO C11.8.6.4.

- Seismic Passive Earth Pressures

  For calculation of seismic passive earth pressures, use AASHTO 11.8.6.3 and A11.4. In these articles, the log spiral procedure or the nonlinear failure surface are recommended to be used with the wall interface friction. Do not use the M-O Method for estimating the seismic passive earth pressure. According to AASHTO 11.6.5.5, a wall interface friction equal to two-thirds of the soil friction angle can be used for the calculation of the seismic passive pressures when there are no specific guidance or research results for a seismic wall interface friction. For the wall with embedment depth less than 5 feet below finished grade, use the static method for the seismic passive pressure according to AASHTO 11.6.5.5. For non-gravity cantilever walls using solider piles, use the static method according to AASHTO 11.8.6.3.

Limit State Design

The design of ground anchor ERS must meet displacement and stability requirements for each limit state below, and appropriate scour shall be considered in each limit state if the wall is in a flood prone area. For the guidance related to scour evaluation, refer to AASHTO and California Amendment 2.6.4.4.2 and 11.7.2.3.

Service Limit State

Displacement

The design of the wall must ensure that the vertical and lateral displacement does not affect the performance of the wall system. For calculation of settlement of the wall system use AASHTO Figure C11.9.3.1-1. For walls supporting low-displacement tolerance structure, numerical analysis such as finite element or finite difference methods may be required. For ground anchor ERS constructed by excavating a vertical face, there is no vertical stress increase in soils, and settlement due to net vertical stress does not occur.

Global Stability

The global stability is evaluated using acceptable methods of LE slope stability analysis such as Morgenstern-Price, Modified Bishop, Janbu, or Spencer methods. For the resistance factors used in the global stability, refer to AASHTO 11.6.2.3.

Strength Limit State

Axial capacity of vertical element (bearing failure)

For calculation of axial capacity of a vertical element, use AASHTO 10.7 through 10.9 and Caltrans Pile Foundations Manuals. Embedment depth of vertical elements must withstand factored axial load demands induced by the vertical component of total anchor loads, dead weight of the wall system and other external loads.

Basal Heave Stability

For walls embedded/supported on soft to medium clays, check basal heave stability to ensure adequate soil bearing capacity at the base of the excavation and no substantial increase in lateral earth pressure. The methods (Equations 34 to 37) to calculate a factor of safety against basal heave stability are presented in Section 5.8.2 “Basal Stability” of *Geotechnical Engineering Circular No. 4*, “Ground Anchors and Anchored Systems.” According to Section 5.8.2, significant ground movements towards the excavation will occur when the soil bearing capacity at the base of excavation is approached regardless of the strength of the supports, and it will substantially increase the load on the lowest ground anchor with decreasing FoS of basal heave stability. *Geotechnical Engineering Circular No. 4* recommends the minimum FoS of 1.5 for support of the excavation. Because there is no LFRD guideline on basal heave stability, allowable stress design (ASD) with the minimum FoS of 1.5 can be used until LRFD guidelines on basal heave stability is available in AASHTO.

Anchor Pullout

Determination of anchor pullout resistance and corresponding anchor bond length are the Contractor’s responsibility.

Minimum Anchor Unbonded Length

For determination of the minimum anchor unbonded length, refer to AASHTO 11.9.1., 11.9.4.2, C11.9.4.2, and Figure 11.9.1-1. For complex wall geometry, or subsurface profile for which Figure 11.9.1-1 cannot be used, use LE slope stability analysis for the minimum anchor unbonded length. The minimum anchor unbonded length is the distance from wall face to the failure surface plus a minimum distance between potential failure surface and frontal anchor bond zone, 5 feet or H/5, whichever is greater. According to AASHTO Figure 11.9.1-1, H is defined as design height of vertical element above design grade.

Extreme Event Limit State

Minimum Anchor Unbonded Length

For determination of the minimum anchor unbonded length, use AASHTO 11.9.6. For LE slope stability analysis, use the k_h associated with a tolerable seismic permanent displacement of ground anchor ERS.

Seismic Global Stability

For seismic global stability analysis, use AASHTO 11.8.6.1 and 11.9.3.2. There are two benchmark values for k_h used in the seismic global stability analysis depending on tolerable seismic displacements: 1/2 or 1/3 horizontal peak ground acceleration (HPGA), which is the acceleration at zero period (T =0) calculated from Caltrans ARS Online (v.3.0). Use k_h of 1/3 HPGA if a mean seismic permanent displacement of about 4.0 inches is acceptable. Use k_h of 1/2 HPGA if the maximum 2.0-inch seismic displacement is acceptable. For details of the seismic global stability analysis procedure, refer to *Geotechnical Seismic Design of Earth Retaining Systems*.

Ground Anchors for Slope Stabilization

Ground anchors can be paired with either a structural reaction frame or an ERS, i.e. soldier piles with lagging, to stabilize a slope. Three types of structural reaction frames are commonly used in Caltrans projects: continuous reinforced concrete beam (known as waler), discrete reinforced concrete blocks, and a hybrid system. The selection of ground anchor reaction frame types or ERS depends on project-specific requirements and constraints, including right of way, construction easement, accessibility and the condition of the slope or highway foundation to be stabilized. Compared to ground anchor ERS, ground anchors with structural reaction frames are more suitable and cost-effective when extensive re-grading or excavation of the slope is not needed or can be reduced to the minimum as the structural reaction frames are placed on and against the slope surface. On the other hand, ground anchor ERS for slope stabilization requires excavation of the slope in front of the wall to install the ground anchors.

Design Considerations

- Ground Anchor with Structural Reaction Frame

  For the design of ground anchor with structural reaction frames, include only the ground anchor force in the slope stability analysis model. The following are the geotechnical design steps for ground anchor with structure reaction frames:

  1. Determine and enter ground anchor design parameters such as inclination angles, locations (vertical spacings for multiple rows of anchors), and a stabilizing anchor force (kips/ft) (a point load on slope surface) at each anchor location into a slope model and perform slope stability analysis to arrive at the optimal ground anchor design that satisfies the minimum factor of safety of the slope.
  2. Determine a minimum size of the reaction frame (width and/or length) based on the determined anchor force in step 1 and bearing capacity of a reaction frame at each anchor location. For granular soils where bearing capacity varies with footing dimensions, iterative process is necessary to arrive at the minimum size.
  3. Provide the following ground anchor design parameters and a minimum size of the reaction frame for the design of structural reaction frame:
     a) Anchor force (kips/ft) at each anchor location

b) Anchor inclination angle with respect to the horizonal line
c) Unbonded anchor length
d) Minimum reaction frame width and/or length

- Ground Anchor ERS – Soldier piles with Lagging

  For the design of ground anchor ERS for slope stabilization, follow ground anchor ERS design described in Design section. Use Method 2 for the determination of the magnitude and distribution of apparent earth pressure.

For the required minimum factor of safety of a stabilized slope, refer to *Landslides and Soil Cut Slopes*. When bridge abutment or any critical structure with low tolerance of failure are involved in slope stabilization, use a minimum factor of safety of 1.5. Refer to *FHWA IF-99-015 Section 5.7 Anchored Slopes and Landslide Stabilization Systems* for ground anchor design concepts and methods using limit equilibrium method (slope stability computer program).

When creep and associated loss of pre-stressed anchor load is a concern in the performance of ground anchors, consider increasing the number of ground anchors to lower the load demand per anchor while assuring a minimum center to center spacing of three times drilled hole diameter. For soft to very soft cohesive soils or materials with high organic contents that are highly susceptible to creep, ground anchor system should not be used.

Ground Anchor Construction Quality Assurance

Quality assurance testing of ground anchors is an important part of a project. Every ground anchor is to be tested to verify the adequacy of the contractor's drilling and grouting operations, load transfer mechanism, and its load-carrying capacity with specific acceptance criteria on creep deformation. Types and purpose of ground anchor tests are summarized in following table:

Table 1: Type and purpose of Ground Anchor Construction Quality Assurance Tests

Type	Purpose	Sacrificial or Production Anchor	Test Load (often called as Factored Test Load)	Lock-Off Load
Verification Test (Preproduction Pullout Test)- five load cycle	Verify grout/ground bond strength as well as creep and load-deformation behavior	Sacrificial Anchor	Factored design load (FDL) divided by resistance factor or seismic load whichever greater	N/A
Performance Test – five load cycle	Verify anchor capacity to resist test load as well as creep and load-deformation behavior.	Production Anchor	Factored designed load (FDL) or seismic load whichever greater	0.55 FDL to 0.75 FDL
Proof Test – one load cycle	Verify anchor capacity to resist test load as well as creep behavior	Production Anchor	Factored designed load (FDL) or seismic load whichever greater.	0.55 FDL to 0.75 FDL

Performance and Proof Tests

For details of loading schedules and acceptance criteria, refer to Caltrans Standard Specifications 46-2-01D(b) and 46-2.01D(3).

For performance testing, recommend a minimum of 3 anchors per wall zone but at least 5 percent of total production anchors. All remaining production ground anchors are subjected to proof tests.

After successful ground anchor tests, the anchor is stressed to a specified load (lock-off load) and lock off against the structure element. A lock-off load of 0.55 FDL is recommended for most ground anchor walls in Caltrans. However, when ground anchors support bridge abutments or any critical structures with a low tolerance of failure or displacement, recommend a lock-off load of 0.67 FDL. For the ground anchors to stabilize slopes, a lock-off load may be equal to or greater than 0.75 FDL. However, when recommending a higher lock-off load, associated creep displacement and loss of pre-stressed load in the anchors should be expected and considered for the performance of the ground anchor system.

Verification Tests (Preproduction Pullout Tests)

Caltrans practice in ground anchor installation allows single-stage grouting, by which both anchor bond length and unbonded length are grouted in one stage. Single-stage grouting will cause load-transfer of unknown magnitude through the grout column (unbonded length) above the anchor bond length. The load transfer can lead to erroneous interpretation of ground anchor capacities. When unknown load-transfer to the unbonded zone and/or verification of nominal ultimate bond strength and adequacy of design bond length determined by the Contractor are of concern for a specific project, verification tests can be recommended. Refer to *AASHTO LRFD BDS 11.9.8.1 and C11.9.8.1* where verification testing is called the preproduction pullout test. Sacrificial verification test ground anchors are installed with only the bond length grouted for testing.

Since the verification test is not a standard test for Caltrans projects, Non-Standard Special Provisions (NSSP) shall be developed when needed. Before recommending verification tests, consult with Structure Design and Structure Construction about any structural or construction issues or constraints that should be considered for the efficiency and feasibility of tests. For example, when an unusually large structural reaction frame is needed due to a high-test load, one option is to test sacrificial ground anchors to the maximum performance test load by allowing the bond length to be reduced. This approach is only suitable when material properties in the bond zone are uniform.

The Ground Anchor Verification Test nSSP (section 46-2) is available on the NSSP for Geotechnical Design intranet page. Use this nSSP and modify it as necessary for any project-specific requirements. The factored test load (FTL) of the verification test shall be taken as the factored design load divided by a resistance factor or seismic, whichever greater. For the resistance factor for presumptive ground anchor pullout resistance, refer to *California Amendments to AASHTO LRFD BDS* Table 11.5.7.1.

Deadman

Design guidelines for deadman anchors are not presented in AASHTO. If the deadman anchors are needed, use *Caltrans (LFD Version) Bridge Design Specifications (BDS)* Article 5.8.6.2 (2003). In *Caltrans BDS*, a deadman is termed a structural anchor. The deadman typically consists of a concrete anchor with large masses of precast or cast-in-placed concrete (*Caltrans BDS* Article 5.8.6.2.1), an anchor pile with driven or cast-in-drilled-hole (CIDH) piles (*Caltrans BDS* Article 5.8.6.2.2) and pile anchors with driven piles (*Caltrans BDS* Article 5.8.6.2.3). According to *Caltrans BDS* Article 5.8.6.2.1, the depth of the concrete anchor deadman shall be based on the active and passive earth pressures behind and in front of the deadman, and the deadman must be located far enough from the wall face to ensure the passive zone in front of the deadman does not overlap with active zone behind the wall.

For the design of the deadman, allowable stress design (ASD) may need to be used until resistance factors for the design of the deadman are calibrated and published in

AASHTO. Before recommending and designing the deadman anchor, verify with the structure designer whether ASD or LRFD will be used.

Unlike ground anchor ERS, in which AEP is used in the design, *Caltrans BDS* Article 5.8.2 recommends active earth pressure with triangular distribution for design of the deadman anchors. The recommendation of the active earth pressure with triangular distribution may indicate that the deadman is a passive anchorage with no pre-stressing, and the anchor is activated when the retained mass is deformed.

Reporting

Prepare the report per *Foundation Reports for Earth Retaining Systems (ERS).*

Design Parameters

Provide the magnitude and distribution of lateral earth pressure along vertical wall elements or provide the engineering properties required for the calculation of lateral earth pressure. For walls with simple geometry and homogeneous soils, provide the following engineering properties for foundation soils and rocks below design grade and retained soils and rocks behind the wall:

- Moist unit weight
- Cohesion
- Friction angle
- Rock strength
- Static earth pressure coefficients
- Horizontal seismic coefficient with expected displacement
- Seismic earth pressure coefficients
- Design groundwater elevation

Based on the design parameters provided above, the distribution and magnitude of lateral earth pressure is calculated by the structure engineer for structural analyses.

For walls with complex geometry, inhomogeneous soils, or cohesive soil layers, or when requested by Structure Design, provide the following information in addition to the above information:

- Static AEP diagram, or total stabilizing force (unfactored), P_{AEP} for retained soils
- Static earth pressure diagram, and static earth pressure coefficient for embedded soils and rocks
- Seismic earth pressure diagram and seismic earth pressure coefficient
- Steady-state water pressure distribution
- Soil spring parameters such as p-y curves (AASHTO 11.8.6.4) if soil structure interaction analysis is performed by the structure designer

Recommendations

- Minimum pile embedment below design grade for required geotechnical axial capacity, global stability, and basal heave stability
- Length of anchor unbonded zone calculated under service limit state or extreme event limit state, whichever is longer.
- Minimum lagging or wall face embedment below finish grade for erosion, local stability and global stability
- Drainage system details and specifications, if horizontal drains are recommended to intercept the flow at a distance well behind the wall

Notes for Specifications

Follow the instructions provided in *Geotechnical Notes for Specifications* for the information and recommendations to be provided in this section.

Send the Ground Anchor Verification Test nSSP (section 46-2), available on the NSSP for Geotechnical Design intranet page, to the Specification Engineer so it can be included in the contract Special Provisions of the project.

Geotechnical Seismic Design of Earth Retaining Systems

This module presents the Caltrans practice for the geotechnical seismic design and analyses of ordinary ERS built within the State Right Way and provides guidance on how to determine the horizonal seismic acceleration coefficient (k_h) for seismic analysis and design. The geotechnical seismic analyses of ERS include brief discussions on seismic bearing capacity, geotechnical seismic capacity of piles, seismic global stability, simplified seismic permanent displacement, liquefaction potential, lateral spreading and surface fault rupture.

Project-specific seismic analysis including numerical analysis should be performed for the following ERS:

- Failure of the ERS would cause a substantial economic impact
- The ERS is designated by the sponsoring district or local agency as critical, in consultation with Caltrans Division of Engineering Services
- ERS site has geotechnical complexities such as excessive amount of soft clay, surface rupture, liquefaction, lateral spreading, and other geotechnical instabilities
- ERS is a tunnel portal wall

Design Manuals and Guidelines

For seismic design of ERS, use this module, and the following design manuals and policy:

- *2017 (8th Edition) AASHTO LRFD Bridge Design Specifications* and *2019 California Amendments*
- ERS related Geotechnical Modules in the *Geotechnical Manual*
- *2020 Structure Technical Policy 11.29,* Seismic Design Criteria for Earth Retaining Systems

For design cases that are not covered in the above documents, refer to other FHWA reference manuals including FHWA (2005) and NCHRP Report 611 (Anderson et al., 2008).

Geotechnical Services' Responsibilities for Seismic Design of ERS

For seismic design of ERS, Geotechnical Services' responsibilities are:

- Provide the design horizontal acceleration coefficient (k_h). When applicable, assist the structure designers in estimating a horizontal yield acceleration coefficient (k_y) for sliding ERS. The k_y is the horizontal seismic acceleration

coefficient that causes imminent sliding or global slope failure (a factor of safety of 1.0) of the ERS

- Provide the engineering properties of soil, such as unit weight, cohesion, friction angle and seismic active earth pressure coefficient (k_{ae}) when the Mononobe-Okabe method (M-O) is applicable
- Provide the magnitude and distribution of seismic lateral earth pressure for complex wall geometries to the structure designer where the M-O is not applicable or when requested by structure designers
- Check liquefaction, liquefaction induced settlement, and lateral spreading when applicable
- Perform seismic global stability analysis
- Analyze the seismic bearing capacity for ERS supported on shallow foundations
- Analyze the geotechnical seismic capacity of piles and provide design pile tip elevations for ERS supported on deep foundations

Classifications of ERS for Geotechnical Seismic Design

The seismic design of ERS is based on performance during and after the design seismic event.

For the seismic design, an ERS is classified as either sliding ERS or non-sliding ERS. The ERS that will slide along the base during the design seismic event are sliding ERS and include conventional gravity or semi-gravity walls on spread footings. Under current design method and practice per AASHTO 11.10.7, soil nail walls and MSE walls can be included in the sliding ERS category. The external stability calculation of these ERS are based on a composite mass consisting of reinforcements, reinforced soils, and the facing elements. For the sliding ERS, simplified methods presented in AASHTO A11.5 can be used to calculate seismic displacement analysis. When deformation and/or rotation are of concern for project-specific sliding ERS, numerical analysis should be considered.

Non-sliding ERS are retaining walls that move during the design seismic event due to the deformation and/or rotation of walls (other than sliding at the base). For the calculation of the seismic movement/deformation of ERS, numerical analyses may be needed and include beam-column analysis using p-y modeling and numerical deformation analysis (refer to AASHTO 11.8.6.2 and AASHTO 11.9.6). The simplified methods based on Newmark sliding analysis are not applicable because they cannot account for the deformation and/or rotation of ERS.

Determining Horizontal Seismic Acceleration Coefficients and Seismic Displacement

The displacement of an ERS will reduce the dynamic forces and soil pressure on the ERS during a seismic event. The more an ERS moves, the less seismic lateral earth pressures on the ERS.

In the seismic design of ERS, there is the concept of the “seismic fuse” also known as horizontal yield seismic coefficient. This caps the dynamic force experienced by the ERS at the level that causes the controlling mode of failure to occur. The ERS experience the controlling mode of failure when the seismic fuse of an ERS is triggered before dynamic force is high enough to cause any other response. The horizontal seismic forces experienced by the ERS will not be greater than the horizontal seismic force that caused controlling mode of failure.

For the ERS classified as sliding ERS, the seismic fuse is either k_{y_global}, (a horizontal seismic acceleration coefficient causing sliding of potential global slope failure mass), or $k_{y_sliding}$ (a horizontal seismic acceleration coefficient causing sliding along the base of ERS), whichever is lower (see Figure 1). The seismic displacement occurs either via sliding along the base of ERS or sliding of global slope failure mass.

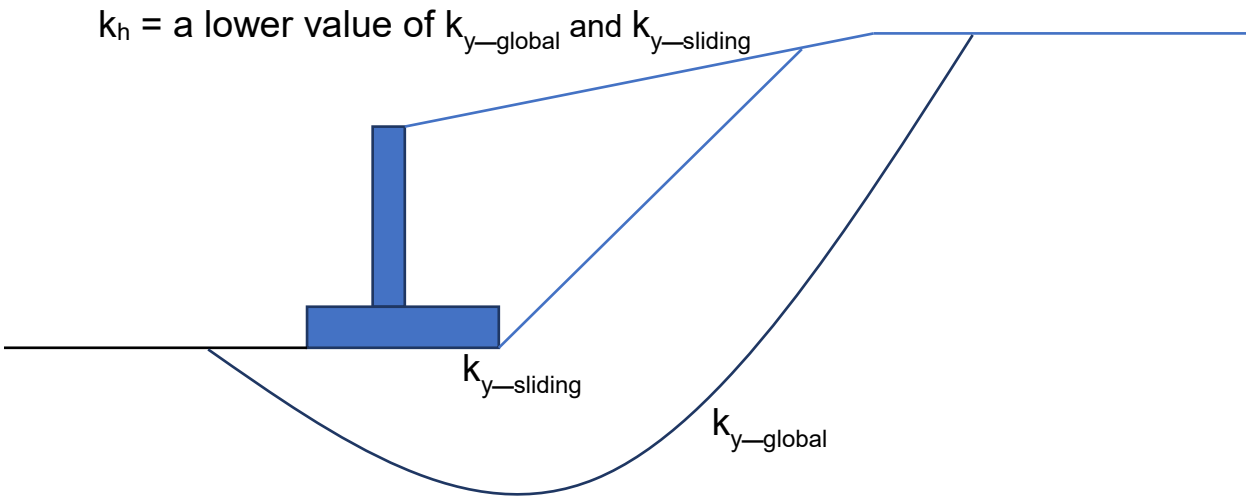


Figure 1. Seismic fuses (k_y) for a semi-gravity ERS on spread footing

Benchmark k_h Values for Seismic Design of ERS

According to AASHTO 11.6.5 the horizonal seismic acceleration coefficient, k_h is 1/2 horizontal peak ground acceleration (HPGA) for an expected displacement of 1.0 to 2.0 inches. The HPGA is calculated using ARS Online (v3.0) for a zero period (T=0.0 sec.).

Another benchmark value that can be used for sliding ERS is k_h equal to 1/3 HPGA for more seismic displacement than 2 inches. The sliding ERS designed for this k_h are expected to produce a permanent expected mean displacement of about 5 inches in the design seismic event.

When benchmark k_h value are used for the design of sliding ERS, seismic displacement analysis is not required. However, when a calculated sliding FOS is greater than 1.1, the ERS may not experience the magnitude of displacement associated with benchmark horizontal acceleration coefficient. In such cases, horizontal yield acceleration coefficient ($k_{y_sliding}$) can be greater than the benchmark values and the increase in horizontal acceleration coefficient may need to be considered in the design.

Steps to Determine Horizontal Seismic Acceleration Coefficient, k_h.

To determine a horizontal acceleration coefficient for the seismic design of ERS, first define a retaining wall as either sliding ERS or non-sliding ERS, then follow the steps listed below:

Sliding ERS

Ordinary ERS classified as sliding ERS can move and/or tolerate a mean seismic displacement greater than about 5 inches in the design seismic event. For the seismic design of these ERS, use the following steps:

1. Calculate k_{y_global} of the ERS to determine whether seismic global stability controls k_h
2. Determine the controlling value of k_h as the lower value of 1/3 HPGA and k_{y_global}; when k_h is controlled by k_{y_global}, calculate the seismic displacement using simplified methods presented in AASHTO A11
3. Verify with the structure engineer that the ERS analyzed using the controlling k_h have the seismic sliding FOS between 1.0 and 1.1

When sliding FOS is less than 1.0, obtain $k_{y_sliding}$ from the structure engineer and perform seismic displacement analysis using simplified method to verify that the ERS can tolerate calculated seismic displacement.

When sliding FOS is greater than 1.1, obtain ky_sliding from the structure engineer and determine the controlling value of kh for the design as the lower value of ky_sliding and ky_global.

Non-sliding ERS

For the seismic design and analysis, recommend k_h = 1/2 HPGA and verify with the structure engineers that the ERS tolerates the seismic movement of 2 inches. When the ERS can tolerate seismic movement greater than 2 inches, greater reduction in the horizonal acceleration coefficient than 50% may be used in the design.

According to AASHTO 11.8.6.2 and AASHTO 11.9.6, numerical analyses may need to be performed for non-sliding ERS to verify acceptable wall movement. The numerical analyses include beam-column analysis using p-y modeling and numerical deformation analysis.

Standard Plans ERS

Standard Plan ERS (semi-gravity cantilever retaining walls) were designed based on the horizontal seismic acceleration coefficient, k_h of 0.2 that is corresponding to a HPGA of 0.6 g in Caltrans seismic design practice. Therefore, Standard Plan ERS historically have only been used in areas with HPGA of 0.6g or less.

Standard Plan ERS have been commonly used to support slopes, highway embankment, and other flexible components that are not displacement-sensitive and can tolerate a lateral displacement that may be considered as excessive such as 12 inches or even greater. Case histories have proven that Caltrans Standard Plan ERS perform well without collapse during seismic events. Therefore, Standard Plan ERS may be considered in areas with a HPGA greater than 0.6 g if resulting permanent displacement are within tolerance for the project.

When the design requirements of a Standard Plan ERS have been satisfied except for the seismic design requirement, proceed with following steps before opting for Special Design ERS.

1. Use 0.2 as the yield horizonal seismic coefficient, ky for Standard Plan ERS, or obtain the ky from the ERS specialist responsible for the applicable Standard Plan ERS
2. Calculate permanent seismic displacement using the yield horizontal seismic coefficient, ky
3. Provide the calculated permanent seismic displacement to the District Project Engineer to determine whether calculated permanent displacement is acceptable or not
4. If the value is not acceptable, proceed with Special Design ERS

If sufficient subsurface data is available, perform this evaluation in project planning (0) phase and include the calculated permanent seismic displacement and recommendation in District Preliminary Geotechnical Report (DPGR) to assist District in determining whether the Special Design ERS is needed or not.

Permanent Seismic Displacement for Sliding ERS

When the calculation of permanent seismic displacements is needed, use AASHTO A11.5 which presents three different methods to estimate the seismic displacement using horizonal seismic acceleration coefficient and vice versa. Because Caltrans uses ARS Online webtool for seismic ground motions and response spectra, some parameters used in the methods presented in AASHTO A11.5 need to be modified:

A11.5.1 – Kavazanjian et al. (1997)

For Equation A11.5.1-1, use k_y for k_h and HPGA for As.

A11.5.2 –Anderson et al. (2008)

For Equations A11.5.2-1 and A11.5.2-3, use HPGA/g for k_{h0} with a wave scattering factor (a) of 1.0. The wave scattering effect has not been used in Caltrans GS practice and may not be significant for most of Caltrans ERS. If there is a need to consider the wave scattering effect on k_h, use Equation A11.5.2-2 with a site class adjustment factor (F_v) of 1.0. The site class adjustment factor shall be set as 1.0 for all applicable equations as the ARS Online webtool generates an ARS curve that reflects site conditions using a time-averaged shear wave velocity for the upper 30 meters of the soil profile, V_{s30}. For details on how to use the ARS Online webtool, refer to the *Design Acceleration Response Spectrum Module*.

A11.5.3 –Bray et al. (2010) and Bray and Travasarou (2009)

For Ts < 0.05 second – treat the potential sliding mass as a block and use Ts = 0:

$$d\ (cm) = exp\big(-0.22 - 2.83\ln(k_y) - 0.33(\ln(k_y))^2 + 0.566\ln(k_y)\ln(HPGA) + 3.04\ln(HPGA) - 0.244(\ln(HPGA))^2 + 0.278(M-7)\big)$$

For Ts ≥ 0.05 second:

$$d\ (cm) = exp\big(-1.10 - 2.83\ln(k_y) - 0.33(\ln(k_y))^2 + 0.566\ln(k_y)\ln(S_a) + 3.04\ln(S_a) - 0.244(\ln(S_a))^2 + 1.5T_s + 0.278(M-7)\big)$$

Where:

T_s: Fundamental period of the wall

M: Moment magnitude of the design earthquake

S_a: Spectral acceleration (5% damping) at a degraded period of $1.5T_s$

To determine M and Sa, use ARS Online (v3.0). To calculate Ts, use the following equation:

$T_s = 4H'/V_s$

Where:

H': 80 percent of the height of the wall measured from the bottom of the wall

V_s: Shear wave velocity of the soils behind the wall

Note that AASHTO Equation A11.5.3-1 is basically the same as the equation for Ts ≥ 0.05 second above but expressed in terms of k_y that is replaced by k_h. AASHTO Equation A11.5.3-1 is typically used to calculate a horizontal seismic acceleration coefficient given the acceptable permanent seismic displacement of the wall.

GEOTECHNICAL SEISMIC ANALYSIS AND DESIGN

For the seismic design of ERS and the magnitude and distribution of seismic lateral earth pressure, use the k_h determined as above.

Seismic Lateral Earth Pressure

For the magnitude and distribution of seismic lateral earth pressure, refer to AASHTO 11.6, A11.3, and A11.4, and ERS related GS modules.

Bearing Resistance

For the bearing resistance of ERS, refer to AASHTO 11.5.8 and 11.6.3, and ERS related GS modules. When calculating the bearing resistance, use the effective footing width. When there is a descending slope on or near an ERS, adjust the bearing capacity equation to account for sloping ground conditions (AASHTO 10.6.3.1.2c).

Geotechnical Seismic Capacity of Piles

For the geotechnical seismic capacity of piles, refer to “Driven Pile Foundations” and “CIDH Pile Foundations” in the *Geotechnical Manual*.

Seismic Global Stability

Perform iterative global seismic stability analyses and find the horizontal yield acceleration coefficient (k_{y_global}) that results in a FOS of 1.0 for global seismic stability analysis.

For sliding ERS, seismic global stability is considered acceptable when the k_{y_global} is equal to or greater than 1/3 HPGA. When the k_{y_global} is less than 1/3 HPGA, calculate the seismic displacement of potential slope failure mass to verify that calculated displacement is acceptable.

For non-sliding ERS, calculate the seismic displacement of potential slope failure mass associated with the k_{y_global} to verify that calculated displacement is acceptable.

Liquefaction Potential

When foundation soils to support ERS are identified as liquefiable, the effects of liquefaction on ERS must be considered in design. The effects include liquefaction induced settlements, reduced bearing resistance due to reduced soil shear strength, and lateral spreading. The liquefaction potential should be evaluated during the early stages of a project, and its effects or mitigation measures should be discussed in the Type Selection Meeting.

Refer to *Liquefaction Evaluation* in the Geotechnical Manual.

Lateral Spreading

When foundation soils are subject to liquefaction, reduction in shear strength of liquefiable soils should be considered in seismic global stability, and the associated increase in permanent displacement should be evaluated. The effects and mitigation measures of lateral spreading should be discussed in the Type Selection Meeting.

Refer to *Liquefaction-Induced Lateral Spreading* in the Geotechnical Manual.

Fault Rupture

When an ERS crosses active or potentially active faults, surface fault rupture and the associated displacement hazard needs to be evaluated. The effect of surface fault rupture and mitigation measures if needed should be discussed in the Type Selection Meeting.

Refer to *Fault Rupture* in the Geotechnical Manual.

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5. FHWA (2005). "*LRFD for Highway Bridge Substructures and Earth Retaining Structures: Reference Manual*." FHWA-NHI-05-094, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

6. Kavazanjian, E., N. Matasovic, T. Hadj-Hamou, and P. J. Sabatini. 1997. "*Design Guidance: Geotechnical Earthquake Engineering for Highways*," Geotechnical Engineering Circular No. 3, Vol 1-Design Principles, FHWA-SA-97-076. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C.

Shallow Foundations for Bridges

This module presents the design methods, examples (Appendices 1 and 2), and communication steps between Structure Design (SD) and Geotechnical Services (GS) for the load and resistance factor design (LRFD) of shallow footing foundations for bridges. It may aid in the design of shallow foundations for retaining walls, non-standard walls, and other structures, however many design aspects for those other structures are not adequately addressed.

Upon receipt of a foundation design request from SD, the Geoprofessional must review the request and verify that the information is sufficient to allow the design process to begin.

Design of a spread footing is an iterative process between the Structure Designer and the Geoprofessional. Some of the communication and data exchanges are specific to Caltrans and may not be applicable to consultant work. The process presented in this module begins after adequate subsurface site data has been obtained and a shallow foundation has been type selected.

The reference standards for shallow foundation investigations, design, and reporting are:

- Caltrans Seismic Design Criteria (SDC)
- AASHTO LRFD Bridge Design Specifications with CA Amendments (AASHTO)
- Bridge Memos to Designers (MTD) 4-1, *Spread Footings*
- Caltrans Geotechnical Manual
    - *Foundation Reports for Bridges*
    - *Geotechnical Investigations*

Geotechnical Services provides the Structure Designer with a foundation report addressing:

- Permissible Net Contact Stress (q_{pn}): The vertical footing stress, that produces a permissible settlement under the Service Limit State (resistance factor, φ=1).
- Gross Nominal Bearing Resistance (q_n): The soil's ability to resist a uniform bearing stress (or a maximum bearing stress for rock) that will cause a bearing capacity failure.
- Factored Gross Nominal Bearing Resistance (q_R): This is determined by multiplying the Gross Nominal Bearing Resistance by a resistance factor, $q_R = \varphi_b \cdot q_n$. The q_R uses a resistance factor, φ, which varies for the Strength and/or Construction Limit State, or uses resistance factor, φ=1, for the Extreme Event Limit State.
- The recommended internal friction angle (drained), or the undrained shear strength at the bottom of footing elevation for cohesionless or cohesive soil respectively. SD inputs these values into the CT Abut/WinFoot computer programs for sliding analysis.

- The bearing stress distribution between the footing and the soil/rock. In soil, the bearing stress is based on a uniform stress distribution applied to the effective footing area. In rock, the bearing stress is based on a triangular or trapezoidal stress distribution, as appropriate, on the footing area.
- Global stability analysis for Service-I Limit State and Extreme Event Limit State for a footing on or near sloping ground.
- The stiffness matrix for dynamic response analyses (per GEC Circular No. 3), if requested.
- That the footing size selected by the Structure Designer is adequate.

The Structure Designer will:

- Provide Foundation Footing Design and Load Information to GS (per MTD 4-1)
- Check Eccentricity
- Check Overturning Stability
- Check Sliding Stability

For further information on the Structure Designer's role, refer to Bridge Design Practice (BDP), Chapter 15.

Investigations

The geotechnical investigation for a shallow foundation should identify the properties and behaviors of soil and/or rock, the groundwater conditions, and other subsurface conditions that might affect the foundation design and performance.

The Geoprofessional must first review existing information related to site geology, strength of soil and rock, groundwater, and geologic hazards. Refer to *Geotechnical Investigations* for direction on performing a literature (data) search.

If the literature search does not provide all required information, the Geoprofessional must develop an exploration plan based on site constraints, geologic variability, and available resources. Locate borings as close as possible to the foundations.

The exploration plan should include:

- An appropriate number of exploratory borings and/or cone penetration tests (CPT) to develop the design soil profile (AASHTO Table 10.4.2-1).
- An appropriate depth for the borings or CPT. The depth of the explorations should generally extend below the foundation to:
    - 4 times the estimated footing width below the proposed bottom of footing
    - a depth where material strength and strain characteristics are acceptable (e.g. very dense or hard soil).
    - to the full depth of soft, loose, and/or weak soils upon which stability, bearing resistance, and settlement is dependent.

- Standard penetration tests (SPT) with sampling intervals no greater than 5 feet. Closer intervals of SPT testing should be considered within a depth of 2 times the footing width below the bottom of footing.
- Groundwater measurements.
- Soil and water samples for corrosion testing in accordance with *Caltrans Corrosion Guidelines*.
- Adequate samples for laboratory testing such as particle analysis tests, Atterberg limit tests, consolidation tests, direct shear tests, and/or triaxial tests, for soil below the bottom of footing within a depth of 4 times the estimated footing width.
    - For cohesionless materials, any remolded samples that will be tested in the laboratory should have characteristics similar to the field conditions.
    - For cohesive soils, consolidation testing may be necessary where settlement magnitude and rate are significant project considerations.

Design

The following design methodologies are used for calculating settlement (Service-I Limit State).

- Use a resistance factor (φ) of 1.0.
- Calculate the settlement of cohesionless soils using the Hough method and SPT data (AASHTO 10.6.2.4.2).
- Calculate the settlement of cohesive soils using AASHTO (10.6.2.4.3).
- Calculate settlement on rock using AASHTO (10.6.2.4.4).
- For foundations on sloping ground, make appropriate reductions in overburden stresses (see design example in the appendix).

In cases where unacceptable settlements are predicted, or low bearing resistance results from the presence of near-surface loose, soft, or non-uniform materials, consider removing the inadequate material and replacing it with structure backfill or lean concrete.

Differential settlement between supports must be evaluated and discussed with the SD. Typically, multi-span structures, and single span structures with end-diaphragm abutments can tolerate no more than ½ inch of differential settlement between adjacent supports. Typically, single span structures with seat abutments can tolerate no more than 2 inches of differential settlement. For multi-column bent supports differential settlement should be evaluated and discussed with the SD when subsurface conditions show variable soil/rock compressibility across a bent support. Differential settlement should be evaluated for soil foundations by calculating the settlement at each support location using the applied net uniform bearing stress.

The following design methodologies are used for calculating bearing resistance (Strength and Extreme Event Limit States) in accordance with AASHTO.

- For bearing resistance calculations use a resistance factor of 0.45 to 0.55 for the strength limit state, and 1.0 for the extreme event limit state.
- Calculate the bearing resistance of soil using the bearing capacity equation (AASHTO 10.6.3.1.2). Use the appropriate bearing capacity equation when there are sloping ground conditions (AASHTO 10.6.3.1.2c). These methods may also be used for design in weak rock that behaves like a very dense or hard soil.
- Calculate bearing resistance on rock using AASHTO 10.6.3.2.

The soil properties used in design should come from: (1) SPT correlations (see *Soil Correlations Module*), and/or (2) results from laboratory tests.

Shallow foundations founded on rock must be designed using procedures developed specifically for the characteristics of rock masses. Foundation engineering on rock differs from foundation engineering on soil in several respects:

- Applied stress distribution pattern is trapezoidal or triangular.
- Nominal bearing resistance and settlement are calculated using the physical footing width, B.
- Rock foundation conditions must be evaluated for both intact rock properties (unconfined compressive strength) and rock mass properties (discontinuity spacing(s), orientation(s), aperture(s), and condition(s)).
- Engineering calculation procedures for rock foundations are tailored to the rock foundation conditions.
- Acceptance of the foundation configuration is performed by comparing the gross maximum bearing stress to the gross nominal bearing resistance.

The design must also account for geologic hazards such as:

- Earthquakes: Guidance for use of shallow foundations at abutments related to structure type and Peak Ground Acceleration (PGA) is in MTD 5-1, Table 1.
- Liquefaction (see *Liquefaction Evaluation* module)
- Lateral spreading (see *Lateral Spreading* module)
- Scour: Shallow foundations are permissible in a watercourse if the top of footing is below the total scour elevation.

Bridge widenings pose a challenge because it is likely that the existing structure was designed using different methodologies. Caltrans designs the foundation for the widening as if it were a separate structure, regardless of the reality that the superstructure and substructure will be connected to the existing structure. Ask SD for the allowable differential settlement between the existing and new foundations.

Design Information and Communication

Shallow foundation design requires an iterative process that begins when Structure Design sends a request to Geotechnical Services including:

- General Plan
- Foundation Plan
- Foundation Data Table (MTD 4-1, Attachment 1, Table 1)
- Scour Data Table (MTD 4-1, Attachment 1, Table 2)

Foundation Data
(MTD 4-1, Attachment 1, Table 1)

Support No.	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Estimated Footing Dimensions (feet)		Permissible Settlement Under Service-I Load (inches)	Approximate Ratio of Permanent/Total Service-I Load*
			B	L		
Abut 1	214.0	206.0	16	72	1	0.87
Bent 2	199.0	192.0	18	18	1	0.85
Abut 3	214.0	206.0	16	72	1	0.87

*For calculating consolidation settlement of spread footing founded on cohesive soils.

Design Process

Step 1: Initial Evaluation of Shallow Foundation

- Verify that the proposed bottom of footing elevation is appropriate. If, for example, the requested bottom of footing elevation is in a layer of unsuitable material (e.g., loose sand), inform the Structure Designer and provide a revised bottom of footing elevation or discuss options for improving the bearing layer by removing unsuitable soils and replacing with engineered fill or lean concrete.
- Verify that the shallow foundation is acceptable considering known geological hazards (e.g., faulting, scour).

Step 2: Calculate the Permissible Net Contact Stress (Service Limit State)

Calculate the Permissible Net Contact Stress (q_{pn}) that produces the permissible settlement (usually 1” or 2”) for the footing dimensions provided by the Structure Designer in the Foundation Data table, and footing dimensions with varying length to width (L/B) ratios as shown in MTD 4-1, Attachment 2. This is an indirect calculation that produces a permissible settlement for an initial guess at the net contact stress. Further guesses are used until the chosen net contact stress produces the required permissible settlement.

The footing dimensions provided in these tables are labeled as "Effective" with an assumed eccentricity of zero (i.e., L=L' and B=B') because the support loads are unknown at this stage of the design process. The Structure Designer will use the data as input for the software that calculates the structural loads and the effective footing sizes. The Permissible Net Contact Stress results are presented differently for abutment and bent supports:

- For abutment design determine the Permissible Net Contact Stress (q_{pn}) for one effective footing length (fixed by the bridge dimensions) and at least five effective footing widths. Determine the range of effective footing widths and physical constraints with the Structure Designer. Present the agreed upon footing size range results in the End Supports (Abutments) table (after MTD 4-1 Attachment 2, Table 1).
- For bent design determine the permissible net contact stress (q_{pn}) for at least five effective footing widths and five L'/B' ratios. Determine the desired range of L'/B' ratios and physical constraints (e.g. maximum widths) with the Structure Designer. Present the effective footing widths and L'/B' ratio range results in the Intermediate Supports (Bents and Piers) table (after MTD 4-1 Attachment 2, Table 2).

Step 3: Calculate the Gross Nominal Bearing Resistance

Calculate the Gross Nominal Bearing Resistance (q_n) for the effective footing sizes in the End Supports (Abutments) table and the Intermediate Supports (Bents and Piers) table. As discussed in step 2, the effective footing widths have an assumed eccentricity of zero at this stage of the design process (i.e., L=L' and B=B'). Enter the results in the appropriate columns of each table.

Step 4: Calculate the Factored Gross Nominal Bearing Resistance (Strength Limit State)

Calculate the Factored Gross Nominal Bearing Resistance (q_R) for the effective footing sizes in the End Supports (Abutments) table and the Intermediate Supports (Bents and Piers) table. Enter the results in the appropriate columns of each table.

Step 5: Send Preliminary Design Data to Structure Design

Email the completed tables to the Structure Designer:

- End Supports (Abutments) table (MTD 4-1 Attachment 2, Table 1)
- Intermediate Supports (Bents and Piers) table (MTD 4-1 Attachment 2, Table 2)

See Appendix 1 and 2 for examples of these tables.

Step 6: Request Design Data from Structure Design

Request that SD provide the following tables with the appropriate load information:

- Foundation Data (MTD 4-1 Attachment 4, Table 1)
- Scour Data (MTD 4-1 Attachment 4, Table 2), if applicable

- LRFD Service-I Limit State Loads for Controlling Load Combination (MTD 4-1 Attachment 4, Table 3)
- LRFD Strength/Construction and Extreme Event Loads for Controlling Load Combinations (MTD 4-1 Attachment 4, Table 4)

The Structure Designer will use the information from the End Supports (Abutments) and the Intermediate Supports (Bents and Piers) tables to design the footings using the CTAbut and WinFoot programs, and will return the design details (dimensions, elevations, loads, and moments). If the range of footing sizes and resistances supplied by GS in Step 5 are not acceptable, the Structure Designer will request the entire procedure begin again with a new preliminary footing size.

Step 7: Evaluate Inclination Factors (if omitted, then go to Step 8)

After receiving the tables from Step 6, calculate the settlement and bearing capacity to verify the footing design and calculate inclination factors to determine if the gross nominal bearing resistance is affected. In general, inclination factors should be omitted when calculating the gross nominal bearing resistance (AASHTO 10.6.3.1.2a-1), however there are some cases when it may be necessary to consider these factors.

Unusual column geometry or loading configurations may require consideration be given to evaluating load inclination factors. A column that is not aligned normal to the footing bearing surface would be one example where inclination factors would be given consideration.

In the rare case where inclination factors are to be evaluated, use the lower value of the bearing resistance using either inclination factors or shape factors. Applying both shape and inclination factors will result in overly conservative design. If the inclination factor produces a lower bearing resistance than the shape factor, the nominal bearing resistances determined in Steps 3 and 4 must be recalculated using the inclination factors in place of the shape factors and provided to the Structure Designer.

Step 8: Verify the Shallow Foundation Design

Use the updated information (MTD 4-1, Attachment 4) to calculate the eccentricities and determine the effective footing dimensions. Check the footing design for all limit states:

- For the Service Limit State, verify that the Permissible Net Contact Stress is greater than or equal to the Net Uniform Bearing Stress (soil) or Net Maximum Bearing Stress (rock).
- For soil foundations, verify if the differential settlement between adjacent supports, using the Net Uniform Bearing Stresses, is acceptable.
- For the Strength and Extreme Event Limit States, verify that the calculated Factored Gross Nominal Bearing Resistance is greater than or equal to the Gross Uniform Bearing Stress (soil) and Gross Maximum Bearing Stress (rock).

If any of the above criteria are not met, inform the Structure Designer.

Step 9: Global Stability Check

The calculated resistance factors for global stability must be less than or equal to the AASHTO LRFD criteria for both the Service-I (static) Limit State ($\varphi = 0.65$), as well as the Extreme Event (pseudo-static) Limit State ($\varphi = 0.9$). If the stability analysis resistance factors are greater than the resistance factors listed in AASHTO 10.5.2.3 and 11.6.2.3, inform the Structure Designer and discuss design alternatives or site improvements to address the issue.

Step 10: Reporting

Report foundation recommendations as required by the *Foundation Reports for Bridges* module.

Appendix 1: Shallow Foundation Design Example (Bridge Abutment)

A spread footing is to be designed for an abutment on a soil slope. The Geoprofessional has received the foundation report request memorandum, the Foundation Data Table, the General Plan, the Foundation Plan, and has completed the field investigation.

Foundation Data Table (MTD 4-1, Attachment 1)

Support No.	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Estimated Footing Dimensions (ft)		Permissible Settlement under Service-I Load (inches)
			B	L	
Abut 1	5	0	10	64	1.0
Abut 2	5	0	10	64	1.0

Information from Site Investigation:

- Soil Identification: Silty Sand (SM) with Gravel; well-graded sand; some silt; little fine gravel.
- Apparent Density: Dense and very dense based on Standard Penetration Test (SPT).
- Groundwater Elevation = -100 ft.

Step 1: Initial Evaluation of Shallow Foundation

The site investigation shows the footing location is acceptable. There are no known geologic hazards that would preclude the use of shallow foundations.

Step 2: Calculate the Permissible Net Contact Stress (Service Limit State)

Calculate the Permissible Net Contact Stress (q_{pn}) that produces the permissible settlement of 1 inch for the footing dimensions provided by the Structure Designer in the Foundation Data table, as well as for a range of other footing widths.

The Hough formula (shown below) is used to calculate settlement as the cohesionless soil is loaded from the proposed footing. The apparent density of the soil varies with depth, therefore multiple layers are used to calculate the total settlement.

$$S_e = \sum_{i=1}^{n} \Delta H_i$$

$$\Delta H_i = \left(\frac{H_c}{C'}\right) \left[\log\left(\frac{\sigma'_o + \Delta\sigma'_v}{\sigma'_o}\right)\right]$$

where:

S_e = total elastic settlement

n = number of soil layers within zone of stress influence of the footing

ΔH_i = elastic settlement of layer *i*

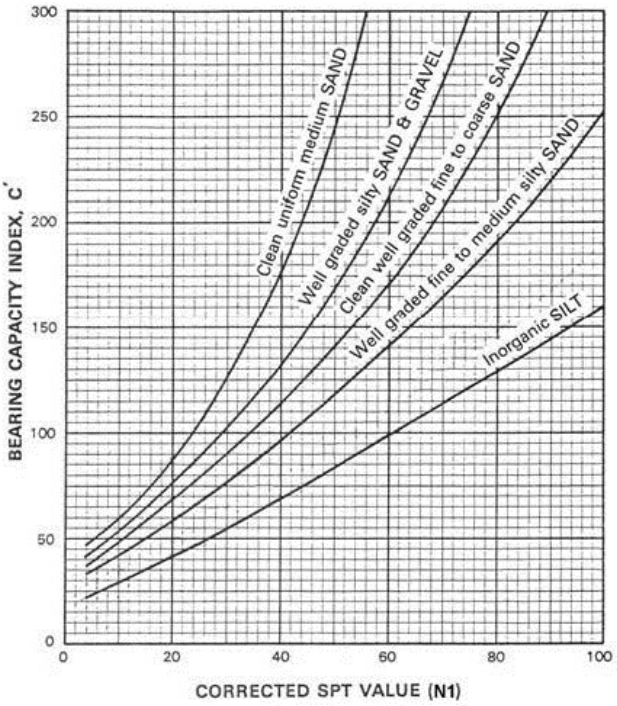
H_c = initial thickness of layer *i* (maximum 10 ft per AASHTO C10.6.2.4.2)

C' = bearing capacity index (from AASHTO Figure 10.6.2.4.2-1)

σ'_o = initial vertical effective stress at midpoint of layer *i*

$\Delta\sigma'_v$ = increase in effective vertical stress at midpoint of layer *i*

Bearing Capacity Index
(per AASHTO, Figure 10.6.2.4.2-1)



- *N1 shall be taken as $N1_{60}$ (SPT blow count corrected for both overburden and hammer efficiency).*

To calculate the permissible settlement of 1 inch, first pick a depth of influence based on the footing dimensions and soil types. Here, the depth of influence is assumed to be three times the footing width (3B = 30 feet). Situations in which the depth of influence should be greater are when the incremental rate of settlement has not tapered off or reduced in the soil layers near a depth of 3B (e.g., if the lowest layer contributed less than 5% of the total settlement, contributions for deeper layers can be considered insignificant assuming demonstrably more compressible soils do not exist below).

The increase in effective vertical stress ($\Delta\sigma'_v$) is determined by projecting an equivalent footing area at the midpoint of each layer assuming a 1:2 (H:V) pressure distribution that extends down from the bottom of the footing (Figure 1). At this stage, the footing dimension is assumed to have an eccentricity of zero (B=B' and L=L'). Eccentricity effects are considered in Step 8.

The pressure distribution used to determine the total settlement is shown below. To calculate the initial effective overburden pressure, use the finished grade above the toe of footing as the original ground elevation (elev. +5 ft in the example shown Figure 1). The applied load is the load at which the cumulative settlement = 1.0 inch. In this example, a 4032-kip load produced 1 inch of settlement for an effective footing width of 10 feet (Table 1).

Figure 1: Elevation View Showing Pressure Distribution Used for Estimating Settlement

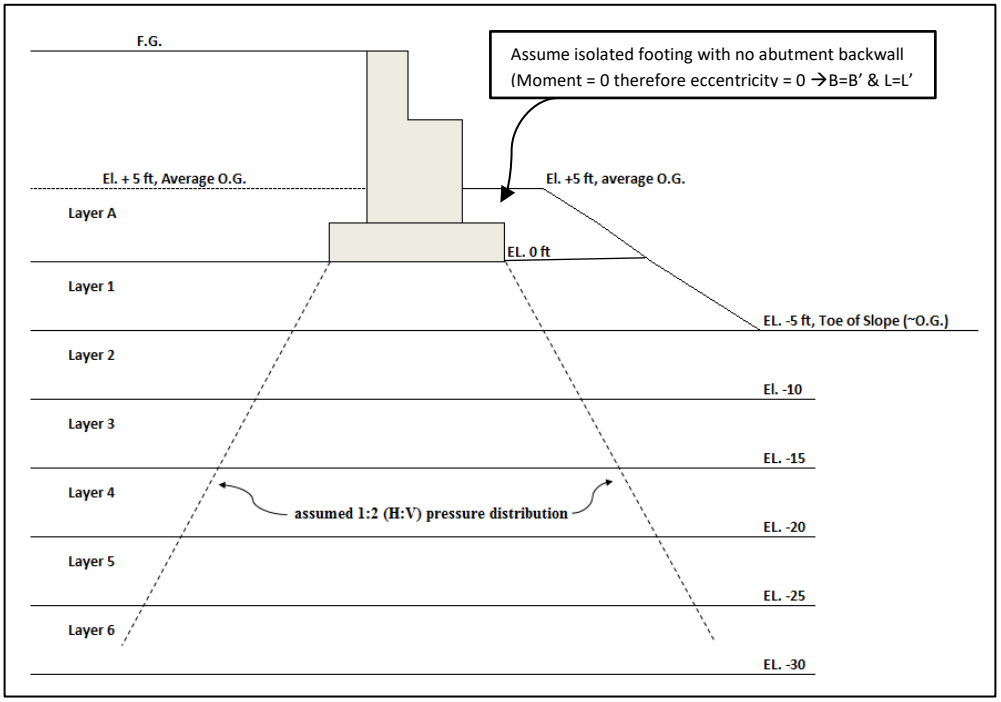


Table 1: Calculating Settlement using the Hough Method

(Footing Size, B x L = 10 feet x 64 feet)													
(Applied Load = 4032 kips)													
Layer No.	Bottom of Layer Elevation (ft)	Layer Thickness H_i (ft)	Depth Below Footing (ft)	Effective Unit Wt. γ' (pcf)	Effective Overburden Pressure at Mid-Layer σ'_o (ksf)	Depth to mid-layer (ft)	SPT Corrected Blow Count $N1_{60}$	Estimated Bearing Capacity Index C'	Projected Equivalent Footing Area at Mid-Layer (ft^2)	Increase in Effective Vertical Stress at Mid-Layer $\Delta\sigma'_v$ (ksf)	$\frac{\sigma'_o+\Delta\sigma'_v}{\sigma'_o}$	Layer Sett. (in)	Cumulative Settlement S_e (in)
A	0.0	5.0	0.0	125	-	-	-	-	-	-	-	-	-
1	-5.0	5.0	5.0	120	0.93	2.5	32	110	831	4.9	6.24	.43	.43
2	-10.0	5.0	10.0	120	1.53	7.5	34	115	1251	3.2	3.11	.27	.70
3	-15.0	5.0	15.0	120	2.13	12.5	38	125	1721	2.3	2.10	.15	.85
4	-20.0	5.0	20.0	120	2.73	17.5	42	140	2241	1.8	1.66	.09	.94
5	-25.0	5.0	25.0	125	3.34	22.5	68	250	2811	1.4	1.43	.04	.98
6	-30.0	5.0	30.0	125	3.96	27.5	68	250	3431	1.2	1.30	.03	1.00

The Permissible Net Contact Stress (q_{pn}) is:

q_{pn} = Load (to induce 1" settlement) / Effective Footing Area = 4032 k / (10 ft * 64 ft) = **6.3 ksf**

Step 3: Calculate Gross Nominal Bearing Resistance (qn)

Since the abutment footing is on a slope the "N_γ" term must be replaced with $N_{\gamma q}$, and N_q = 0, per AASHTO, Sec. 10.6.3.1.2c.

For cohesionless soils: $q_n = 0.5 * \gamma * B' * N_{\gamma q} * s_\gamma * C_{w\gamma}$

where:

γ = total unit weight of soil

B'= effective footing width

$N_{\gamma q}$ = modified bearing capacity factor for footing on/near a slope

s_γ = footing shape correction factor

$C_{w\gamma}$ = correction factor to account for the location of the groundwater table

Calculations:

$N_{\gamma q}$ = 32.5 (AASHTO figure 10.6.3.1.2c-2)

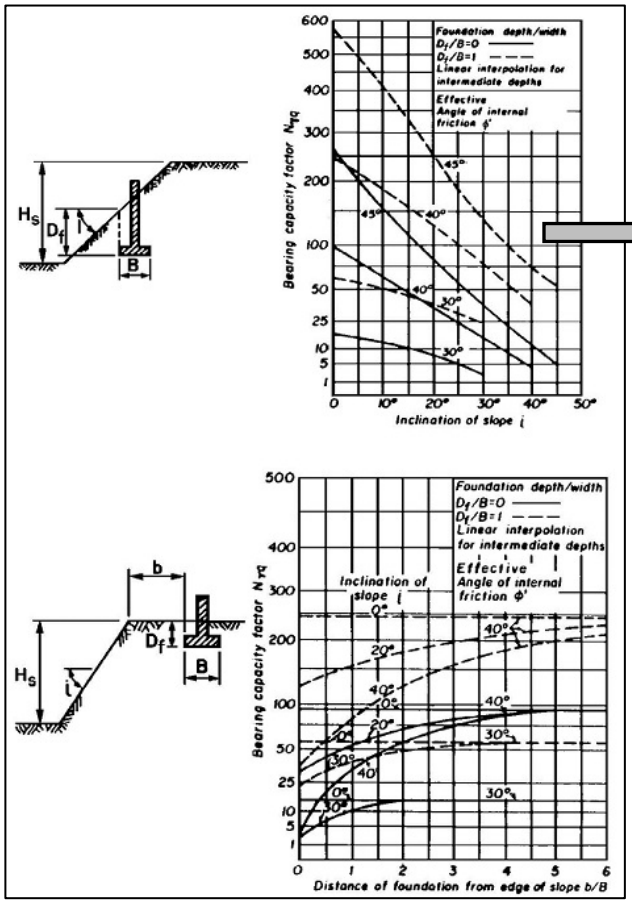
s_γ = 1 - 0.4(B' / L') = 1 – 0.4(10/64) = 0.94 (AASHTO table 10.6.3.1.2a-3)

$C_{w\gamma}$ =1.0 (depth to groundwater is 100 ft) (AASHTO table 10.6.3.1.2a-2)

q_n = 0.5 * 120 pcf * 10 ft * 32.5 * 0.94 * 1.0 = <u>18.3 ksf</u>

Modified Bearing Capacity Factor $N_{\gamma q}$ for Cohesionless Soil

AASHTO Figure 10.6.3.1.2c-2



D_f /B' = 5 ft /10 ft = 0.5 (D_f is the depth of footing from finish grade)
Use the top figure to interpolate for D_f /B' = 0.5 and ϕ_f =34°

D_f/B'	Inclination of Slope = 26.5°		
	$N_{\gamma q}$ -$\phi_{f\text{-}30}$	$N_{\gamma q}$-$\phi_{f\text{-}34}$	$N_{\gamma q}$ -$\phi_{f\text{-}40}$
0	5.0	11.6	21.6
0.50		**32.5**	
1	27.9	53.4	92.0

Use the top figure to find $N_{\gamma q}$ -$\phi_{f\text{-}30}$ & $N_{\gamma q}$ -$\phi_{f\text{-}40}$ for D_f /B'=0 (5.0, 21.6)
Use the top figure to find $N_{\gamma q}$ -$\phi_{f\text{-}30}$ & $N_{\gamma q}$ -$\phi_{f\text{-}40}$ for D_f /B'=1 (27.9, 92.0)

$N_{\gamma q}$ Interpolation:

Step 1:
Interpolate for $N_{\gamma q}$ -$\phi_{f\text{-}34}$ for D_f /B'=0
Interpolate between 5.0 and 21.6 to get $N_{\gamma q}$ -$\phi_{f\text{-}34}$=11.6

Step 2:
Interpolate for $N_{\gamma q}$ -$\phi_{f\text{-}34}$ for D_f /B'=1
Interpolate between 27.9 and 92.0 to get $N_{\gamma q}$ -$\phi_{f\text{-}34}$=53.4

Step 3:
Interpolate for $N_{\gamma q}$ -$\phi_{f\text{-}34}$ for D_f /B'=0.5
Interpolate between 11.6 and 53.4 to get

$N_{\gamma q}$ -$\phi_{f\text{-}34}$ = 32.5 for D_f /B'=0.5

Shape Factors (after AASHTO LRFD 10.6.3.1.2a)

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c , s_γ , s_q	$\phi_f = 0$	$1 + (\frac{B'}{5L'})$	1.00	1.00
	$\phi_f > 0$	$1 + (\frac{B'}{L'})(\frac{N_q}{N_c})$	$1 - 0.4\,(\frac{B'}{L'})$	$1 + (\frac{B'}{L'}\tan\phi_f)$

Correction Factor for Location of Water

Depth of Ground Water Table, D_w	$C_{w\gamma}$	C_{wq}
0	0.5	0.5
D_f	0.5	1.0
> (1.5B' + D_f)	1.0	1.0

Step 4: Calculate Factored Gross Nominal Bearing Resistance (q_R)

Determine the Factored Gross Nominal Bearing Resistance (q_R) for Strength and Extreme Event Limit States by multiplying the Gross Nominal Bearing Resistance (q_n) by the appropriate resistance factors for the Strength and Extreme Event Limit States (AASHTO CA Amendments Table 10.5.5.2.2-1). Here the resistance factor (φ_b) is 0.45 for the Strength Limit State because strength values were based on SPT values and 1.0 for the Extreme Event Limit State.

$q_R = \varphi_b * q_n$

q_R = 0.45 * 18.3 ksf = 8.2 ksf (Strength Limit State)

q_R = 1.0 * 18.3 ksf = 18.3 ksf (Extreme Event Limit State)

Repeat Step 2 through Step 4 for several footing dimensions. Present results in the *End Supports (Abutments)* table (see below).

At this stage, the actual footing dimension is assumed to have an eccentricity of zero, (B = B' and L = L'). Eccentricity effects are considered in Step 8.

Step 5: Send Preliminary Data to Structure Design

Email the *End Supports (Abutments)* table to the Structure Designer.

End Supports (Abutments)

Support Number: Abut 1
Foundation Material (Soil or Rock): Soil
Friction Angle or Undrained Shear Strength: 34°
Permissible Settlement (in): 1
Resistance Factor (Strength) – φ_b: 0.45
Resistance Factor (Seismic) – φ_b: 1.0

	Total Number of B' =	5		
No	Effective Footing Width	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B' (feet)	q_n (ksf)	q_{pn} (ksf)	q_R (ksf)
1	8	17.2	6.9	7.8
2	10	18.3	6.3	8.2
3	12	19.4	5.8	8.7
4	14	20.4	5.5	9.2
5	16	21.4	5.2	9.6

- Select “Soil” or “Rock” depending on design methodology used.
- Based on L’ =____ ft.

Step 6: Request Design Data from Structure Design

Request that the Structure Designer return the following tables (MTD 4-1 Attachment 4) with the controlling load combination information:

- *Foundation Data*
- *LRFD Service-I Limit State Loads for Controlling Load Combination*
- *LRFD Strength, Construction, and Extreme Event Loads for Controlling Load Combinations*

Step 7: Evaluate Inclination Factors

Nothing to do here. Since axial and shear forces are checked against the available resistance using effective footing dimensions in the respective directions (i.e. bearing capacity and sliding), inclination factors were omitted.

Step 8: Verify the Shallow Foundation Design

Receive and review the design tables (requested in Step 6), the footing design information, and controlling load confirmation. The physical footing width is 16 feet.

Spread Footing Design Information (from Structure Designer)

Support No.	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Abutment 1	+5.0	0	16.0	64	1
Abutment 2	+5.0	0	16.0	64	1

LRFD Service-1 Limit State Loads for Controlling Load Combination (from Structure Designer)

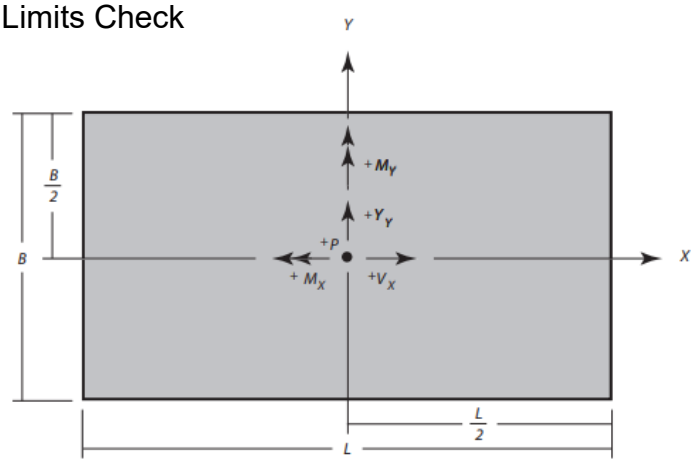
Support Location	Total Load					Permanent Load				
	P_{Total} Net (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{Perm} Net (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)
Abutment 1	2668	6058	N/A	N/A	888	2338	3558	N/A	N/A	686
Abutment 2	2668	6058	N/A	N/A	888	2338	3558	N/A	N/A	686

Note: P_{Total} Net is provided in the table above, however, for calculating eccentricity P_{Total} Gross is required. The Geoprofessional will need to ask for this information in addition to the MTD tables.

LRFD Strength and Extreme Event Loads for Controlling Load Combination Provided by Structure Designer

Support Location	Strength Limit State (Controlling Group)					Extreme Event Limit State (Controlling Group)				
	P_{Total} Gross (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{Total} Gross (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)
Abutment 1	3058	11158	N/A	N/A	1068	N/A	N/A	N/A	N/A	N/A
Abutment 2	3058	11158	N/A	N/A	1068	N/A	N/A	N/A	N/A	N/A

1. Eccentricity Limits Check



For Service Limit State:

- Maximum eccentricity is not to exceed B/6 for footings on soil where B is the actual footing width.
- Eccentricity, $e_y = M_x / P_{Total, Gross}$

  Where:

  e_y= eccentricity

  M_x = factored bending moment

  P_{Total}, Gross = total gross factored axial force

  (In the future, SD will provide $P_{Total, Gross}$ for the Service-I Limit State but until MTD 4-1 is updated, the Geoprofessional will need to request $P_{Total, Gross}$ from SD).

Calculations:

$e_y = M_x / P_{Total, Gross}$

e_y = 6058 kips-ft / 2888 kips = 2.1 ft

B / 6 = 16 ft / 6 = 2.7 ft (maximum allowed eccentricity for soil)

e_y < B/6 (2.1 ft < 2.7 ft) for footing on soil, O.K.

No need to check eccentricity limits under Extreme Event Limit State load combinations because loads were not provided by Structure Design. If loads were provided then we would repeat the above calculations to check eccentricity for the Extreme Event Limit State.

2. Calculate the Permissible Net Contact Stress after using loads and moments to determine the eccentricity and the effective footing dimensions. Verify that the calculated Permissible Net Contact Stress is greater than or equal to the Net Uniform Bearing Stress ($q_{pn} \geq q_{n,u}$).

   Calculate B' for Service I Limit State:

   $B' = B - 2 * e_y$

   B' = 16 ft – 2 * 2.1 ft = 11.8 ft

   $q_{nu} = P_{Total, Net} / (B' * L')$ = 2668 kips / (11.8 ft * 64 ft) = 3.5 ksf (Demand)

   q_{pn} = 5.9 ksf (repeat Step 2 to calculate q_{pn} with B' = 11.8 ft) (Stress required to induce 1" settlement)

   $q_{pn} > q_{n,u}$ (5.9 ksf > 3.5 ksf) O.K.

3. Calculate the Factored Gross Nominal Bearing Resistance for Strength Limit State (and Extreme Event Limit State, if applicable) using loads, moments, and corresponding effective footing dimensions. Verify that the calculated Factored Gross Nominal Bearing Resistance is greater than or equal to the Gross Uniform Bearing Stress ($q_R \geq q_{g,u}$).

   Calculate B' for Strength Limit State: $B' = B - 2 * e_y$

   Where:

   $e_y = M_x / P_{Total\ Gross}$

$B' = B - 2 * M_x / P_{Total\ Gross}$

$B' = 16\ ft - 2 * 11158\ kips\text{-}ft / 3058\ kips = 8.7\ ft$

Calculate $q_{g,u}$ and q_R:

$N_{\gamma q} = 35.6$ (repeat Step 3 to calculate $N_{\gamma q}$ with $D_f / B' = 5.0/8.7\ ft = 0.57$)

$q_{g,u} = P_{Total\ Gross} / (B' * L') = 3058\ kips / (8.7\ ft * 64\ ft) = 5.5\ ksf$ (Demand)

$q_R = \varphi_b * 0.5 * \gamma' * B' * N_{\gamma q} * s_\gamma * C_{w\gamma}$

$= 0.45 * 0.5 * .120\ kcf * 8.7\ ft * 35.6 * (1 - 0.4 * (8.7\ ft / 64\ ft)) * 1 = 7.9\ ksf$ (Resistance)

$q_R > q_{g,u}$ (7.9 ksf > 5.5 ksf) O.K.

If any of the above design criteria are not met, inform the Structure Designer.

Step 9: Global Stability Check

The global stability was determined using the Slide program. The calculated resistance factors for global stability met the current AASHTO LRFD criteria for both the Service-I (static) Limit State, φ = 0.65 (equivalent to a factor of safety ~ 1.5 for Slide), as well as the Extreme Event (pseudo-static) Limit State, φ = 0.9 (equivalent to a factor of safety ~ 1.1 for Slide).

Step 10: Design Approval and Reporting

Email the Structure Designer to communicate design approval. Prepare the Foundation Report as per *Foundation Reports for Bridges*. The required tables are presented below with values generated from this example (abutment 1 only).

Foundation Design Recommendations for Spread Footing
(after MTD 4-1, Attachment 5, Table 2)

Support Location	Footing Size (feet)		Bottom of Footing Elevation (feet)	Minimum Footing Embedment Depth (feet)	Total Permissible Support Settlement (inches)	Service Limit State	Strength or Construction Limit State (ϕb=0.45)	Extreme Event Limit State (ϕb=1.0)
	B	L				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Abutment 1	16	64	0	5	1.0	5.9 (B' = 11.8 ft)	7.9 (B' = 8.7 ft)	N/A
Abutment 2						(B' =)	(B' =)	N/A

- Abutment 2 not completed in this example

Spread Footing Data Table
(after MTD 4-1, Attachment 5, Table 3)

Support Location	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (φ_b=0.45) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (φ_b=1.0) (ksf)
Abutment 1	5.9	7.9	N/A
Abutment 2			N/A

- Abutment 2 not completed in this example

Appendix 2: Shallow Foundation Design Example for Intermediate Bridge Support

A spread footing is designed for a single column bent supported by soil below level ground. The Geoprofessional has received the foundation report request memorandum, the Foundation Data Table, the General Plan, the Foundation Plan, and has completed the field investigation.

Foundation Data Table (MTD 4-1, Attachment 1)

Support No.	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Estimated Footing Dimensions (feet)		Permissible Settlement under Service-I Load (inches)	Approximate Ratio of Permanent/Total Service I Load
			B	L		
Bent 2	48.5	40.0	22	22	1.0	

Information from Site Investigation:

- Soil Identification: Silty Sand (SM) with Gravel; well-graded sand; some silt; little fine gravel.
- Apparent Density: Dense and very dense based on Standard Penetration Test (SPT).
- Groundwater Elevation = 10 ft.

Step 1: Initial Evaluation of Shallow Foundation

The site investigation shows the footing location is acceptable. There are no known geologic hazards that would preclude the use of shallow foundations.

Step 2: Calculate the Permissible Net Contact Stress (Service Limit State)

Calculate the Permissible Net Contact Stress (q_{pn}) that produces the permissible settlement of 1 inch for the footing dimensions provided by the Structure Designer in the Foundation Data Table, as well as for a range of other footing widths and lengths.

The Hough formula (shown below) is used to calculate settlement as the cohesionless soil is loaded from the proposed footing. The apparent density of the soil varies with depth, therefore multiple layers are used to calculate the total settlement.

$$S_e = \sum_{i=1}^{n} \Delta H_i$$

$$\Delta H_i = \left(\frac{H_c}{C'}\right) \left[\log\left(\frac{\sigma'_o + \Delta\sigma'_v}{\sigma'_o}\right)\right]$$

where:

S_e = total elastic settlement

n = number of soil layers within zone of stress influence of the footing

ΔH_i = elastic settlement of layer *i*

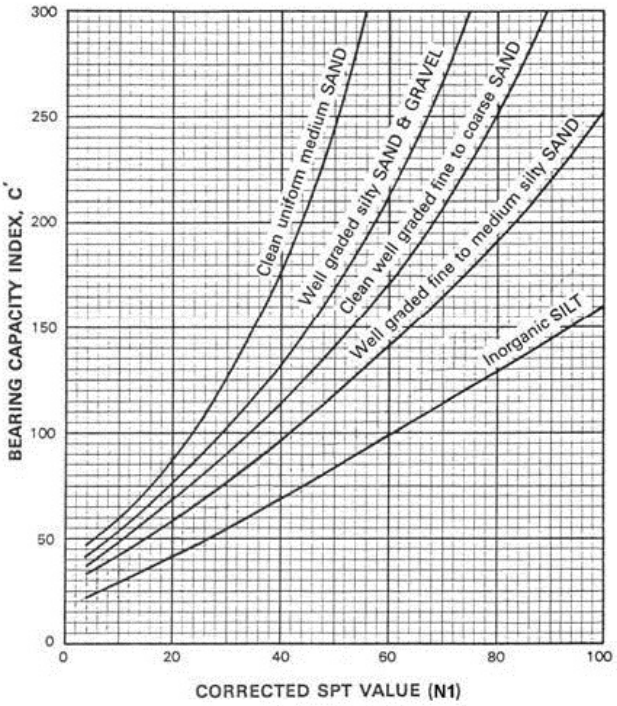
H_c = initial thickness of layer *i* (maximum 10 ft per AASHTO C10.6.2.4.2)

C' = bearing capacity index (from AASHTO Figure 10.6.2.4.2-1)

σ'_o = initial vertical effective stress at midpoint of layer *i*

$\Delta\sigma'_v$ = increase in effective vertical stress at midpoint of layer *i*

Bearing Capacity Index
(per AASHTO, Figure 10.6.2.4.2-1)



- *N1 shall be taken as $N1_{60}$ (SPT blow count corrected for both overburden and hammer efficiency).*

To calculate the permissible settlement of 1 inch first pick a depth of influence based on the footing dimensions and soil types. For this example, the depth of influence is assumed to be three times the footing width (3B = 66 feet). The depth of influence should be increased when significant settlement occurs in the lower layers.

The increase in effective vertical stress ($\Delta\sigma'_v$) is calculated by projecting an equivalent footing area at the midpoint of each layer assuming a 1:2 (H:V) pressure distribution (Figure 1).

The distributed stress used to determine the total settlement is shown in the figure below. In this example, a 2710-kip load produced 1 inch of settlement (Table 1).

Figure 1: Elevation View Showing Pressure Distribution Used for Estimating Settlement

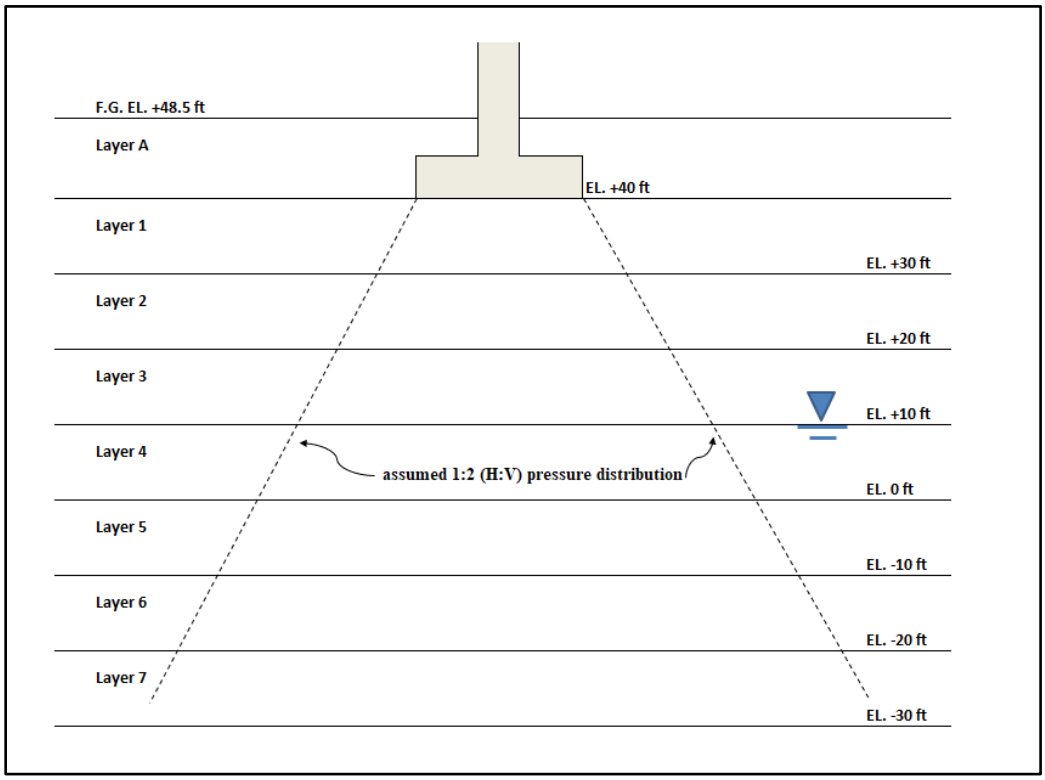


Table 1: Determining Settlement using Hough Method

(Footing Size, B x L = 22 feet x 22 feet)													
(Applied Load = 2710 kips)													
Layer No.	Bottom of Layer Elevation (feet)	Layer Thickness H_i (feet)	Depth Below Footing (feet)	Effective Unit Wt. γ' (pcf)	Effective Overburden Pressure at Mid-Layer σ'_o (ksf)	Depth to mid-layer (feet)	SPT Corrected Blow Count $N1_{60}$	Estimated Bearing Capacity Index C'	Projected Equivalent Footing Area at Mid-Layer (feet^2)	Increase in Effective Vertical Stress at Mid-Layer $\Delta\sigma'_v$ (ksf)	$\frac{\sigma'_o+\Delta\sigma'_v}{\sigma'_o}$	Layer Sett. (inches)	Cumulative Settlement S_e (inches)
A	40.0	8.5	0.0	125	0.53	4.25	36	-	-	-	-	-	-
1	30.0	10.0	10.0	125	1.69	5.0	38	119	729	3.7	3.2	.51	.51
2	20.0	10.0	20.0	125	2.94	15.0	32	102	1369	2.0	1.7	.26	.77
3	10.0	10.0	30.0	125	4.19	25.0	40	125	2209	1.2	1.3	.11	.88
4	0.0	10.0	40.0	63	5.13	35.0	43	134	3249	0.8	1.2	.06	.94
5	-10.0	10.0	50.0	63	5.76	45.0	37	116	4489	0.6	1.1	.04	.98
6	-20.0	10.0	60.0	63	6.39	55.0	50	156	5929	0.5	1.1	.02	1.00
7	-30.0	10.0	70.0	63	7.02	65.0	60	210	7569	0.4	1.1	.01	1.01

The *Permissible Net Contact Stress* (q_{pn}) is:

q_{pn} = Load (to induce 1" settlement) / Effective Footing Area = 2710 k / (22 ft * 22 ft) = **5.6 ksf**

(Note: At this initial stage, the actual footing dimension is assumed to have an eccentricity of zero, (B = B' and L = L')

Step 3: Calculate Gross Nominal Bearing Resistance (q_n)

For cohesionless soils: $q_n = (\gamma * D_f * N_q * s_q * C_{wq}) + (0.5 * \gamma * B' * N_\gamma * s_\gamma * C_{w\gamma})$

where:

γ = total unit weight of soil

D_f = footing embedment depth

B'= effective footing width

N_q and N_γ = bearing capacity factors for footing on level ground

s_q and s_γ = footing shape correction factors

C_{wq} and $C_{w\gamma}$ = correction factors to account for the location of the groundwater table

Calculations:

For ϕ=35°, N_q = 33.3 and N_γ = 48.0 (AASHTO figure 10.6.3.1.2a-1)

s_q = 1 + (B'/L' * tan ϕ) = 1 + (22/22 * tan 35) = 1.7 (AASHTO table 10.6.3.1.2a-3)

s_γ = 1 - 0.4 * (B'/ L') = 1 – 0.4 * (22/22) = 0.6 (AASHTO table 10.6.3.1.2a-3)

C_{wq} = 1.0 and $C_{w\gamma}$ = 0.96 for depth to groundwater of 30 ft (AASHTO table 10.6.3.1.2a-2

q_n = (0.125 kcf * 8.5 * 33.3 * 1.7 * 1.0) + (0.5 * 0.125 kcf * 22 ft * 48.0 * 0.6 * 0.96) = 98.2 ksf

Shape Factors (after AASHTO LRFD 10.6.3.1.2a-3)

Factor	Friction Angle	Cohesion Term (s_c)	Unit Weight Term (s_γ)	Surcharge Term (s_q)
Shape Factors s_c , s_γ , s_q	$\phi_f = 0$	$1 + (\frac{B'}{5L'})$	1.00	1.00
	$\phi_f > 0$	$1 + (\frac{B'}{L'})(\frac{N_q}{N_c})$	$1 - 0.4\ (\frac{B'}{L'})$	$1 + (\frac{B'}{L'}\tan\phi_f)$

Correction Factor for Location of Water (after AASHTO LRFD 10.6.3.1.2a-2)

Depth of Ground Water Table, D_w	**Cwγ**	**Cwq**
0	0.5	0.5
D_f	0.5	1.0
>(1.5B' + D_f)	1.0	1.0

Step 4: Calculate Factored Gross Nominal Bearing Resistance (q_R)

Determine the Factored Gross Nominal Bearing Resistances (q_R) for Strength and Extreme Event Limit States by multiplying the Gross Nominal Bearing Resistance (q_n) by the appropriate resistance factors for the Strength and Extreme Event Limit States (AASHTO CA Amendments Table 10.5.5.2.2-1). Here the resistance factor (φ_b) is 0.45

for the Strength Limit State because strength values were based on SPT values and 1.0 for the Extreme Limit State.

$q_R = \varphi_b * q_n$

$q_R = 0.45 * 98.2$ ksf = 44.2 ksf (Strength Limit State)

$q_R = 1.0 * 98.2$ ksf = 98.2 ksf (Extreme Event Limit State)

Repeat Steps 2 through 4 for the other footing widths and L/B ratios. Present results in the *Intermediate Supports (Bents)* table (see example below).

Notice that the column heading “Effective Footing Width” is a misnomer because the actual footing width was used in the calculation. Eccentricity effects are considered in Step 8.

Step 5: Send Preliminary Data to Structure Design

Email the *Intermediate Supports (Bents)* table to the Structure Designer.

Intermediate Supports (Bents)

Support Number: Bent 2
Foundation Material (Soil or Rock): Soil
Friction Angle or Undrained Shear Strength: 35°
Permissible Settlement (in): 1.0
Resistance Factor (Strength) – φ_b: 0.45
Resistance Factor (Seismic) – φ_b: 1.0

Total Number of unique *L′/B′*	5
Total Number of *B*′s per *L′/B′*	5

No	Effective Footing Width	Effective Footing Size Ratio	Gross Nominal Bearing Resistance	Permissible Net Contact Stress (Settlement)	Factored Gross Nominal Bearing Resistance (Strength)
	B′ (ft)	*L′ / B′*	q_n (ksf)	q_{pn} (ksf)	q_R (ksf)
1	14	1.00			
2	18	1.00			
3	22	1.00	98.2	5.6	44.2
4	26	1.00			
5	30	1.00			
1	14	1.25			
2	18	1.25			
3	22	1.25			
4	26	1.25			
5	30	1.25			
1	14	1.50			
2	18	1.50			
3	22	1.50			
4	26	1.50			
5	30	1.50			
1	14	1.75			
2	18	1.75			
3	22	1.75			
4	26	1.75			
5	30	1.75			
1	14	2.00			
2	18	2.00			
3	22	2.00			
4	26	2.00			
5	30	2.00			

Step 6: Request Design Data from Structure Design

Request that the Structure Designer send the following tables (MTD 4-1 Attachment 4) with the appropriate load information.

- *Foundation Data*
- *LRFD Service-I Limit State Loads for Controlling Load Combination*
- *LRFD Strength, Construction, and Extreme Event Loads for Controlling Load Combinations*

Step 7: Evaluate Inclination Factors

Nothing to do here. Since axial and shear forces are checked against the available resistance using effective footing dimensions in the respective directions (i.e. bearing capacity and sliding), inclination factors were omitted.

Step 8: Verify the Shallow Foundation Design

The Structure Designer sends the following new tables as requested in Step 6. The physical footing width is 22 feet.

Spread Footing Design Information Provided by the Structure Designer

Support No.	Finished Grade Elevation (feet)	Bottom of Footing Elevation (feet)	Footing Dimensions (feet)		Permissible Settlement under Service Load (inches)
			B	L	
Bent 2	48.5	40.0	22	22	1

LRFD Service-1 Limit State Loads for Controlling Load Combination Provided by Structure Designer

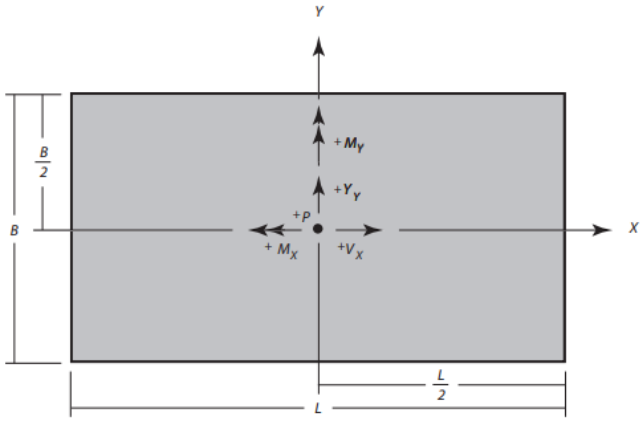
Support Location	Total Load					Permanent Load				
	P_{Total} (kips) Net	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{Perm} (kips) Net	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)
Bent 2	1287	3697	583	N/A	N/A	927	2062	108	N/A	N/A

Note: P_{Total}, Net is provided in the table, however, for calculating eccentricity P_{Total}, Gross is required. The Geoprofessional will need to ask for this information in addition to the MTD tables.

LRFD Strength and Extreme Event Loads for Controlling Load Combination
Provided by Structure Designer

Support Location	Strength Limit State (Controlling Group)					Extreme Event Limit State (Controlling Group)				
	P_{Total} Gross (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)	P_{Total} Gross (kips)	M_X (kip-ft)	M_Y (kip-ft)	V_X (kips)	V_Y (kips)
Bent 2	2287	2260	3140	N/A	N/A	1400	10588	10588	398	398

1. Eccentricity Limits Check



For Service Limit State:

- Maximum eccentricity is limited to B/6 for footings on soil. B is the footing width.
- Eccentricity, $e_y = M_x / P_{Total, Gross}$, $e_x = M_y / P_{Total, Gross}$

  (ask Structure Designer for the $P_{Total, Gross}$ for the Service-I Limit State)

  Where:

  $e_{x,y}$ = eccentricity

  $M_{x,y}$ = factored bending moment

  $P_{Total, Gross}$ = total gross factored axial force

Service Limit State Calculations:

$e_y = M_x / P_{Total, Gross}$

e_y = 3697 kips-ft / 1570 kips = 2.4 feet

B / 6 = 22 ft / 6 = 3.7 ft (maximum allowed eccentricity for soil)

e < B/6 (2.4 ft < 3.7 ft) for footing on soil, O.K.

For Extreme Event Limit State:

- Maximum eccentricity is limited to B/2.5 for footings on soil. B is the footing width.
- Eccentricity, $e_y = M_x / P_{Total, Gross}$

Extreme Event Limit State Calculations:

$e_y = M_x / P_{Total, Gross}$

e_y = 10588 kips-ft / 1400 kips = 7.6 feet

B / 2.5 = 22 ft / 2.5 = 8.8 ft (maximum allowed eccentricity for soil)

e < B/2.5 (7.6 ft < 8.8 ft) for footing on soil, O.K.

2. Calculate the Permissible Net Contact stress after using loads and moments to determine the eccentricity and the effective footing dimensions. Verify that the calculated Permissible Net Contact Stress is greater than or equal to the Net Uniform Bearing Stress ($q_{pn} \geq q_{n,u}$).

Calculate B' for Service I Limit State:

$B' = B - 2 * e_y$

B' = 22 ft – 2 * 2.4 ft = 17.2 ft

Calculate L' for Service I Limit State:

$L' = L - 2 * e_x$

$e_x = M_y / P_{Total, Gross}$

e = 583 kips-ft / 1570 kips = 0.37 feet

L' = 22 ft – 2 * 0.37 ft = 21.3 ft

$q_{n,u} = P_{Total, Net} / (B' * L') = 1287$ kips / (17.2 ft * 21.3 ft) = 3.5 ksf (Demand)

q_{pn} = 6.2 ksf (repeat Step 2 to calculate q_{pn} with B' = 17.2 ft and L' = 21.3 ft) (Stress required to induce 1" settlement)

$q_{pn} > q_{n,u}$ (6.2 ksf > 3.5 ksf) O.K.

3. Calculate the Factored Gross Nominal Bearing Resistance for Strength Limit State (and Extreme Event Limit State, if applicable) using loads, moments, and corresponding effective footing dimensions. Verify that the calculated Factored Gross Nominal Bearing Resistance is greater than or equal to the Gross Uniform Bearing Stress ($q_R \geq q_{g,u}$).

   Calculate B' and L' for Strength Limit State:

   - $B' = B - 2 * e_y$

     Where:

     $e_y = M_x / P_{Total\ Gross}$

     $B' = B - 2 * M_x / P_{Total\ Gross}$

     B' = 22 ft – 2 * 2260 kips-ft / 2287 kips = 20.0 ft (this will become the L')

   - $L' = L - 2 * e_x$

     Where:

     $e_x = M_y / P_{Total\ Gross}$

     $L' = L - 2 * M_y / P_{Total\ Gross}$

     L' = 22 ft – 2 * 3140 kips-ft / 2287 kips = 19.2 ft (this will become the B')

     Switch the B' and L'

     B' is the smaller of ($B - 2 * e_y$) and ($L - 2 * e_x$). In this example for the Strength Limit State design, B' changed from the longitudinal to the transverse direction. B' is 19.2 ft in transverse direction and L' is 20.0 ft is longitudinal direction.

   Calculate q_R and $q_{g,u}$ for Strength Limit State:

   - $q_{g,u} = P_{Total\ Gross} / (B' * L') = 2287$ kips / (19.2 ft * 20.0 ft) = 6.0 ksf (Demand)
   - $q_n = (\gamma * D_f * N_q * s_q * C_{wq}) + (0.5 * \gamma * B' * N_\gamma * s_\gamma * C_{w\gamma})$

Calculations:

N_q = 33.3 and N_γ = 48.0 (AASHTO figure 10.6.3.1.2a-1)

$s_q = 1 + (B'/L' * \tan \phi) = 1 + (19.2/20.0 * \tan 35) = 1.7$ (AASHTO table 10.6.3.1.2a-3)

$s_\gamma = 1 - 0.4 * (B'/ L') = 1 – 0.4 * (19.2/20.0) = 0.6$ (AASHTO table 10.6.3.1.2a-3)

C_{wq} = 1.0 and $C_{w\gamma}$ = 1.0 for depth to groundwater of 30 ft (AASHTO table 10.6.3.1.2a-2)

$q_n = (0.125 \text{ kcf} * 8.5 * 33.3 * 1.7 * 1.0) + (0.5 * 0.125 \text{ kcf} * 19.2 \text{ ft} * 48.0 * 0.6 * 1.0)$

$= 94.7$ ksf

$q_R = \varphi_b * q_n = 0.45 * 94.7 \text{ ksf} =$ <u>42.6 ksf</u> (Resistance)

$q_R > q_{g,u}$ (42.6 ksf > 6.0 ksf) O.K.

Calculate B' and L' for Extreme Event Limit State:

- $B' = B – 2 * e_y$

  Where:

  $e_y = M_x / P_{\text{Total Gross}}$

  $B' = B – 2 * M_x / P_{\text{Total Gross}}$

  B' = 22 ft – 2 * 10588 kips-ft / 1400 kips = 6.9 ft

- $L' = L – 2 * e_x$

  Where:

  $e_x = M_y / P_{\text{Total Gross}}$

  $L' = L – 2 * M_y / P_{\text{Total Gross}}$

  L' = 22 ft – 2 * 10588 kips-ft / 1400 kips = 6.9 ft

Calculate q_R and $q_{g,u}$ for Extreme Event Limit State:

- $q_{g,u} = P_{\text{Total Gross}} / (B' * L') = 1400$ kips / (6.9 ft * 6.9 ft) = <u>29.4 ksf</u> (Demand)
- $q_n = (\gamma * D_f * N_q * s_q * C_{wq}) + (0.5 * \gamma * B' * N_\gamma * s_\gamma * C_{w\gamma})$

Calculations:

N_q = 33.3 and N_γ = 48.0 (AASHTO figure 10.6.3.1.2a-1)

$s_q = 1 + (B'/L' * \tan \phi) = 1 + (6.9/6.9 * \tan 35) = 1.7$ (AASHTO table 10.6.3.1.2a-3)

$s_\gamma = 1 - 0.4 * (B'/ L') = 1 – 0.4 * (6.9/6.9) = 0.6$ (AASHTO table 10.6.3.1.2a-3)

$C_{wq} = 1.0$ and $C_{w\gamma}$ = 1.0 for depth to groundwater of 30 ft (AASHTO table 10.6.3.1.2a-2)

q_n = (0.125 kcf * 8.5 * 33.3 * 1.7 * 1.0) + (0.5 * 0.125 kcf * 6.9 ft * 48.0 * 0.6 * 1.0)

= 72.6 ksf

$q_R = \varphi_b * q_n$ = 1.0 * 72.6 ksf = <u>72.6 ksf</u> (Resistance)

$q_R > q_{g,u}$ (72.6 ksf > 29.4 ksf) O.K.

Step 9: Global Stability Check

Global stability calculations need not be performed in this example because the footing is on level ground.

Step 10: Design Approval and Reporting

Email the Structure Designer to communicate design approval. Write the Foundation Report as per *Foundation Reports for Bridges*.

Foundation Design Recommendations for Spread Footing
(after MTD 4-1, Attachment 5, Table 2)

Support Location	Footing Size (feet)		Bottom of Footing Elevation (feet)	Minimum Footing Embedment Depth (feet)	Total Permissible Support Settlement (inches)	Service Limit State	Strength Limit State (φ_b=0.45)	Extreme Event Limit State (φ_b=1.0)
	B	L				Permissible Net Contact Stress (ksf)	Factored Gross Nominal Bearing Resistance (ksf)	Factored Gross Nominal Bearing Resistance (ksf)
Bent 2	22	22	40	8.5	1.0	6.2 (B' = 17.2 feet)	42.6 (B' = 19.2 feet)	72.6 (B' = 6.9 feet)

Spread Footing Data Table
(after MTD 4-1, Attachment 5, Table 3)

Support Location	Service Permissible Net Contact Stress (Settlement) (ksf)	Strength/Construction Factored Gross Nominal Bearing Resistance (φ_b=0.45) (ksf)	Extreme Event Factored Gross Nominal Bearing Resistance (φ_b=1.0) (ksf)
Bent 2	6.2	42.6	72.6

Driven Pile Foundations

This document presents the design methods and communication steps between Structure Design (SD) and Geotechnical Services (GS) for the load and resistance factor design (LRFD) of driven pile foundations used for support of bridges, retaining walls, non-standard walls, signs, and other structures. This module discusses design of driven Standard Plan Class piles (Alternative V, W, X, and Y), Cast-In-Steel-Shell (CISS), and H-piles. The Appendices include three design calculation examples.

Standards relating to driven pile foundation investigations, design, and reporting are:

- Seismic Design Criteria (SDC)
- AASHTO LRFD Bridge Design Specifications with CA Amendments (AASHTO)
- American Petroleum Institute (API, 2000) publication RP 2A
- Standard Specifications, Standard Plans, Bridge Standard Detail Sheets (XS Sheets)
- Bridge Memos to Designers (MTD) 3-1, *Deep Foundations*
- Bridge Design Aids
- Bridge Construction Records and Procedures Manual, Volume II
- Geotechnical Manual
    - *Foundation Reports for Bridges*
    - *Geotechnical Investigations*

Geotechnical Service's role in driven pile foundation design is to provide the Structure Designer with a foundation report addressing the following:

- Design Tip Elevations for piles for Service, Strength and Extreme Event Limit State.
- The Controlling Design Tip Elevation.
- The estimated Nominal Driving Resistance.
- Pile acceptance criteria.
- Recommendations relating to specifications and construction.

The Structure Designer's role in deep foundation design includes:

- Providing GS with the foundation design data and factored design load information.
- Providing GS with the latest plan sheets pertinent to foundation design (e.g. General Plan, Foundation Plan, Foundation Detail Sheets, etc.).

Investigations

The goal of the geotechnical investigation for a driven pile foundation is to determine the properties and behaviors of the soil and/or rock, and the groundwater condition that can affect foundation design and construction. All subsurface conditions that might influence the foundation design and performance should be investigated.

Perform a literature search (see *Geotechnical Investigations*) to gather all relevant information related to site geology, strength of soil and rock, and geologic hazards. Then develop a prudent exploration plan considering site constraints, geologic variability, and available resources. Borings should be located as close as possible to the proposed foundation.

The exploration plan should include:

- An appropriate number of exploratory borings and/or cone penetration tests (CPT) to develop the design soil profile (AASHTO Table 10.4.2-1).
- An appropriate depth of exploration for the borings or CPT. The depth of exploration should generally extend below the anticipated pile tip elevation a minimum of 20 feet, or a minimum of two times the maximum pile group dimension, whichever is deeper (AASHTO Table 10.4.2-1).
- Standard penetration tests (SPT). When SPTs are to be performed, sampling intervals should be limited to no more than 5 feet.
- Groundwater measurements.
- Soil and water samples for corrosion testing in accordance with current *Caltrans Corrosion Guidelines*.
- Adequate samples for laboratory testing (e.g. classification tests, consolidation test, soil strength parameters required for design).

Design

The following provides design methodologies used for calculating settlement (Service-I Limit State) and pile resistance (Strength and Extreme Event Limit States) in accordance with AASHTO. For appropriate resistance factors refer to AASHTO Table 10.5.5.2.3-1.

Design methods for calculation of axial pile resistance for various pile and soil types are presented in AASHTO 10.7.3.8.6.

For steel pipe and CISS piles larger than 18" in diameter, the static analysis methods from the American Petroleum Institute (API, 2000) publication RP 2A must be used. It is important to note, the CISS pile tip resistance is the sum of tip resistance of steel shell section and the lesser of (1) Soil plug tip resistance, or (2) Soil plug side resistance based on the final plug length after the pile concrete placement. The plug length is limited to 4 times the pile diameter.

The soil properties used in design should come from: (1) SPT correlations (see *Soil Correlations* module) and/or (2) results from laboratory tests under similar field conditions.

The design must also account for geologic hazards such as:

- Liquefaction (see *Liquefaction* module)
- Lateral spreading (see *Lateral Spreading* module)
- Scour: Foundations that are constructed in a watercourse must meet AASHTO guidelines regarding scour depths (AASHTO C2.6.4.4.2). The top of the pile cap must be below the degradation plus contraction scour depth. The bottom of the pile cap must be below the degradation plus contraction plus local pier scour depth.

Design Information and Communication (Preliminary Foundation Report)

After the field investigation and testing has been completed, the Geoprofessional must review the design information provided by the Structure Designer which should include:

- General Plan
- Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Design Process (Preliminary Foundation Report)

Complete the driven pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Preliminary Tip Elevations

- Calculate the preliminary tip elevations meeting the controlling compression and tension requirements for the Strength Limit State at each support location.

Step 3: Complete Preliminary Foundation Design Recommendations table.

- Present the tip elevations for compression in the Preliminary Foundation Design Recommendations table under the Preliminary Tip Elevation column in the Preliminary Foundation Report.

Step 4: Reporting

- Complete the Preliminary Foundation Report according to the *Foundation Reports for Bridges* module.

Design Information and Communication (Foundation Report)

The Geoprofessional must review the design information provided by the Structure Designer which should include:

- General Plan
- Foundation Plan
- Scour Data Table (MTD 3-1, Attachment 1) or Hydraulics Report (if scour potential exists)
- Foundation Design Data Sheet (MTD 3-1, Attachment 1, also below)
- Foundation Factored Design Loads information (MTD 3-1, Attachment 1, also below)

Table X: Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1	Class 140 Alt “V”						
Pier 2	24” diam. CISS						
Abut 3	Class 140 Alt “V”						

Table X: Foundation Factored Design Loads (MTD 3-1, Attachment 1)

Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load per Support	Permanent Load per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Design Process (Foundation Report)

Complete the driven pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Design Tip Elevations for the Piles

- Calculate the design tip elevations meeting the controlling compression and tension requirements for the Strength Limit State and the Extreme Event Limit State at each support location.
- Calculate the design tip elevations meeting the permissible settlement criteria for Service-I Limit State. *(Commentary: Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations. For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived. If the design tip for service limit state is waived, then a note should be placed at the bottom of the pile data table).*
- Using the design tip elevation from Strength or Extreme Limit State, verify the settlement under the Service-I Limit State load is less than the permissible settlement.

Step 3: Complete the Tables

- Present the tip elevations for compression, tension, and settlement in the Foundation Design Recommendations and Pile Data Tables under the Design Tip Elevation column in the draft Foundation Report.

Step 4: Prepare and Send Draft Foundation Report

- Complete the Draft Foundation Report according to the *Foundation Reports for Bridges* module.

Step 5: Determine the Specified Tip Elevation

- The Structure Designer will compare their design tip elevation for lateral loading to the Design Tip Elevations in the Draft Foundation Report. The Structure Designer will communicate this information to the GP.
    - If the lateral tip is higher than or equal to the design tip elevation then the GP can complete the tables for the Foundation Report and proceed to Step 6.
    - If the lateral tip is lower than the design tip elevation then the GP must verify that the pile can be installed to the lateral tip elevation.

Step 6: Determine the Required Nominal Driving Resistance

- For piles driven through zones not utilized for developing the required axial geotechnical resistance (e.g., scourable or liquefiable layers), determine the total driving resistance to reach the specified tip elevation. If the conditions described above are not applicable, then list the Nominal Resistance.

Step 7: Prepare and Send Foundation Report

- Once the design of the driven pile foundation is completed, the Geoprofessional will complete the Foundation Report according to the *Foundation Report for Bridges* module.

Driven Pile Acceptance Criteria

The following information relates to driving system submittals, dynamic monitoring, and Wave Equation Analysis Program (WEAP) bearing acceptance criteria (AASHTO LRFD BDS, CA Amendment 10.7.3.8.4), and pile load testing.

- Require a driving system submittal (see *Notes for Specifications*) for all non-standard plan piles, except H-Piles. For all other piles, the need for a driving system submittal should be evaluated based upon anticipated pile installation difficulties and/or concern about pile damage during installation. The Foundation Testing and Instrumentation Branch must review and approve all driving system submittals submitted to GS by Structure Construction.
- For driven piles with a nominal resistance greater than 600 kips, dynamic monitoring and analyses are required to develop the bearing acceptance criteria.
- For driven piles with a diameter, or side dimension, of 18 inches to 36 inches, dynamic monitoring and analyses are required to develop the bearing acceptance criteria.
- For driven piles that are larger than 36 inches, select dynamic monitoring and a pile load test is required to measure the nominal resistance. Exceptions to the requirement to perform an axial load test may include situations where geologic conditions (e.g., bearing in rock or very dense soil) provide confidence in the nominal resistance development. These cases must be reviewed by Foundation Testing and Instrumentation Branch and require approval of a design exception from the Office of Geotechnical Design Policies and Practices (OGDPP) and Structures Policy and Innovation (SP&I).

The Geoprofessional must consult with the Foundation Testing and Instrumentation (FTI) branch for all language and requirements included in the Pile Load Testing and Dynamic Monitoring section of the Foundation Report. The FTI branch will provide technical assistance in determining dynamic monitoring details, order of work, and layout of the load test piles. In addition, FTI can assist with contractual details and information for the special provisions. If a foundation type that would require a pile load test is being considered, the Geoprofessional must have a meeting with the FTI branch and the

Structure Designer to review proposed pile load test loading schedule, layout, and to determine if any test or anchor piles may be incorporated in the proposed bridge foundation.

Attachments

- Appendix A: Example Design Calculations for Concrete Pile
- Appendix B: Example Design Calculations for H-Pile
- Appendix C: Example Calculations for CISS Pile

Appendix A: Nordlund Method and Alpha (α) Method for Concrete Pile

This design example presents calculations for the Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except for designing for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- 12” square concrete pile is used for this example.
- Foundation information and loads provided in the tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement Under Service Load (inches)	Number of Piles per Support
				B	L		
Pier 2	Class 140 Alt. X	100.0	90.0	14	126	2	82

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	5370	4730	8770	150	0	0	1100	120	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 220 kips and 120 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- Pile Perimeter (C_d) = 4.0'
- Pile width: 12" = 1.0'
- Area of Concrete pile end: A_p = 1.0 ft^2
- Pile taper angle, $\omega = 0^\circ$
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Pier 2	95.0	5.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and Short-Term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
    1. The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
    2. Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- For this example, the Strength Limit State pile tip elevation is based on 100% Long-Term (Degradation + Contraction) scour + 50% Short-Term (Local) scour. For vertical effective stress calculations, only Long-Term scour is considered (down to elevation 95 ft), therefore only five feet of Layer #1 contributes to vertical effective stress. Short-Term scour does not affect vertical effective stress. In this example 50% of Short-Term scour is at elevation 92.5 ft. If the Geoprofessional is performing analysis for the Extreme Event Limit State, 0% of Short-Term scour would be used for both vertical effective stress and pile-soil contact area calculations.

- AASHTO LRFD Bridge Design Specifications – California Amendments, Figure C2.6.4.4.2-2, shows that the bottom of pile cap must be at or below the total scour elevation. The cutoff elevation is at elevation 90.0 feet. This is the total scour elevation and below the calculated elevation for pile-soil contact. Therefore, elevation 90.0 feet is the top of the pile-soil contact in the nominal resistance calculations.

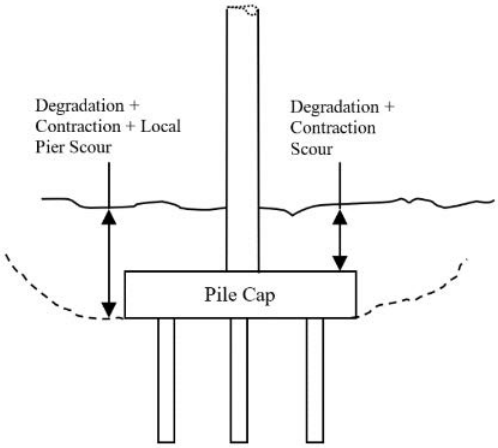
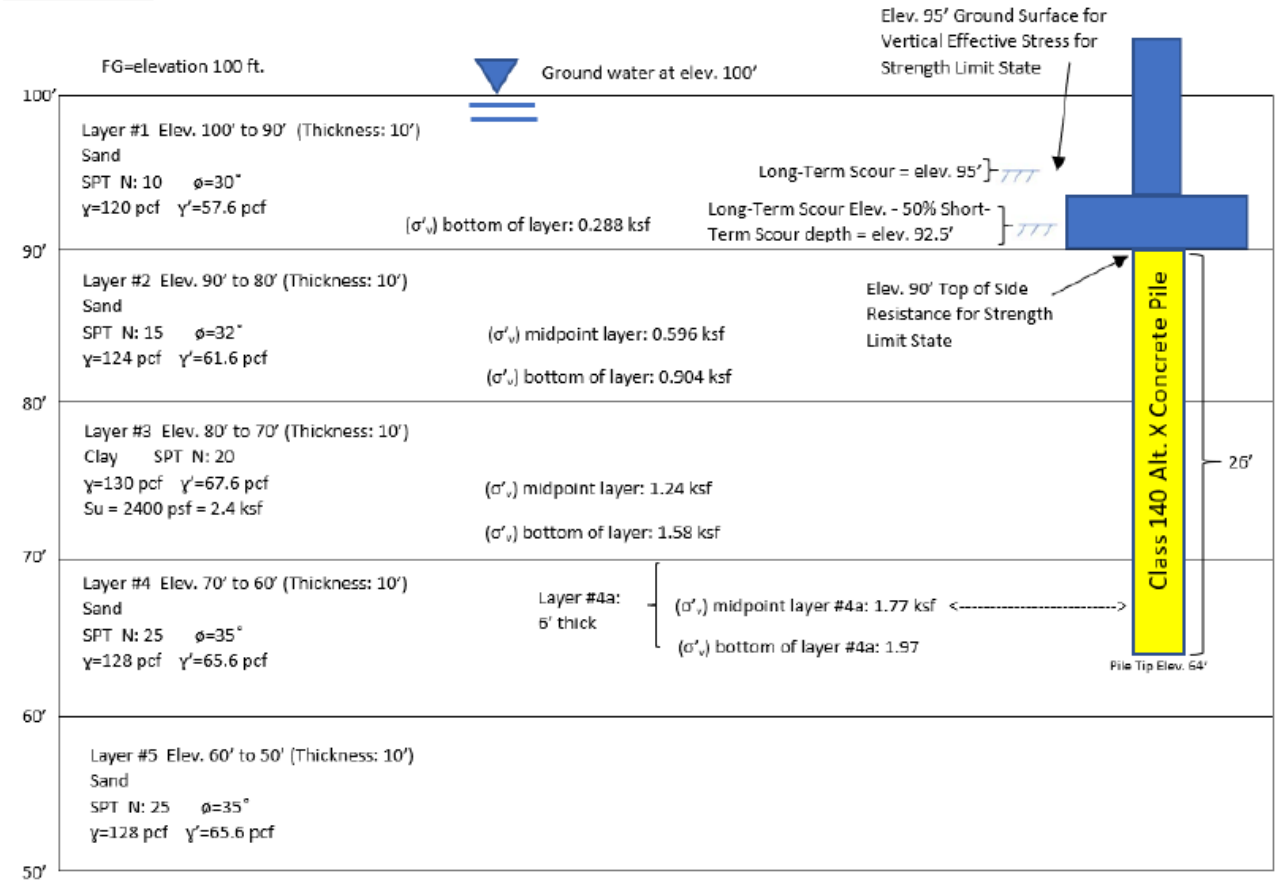


Figure C2.6.4.4.2-2 Deep Foundation Location

Soil Profile:



For cohesionless soil layers, use the Nordlund method as shown in Section 10.7.3.8.6f of AASHTO LRFD Bridge Design Specifications. For cohesive soil layers, use the Alpha (α) method as shown in Section 10.7.3.8.6b of AASHTO LRFD Bridge Design Specifications. The following steps should be used to calculate the nominal pile resistance (R_n), which is the sum of the pile side resistance (R_s) and the pile tip resistance (R_p).

Step 1: Divide the soils into layers and determine the ɣ for all soil layers, determine the ø angle for each cohesionless soil layer, and the undrained shear strength S_u for each cohesive soil layer. (See Soil Profile above).

Step 2: Determine δ, the friction angle between the pile and soil based on displaced soil volume, V, and the soil friction angle, ø.

Step 2a: Compute volume of soil displaced per unit length of pile, V.

Area of pile tip: 12 in^2 = 1.0 ft^2, therefore: **V = 1.0 ft^3/ft**

Step 2b:

Using Figure 10.7.3.8.6f-6 from AASHTO LRFD Bridge Design Specifications, using V = 1.0 ft^3/ft, determine **δ/ϕ_f = 0.77**

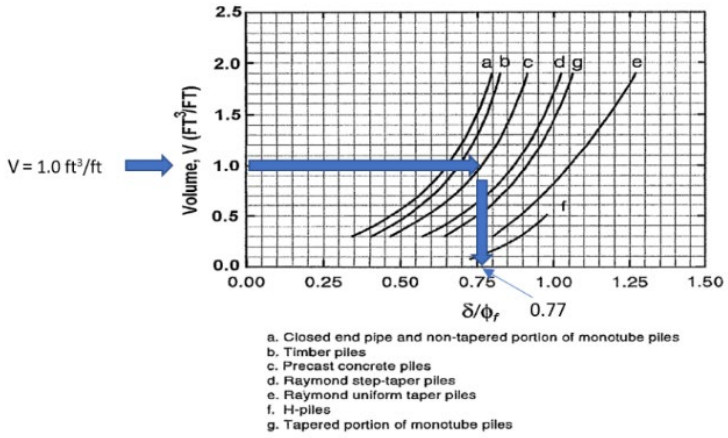


Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

Step 2c: For cohesionless layers, calculate δ from δ/ϕ_f ratio:

Soil layer #1: (0.77) * (30°) = δ = 23.1°
Soil layer #2: (0.77) * (32°) = δ = 24.6°
Soil layer #3: Cohesive
Soil layer #4: (0.77) * (35°) = δ = 27.0°

Step 3: For cohesionless layers, determine the coefficient of lateral earth pressure, K_δ, based on the ø angle of the soil layer.

Determine K_δ for ø angle based on displaced volume, V= 1.0 ft^3/ft, and pile taper angle, $\omega = 0^\circ$, using AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, interpolating where necessary. Figure 10.7.3.8.6f-2 below is an example for soil layer #1 with ø=30° and Figure 10.7.3.8.6f-3 below is an example for soil layer #4 with ø=35° using note "a" below. Soil layer #2 was determined by using the process in note "b", below). Figures 10.7.3.8.6f-1 and 4 are presented for reference.

Soil layer #1: ø=30° K_δ = 1.15
Soil layer #2: ø=32° K_δ = 1.39
Soil layer #3: Cohesive
Soil layer #4: ø=35° K_δ = 1.75

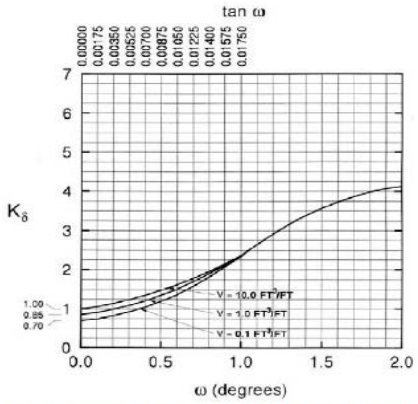


Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

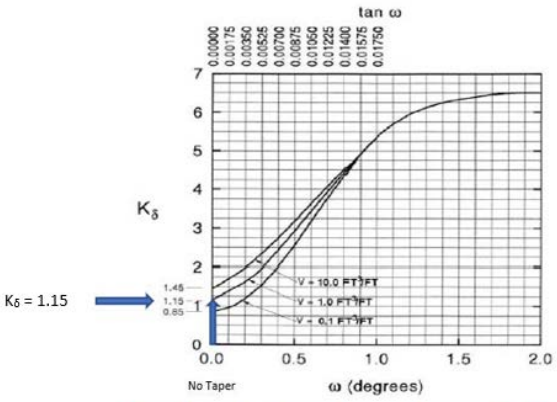


Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

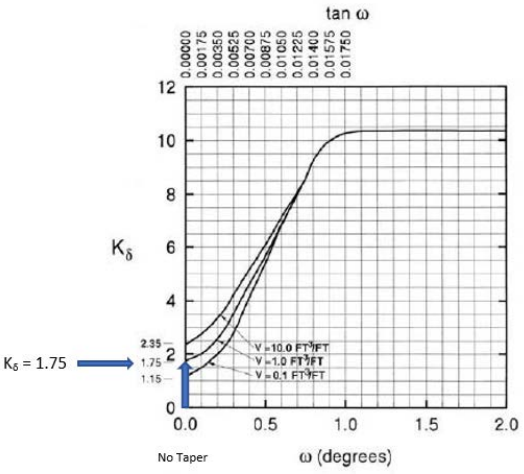


Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

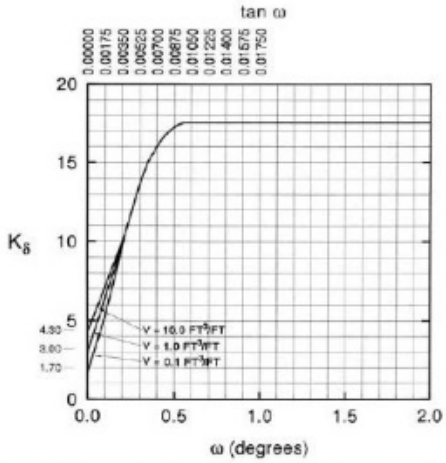


Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

Notes:

a) If the displaced volume is 0.1, 1.0 or 10.0 ft^3/ft, which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle is one of those provided, K_δ can be determined directly from the appropriate figure.

b) If the displaced volume is 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle is different from those provided, use linear interpolation to determine K_δ for the required ø angle.

c) If the displaced volume is other than 0.1, 1.0 or 10.0 ft^3/ft which corresponds to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle corresponds to one of those provided, use log linear interpolation to determine K_δ for the required displaced volume.

d) If the displaced volume is other than 0.1, 1.0, or 10.0 ft^3/ft which correspond to one of the curves provided in in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle does not correspond to one of those provided, first use linear interpolation to determine K_δ for the required ø angle at the displaced volume curves provided for 0.1, 1.0 and 10.0 ft^3/ft. Then use log linear interpolation to determine K_δ for the required displaced volume.

Step 4: For cohesionless layers, determine the correction factor, C_F, to be applied to K_δ, if $\delta \neq$ ø angle using AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-5. (Example below is for ø=30°). $\delta/\phi_f = 0.77$ is from Step 2b, above.

Soil layer #1: ø=30° $\delta/\phi_f = 0.77$ therefore $C_F = 0.92$
Soil layer #2: ø=32° $\delta/\phi_f = 0.77$ therefore $C_F = 0.91$
Soil layer #3: Cohesive
Soil layer #4: ø=35° $\delta/\phi_f = 0.77$ therefore $C_F = 0.89$

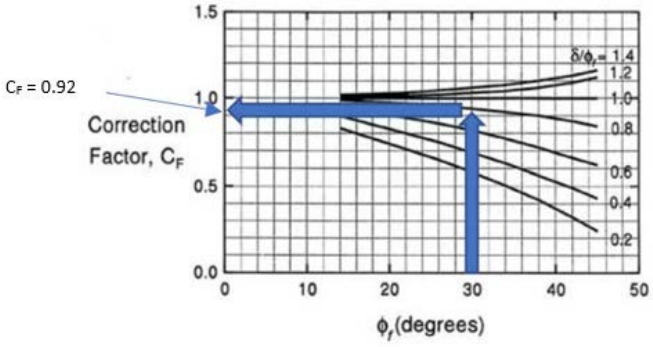


Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

Step 5: Compute the vertical effective stress (σ'_v) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Midpoint of Layer #1: Not relevant for this example
Bottom of Layer #1: 5'(γ'=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Midpoint of Layer #2: 5'(γ'=61.6 pcf) + 288 psf = 596 psf = 0.596 ksf
Bottom of Layer #2: 5'(γ'=61.6 pcf) + 596 psf = 904 psf = 0.904 ksf

Layer #3:
Midpoint of Layer #3: 5'(γ'=67.6 pcf) + 904 psf = 1242 psf = 1.24 ksf
Bottom of Layer #3: 5'(γ'=67.6 pcf) + 1242 psf = 1580 psf = 1.58 ksf

Layer #4:
Midpoint of Layer #4a: 3'(γ'=65.6 pcf) + 1580 psf = 1777 psf = 1.77 ksf
Bottom of Layer #4a: 3'(γ'=65.6 pcf) + 1777 psf = 1974 psf = 1.97 ksf

Step 6: For cohesive layers, determine the adhesion factor, α, using the applicable AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6b-1.

In this example, Layer #3 is below two sand layers, so the top diagram best fits the example.
Using an undrained shear strength (S_u) of 2.4 ksf for Layer #3, using a D_b=less than 10D, α = 1.0.

Where:
- D_b = Pile embedment into the clay layer (10' for this example).
- D = Pile width (1.0' for this example).

Layer # 3: **α = 1.0**

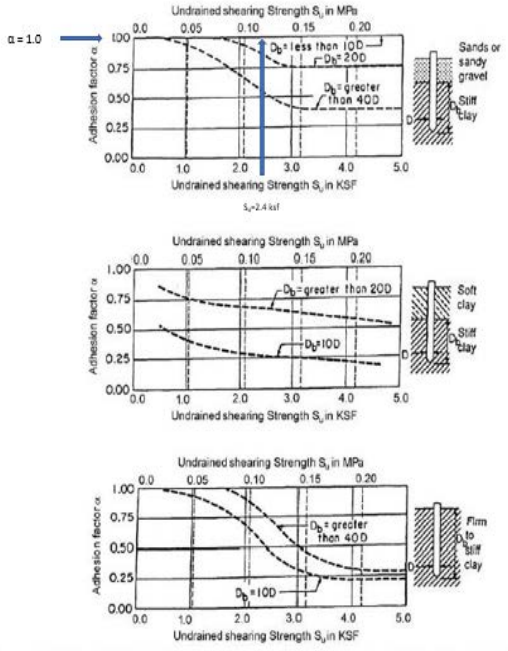


Figure 10.7.3.8.6b-1—Design Curves for Adhesion Factors for Piles Driven into Clay Soils after Tomlinson (1980)

Compute the Strength Limit State Compression Design Tip Elevation

Step 7: Compute the pile side resistance (R_s).

Step 7a: Compute the pile side resistance for each soil layer.

For cohesionless layers:

$q_s = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})$ (AASHTO 10.7.3.8.6f-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- K_δ = Coefficient of lateral earth pressure (from Step 3)
- C_F = Correction factor to be applied to K_δ (from Step 4)
- σ'_v = Vertical effective stress at midpoint of each relevant soil layer (from Step 5) (ksf)
- δ = Friction angle between the pile and soil (from δ in Step 2c)
- ω = Pile taper angle. (In this example no taper, so $\omega = 0^\circ$)
- C_d = Pile perimeter = 4.0’ for this example
- Δd = Thickness of soil layer (ft)

For cohesive layers:

$q_s = (\alpha)(S_u)$ (AASHTO 10.7.3.8.6b-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (\alpha)(S_u)(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- α = Adhesion factor (from Step 6)
- S_u = Undrained shear strength of the soil layer (see soil profile) (ksf)
- C_d = Pile perimeter = 4.0’ for this example
- Δd = Thickness of soil layer (ft)

Layer #1: No side resistance contribution from Layer #1

Layer #2: (cohesionless)

- (K_δ=1.39)(C_F=0.91)(σ'_v=0.596 ksf)($\frac{\sin(24.6^\circ+0^\circ)}{\cos 0^\circ}$)(C_d=4.0')(Δd=10.0') = 12.6 kips

Layer #3 (cohesive):

- (α=1.0)(S_u=2.4 ksf)(C_d=4.0')(Δd=10.0') = 96.0 kips

Layer #4 (cohesionless):

- (K_δ=1.75)(C_F=0.89)(σ'_v=1.77 ksf)($\frac{\sin(27.0^\circ+0^\circ)}{\cos 0^\circ}$)(C_d=4.0')(Δd=6.0') = 30.0 kips

Step 7b: After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the total pile side resistance (R_s).

- Σ R_s = 12.6 + 96.0 + 30 = 138 kips

Step 8: Determine the pile tip resistance (R_p):

$q_p = (\alpha_t)(N'_q)(\sigma'_v)$

$R_p = (q_p)(A_p) = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

Where:

- q_p = Nominal unit tip resistance (ksf)
- α_t = Coefficient from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7
- N'_q = Bearing capacity factor from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8
- σ'_v = Vertical effective stress at bottom of Layer #4a (at pile tip) (ksf)
- A_p = Area of the concrete pile end = 1.0 ft^2

Step 8a: Determine α_t coefficient by entering ø angle of the soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7 intersecting the D/b ratio line. For this example, ø=35°; D/b ratio is: Pile Length/Pile Width= 26'/1.0' = 26.

Therefore, **α_t = 0.67**

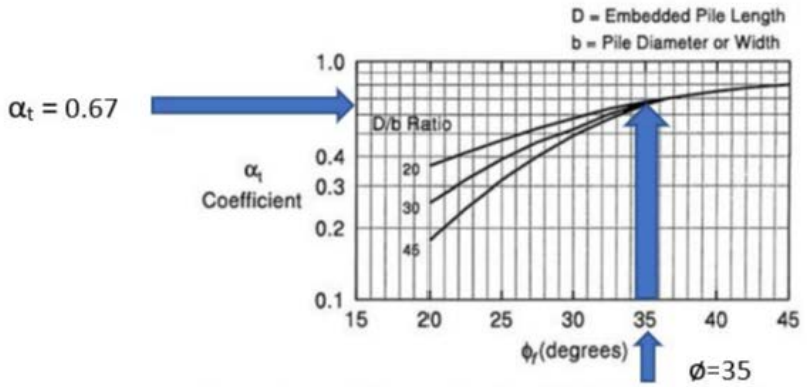


Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

Step 8b: Determine bearing capacity factor, N'_q , by entering ø angle of the soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8. For this example, ø=35°.

Therefore, **N'_q = 65**

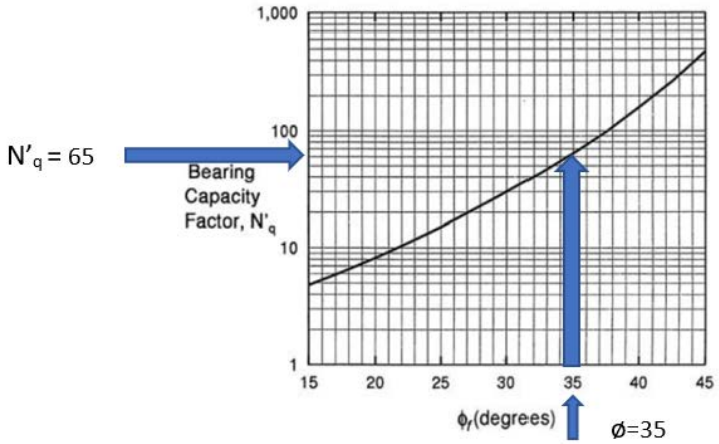


Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q (Hannigan et al., 2006 modified after Bowles, 1977)

Step 8c: Determine the vertical effective stress (σ'_v) at the pile tip.

The vertical effective stress (σ'_v) at the bottom of Layer #4a = 1.97 ksf. AASHTO LRFD Bridge Design Specifications section 10.7.3.8.6f states it must be limited to ≤ 3.2 ksf.

Therefore, σ'_v = 1.97 ksf.

STEP 9: Compute the pile tip resistance (R_p).

$R_p = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

$R_p = (0.67)(65)(1.97\ ksf)(1.0\ ft^2)$ = **86 kips**

Step 9a: Compare pile tip resistance (R_p) to the limiting unit pile tip resistance (q_L)

The calculated pile tip resistance must be compared to the limiting unit pile tip resistance (q_L). The limiting q_L value is obtained from entering the ø angle of soil at the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-9. For this example, ø=35°, therefore: **q_L = 100 ksf**

$R_{p\ limited} = (q_L)(A_p) = (100\ ksf)(1.0\ ft^2)$ = **100 kips**

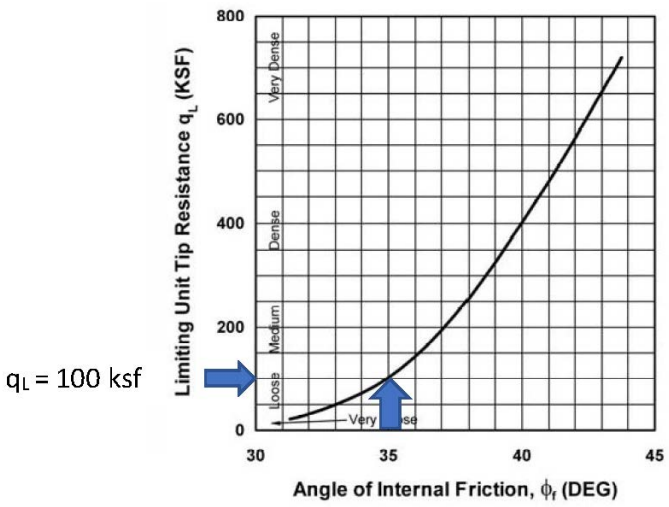


Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

Step 9b: Use the lesser of either the R_p or $R_{p\ limited}$ value. Therefore, use **R_p = 86 kips**

Note: If the pile tip is in a cohesive soil layer, the pile tip resistance (R_p) is determined from AASHTO LRFD Bridge Design Specifications equation 10.7.3.8.6e-1:

$R_p=(9S_u)\ (A_p)$

Step 10: Compute the Nominal Pile Resistance (R_n):

R_n = R_s + R_p = 138 kips + 86 kips = **224 kips** at the design tip elevation of 64.0 feet.

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (in)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp. (φ_{qs}=1.0) (φ_{qp}=1.0)	Tension (φ_{qs}=1.0)			
Pier 2	Class 140 Alt. X	90.0	5370	4730	2	220	0	120	0	64.0 (a-I)	64.0	224

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Pier 2	Class 140 Alt. X	220	0	64.0 (a)	64.0	224

Note: Design tip elevations are controlled by (a) Compression

Appendix B: Nordlund Method for H-pile

This design example is for Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except for designing for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- HP 10X57 pile is used for this example.
- Foundation information and loads will be provided from the Structure Designer, as shown in the example tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (ft)		Permissible Settlement Under Service Load (inches)	Number Of Piles Per Support
				B	L		
Pier 2	HP 10X57 H-pile	100.0	90.0	6	50	1	14

Foundation Factored Design Loads Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
	Total Load Per Support	Permanent Loads Per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	760	550	1090	150	0	0	1020	120	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 220 kips and 120 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- Pile Perimeter (Cd) = 3.37' (Using "box" area of pile side)
- H-pile width: 10.225" = 0.85'
- Area of H-pile steel section: 16.8 in2 = 0.117 ft2
- Pile taper angle, ω = 0°
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (ft)	Short-Term (Local) Scour Depth (ft)
Pier 2	95.0	5.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and short-term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
    1) The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
    2) Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- The Strength Limit State pile nominal resistance calculation is based on 100% long-term (Degradation + Contraction) scour + 50% short-term (local) scour. For vertical effective stress calculations, only long-term scour is considered (down to elevation 95 ft), therefore only five feet of Layer #1 contributes to vertical effective stress. Short-term scour does not affect vertical effective stress. In this example 50% of short-term scour is at elevation 92.5 ft. If the Geoprofessional is performing analysis for the Extreme Event Limit State, 0% of short-term scour would be used for both vertical effective stress and pile-soil contact area calculations.

- AASHTO LRFD Bridge Design Specifications – California Amendments, Figure C2.6.4.4.2-2, shows that the bottom of pile cap must be at or below the total scour elevation. The cutoff elevation is at elevation 90.0 feet. This is the total scour elevation and below the calculated elevation for pile-soil contact. Therefore, elevation 90.0 feet is the top of the pile-soil contact in the nominal resistance calculations.

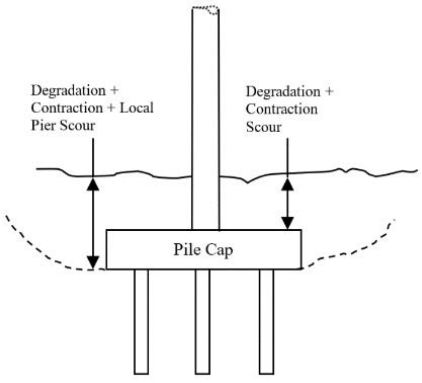
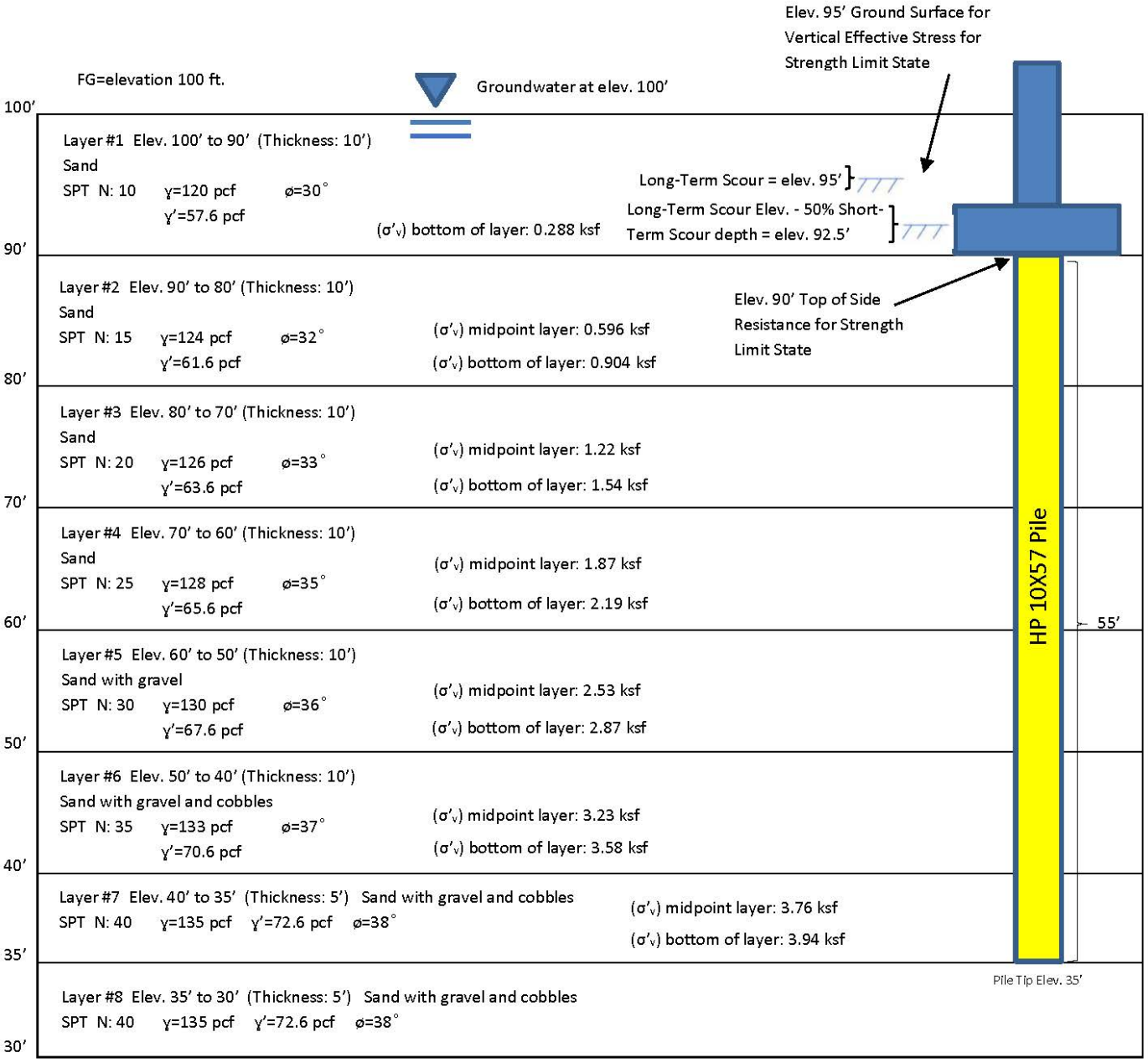


Figure C2.6.4.4.2-2 Deep Foundation Location

Soil Profile:



Using the Nordlund method as shown in Section 10.7.3.8.6f of AASHTO LRFD Bridge Design Specifications, the following steps should be used to calculate the nominal pile resistance (R_n), which is the sum of the pile side resistance (R_s) and the pile tip resistance (R_p).

Step 1: Divide the soils into layers and determine the ɣ and ø angle for each layer. (See Soil Profile above).

Step 2: Determine δ, the friction angle between the pile and soil based on displaced soil volume, V, and the soil friction angle, ø.

Step 2a: Compute volume of soil displaced per unit length of H-pile, V.

Area of H-pile steel section: 16.8 in^2 = 0.117 ft^2, therefore: **V = 0.117 ft^3/ft**

Step 2b: Using Figure 10.7.3.8.6f-6 from AASHTO LRFD Bridge Design Specifications, using V = 0.117 ft^3/ft, determine **δ/ϕ_f = 0.761**

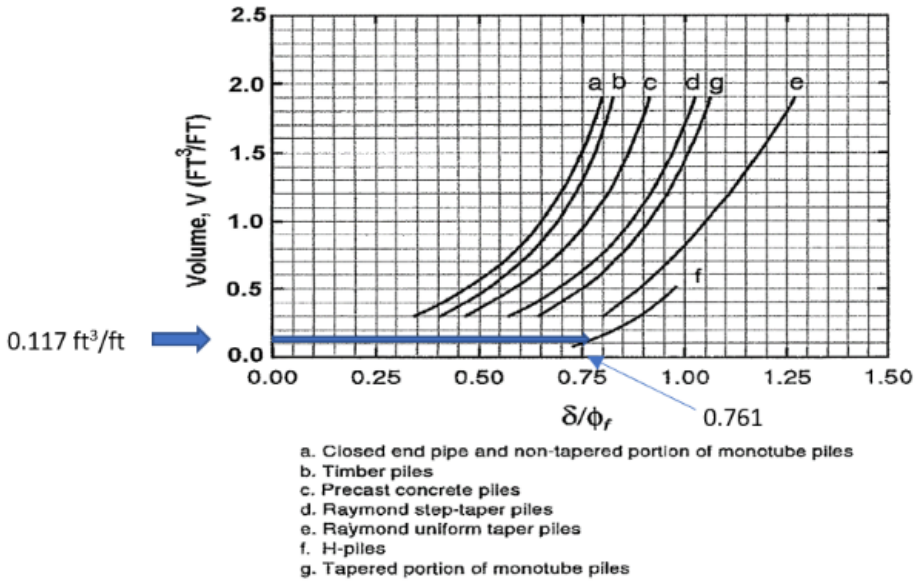


Figure 10.7.3.8.6f-6—Relation of δ/ϕ_f and Pile Displacement, *V*, for Various Types of Piles (Hannigan et al., 2006 after Nordlund, 1979)

Step 2c: Calculate δ from δ/ϕ_f ratio for each soil layer:

Soil layer #1: (0.761) * (30°) = δ = 22.8°
Soil layer #2: (0.761) * (32°) = δ = 24.4°
Soil layer #3: (0.761) * (33°) = δ = 25.1°
Soil layer #4: (0.761) * (35°) = δ = 26.6°
Soil layer #5: (0.761) * (36°) = δ = 27.4°
Soil layer #6: (0.761) * (37°) = δ = 28.2°
Soil layer #7: (0.761) * (38°) = δ = 28.9°

Step 3: Determine the coefficient of lateral earth pressure, K_δ, based on the ø angle of the soil layer.

Determine K_δ for ø angle based on displaced volume, V= 0.117 ft^3/ft, and pile taper angle, $\omega = 0°$, using AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, interpolating where necessary. (Figure 10.7.3.8.6f-2 below is an example for soil layer #1 with ø=30° using the process in note "a" below. Figure 10.7.3.8.6f-3 is an example for soil layer #4 with ø=35° using the process in note "c" below. Other soil layers were determined by using the process in note "d", below). Figures 10.7.3.8.6f-1 and 4 are presented for reference.

Soil layer #1: ø=30° K_δ = 0.87
Soil layer #2: ø=32° K_δ = 0.99
Soil layer #3: ø=33° K_δ = 1.05
Soil layer #4: ø=35° K_δ = 1.18
Soil layer #5: ø=36° K_δ = 1.30
Soil layer #6: ø=37° K_δ = 1.41
Soil layer #7: ø=38° K_δ = 1.53

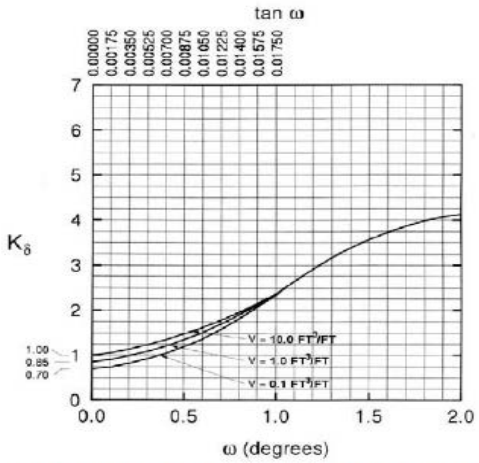


Figure 10.7.3.8.6f-1—Design Curve for Evaluating K_δ for Piles where ϕ_f = 25 degrees (Hannigan et al., 2006 after Nordlund, 1979)

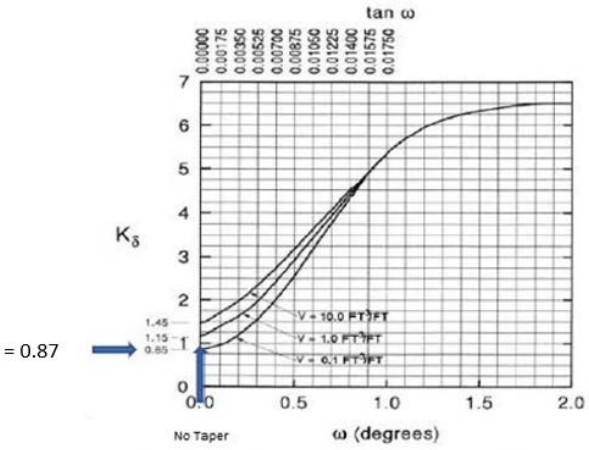


Figure 10.7.3.8.6f-2—Design Curve for Evaluating K_δ for Piles where ϕ_f = 30 degrees (Hannigan et al., 2006 after Nordlund, 1979)

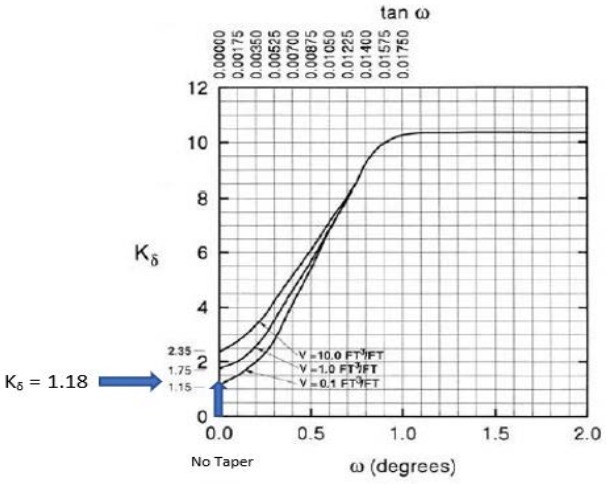


Figure 10.7.3.8.6f-3—Design Curve for Evaluating K_δ for Piles where ϕ_f = 35 degrees (Hannigan et al., 2006 after Nordlund, 1979)

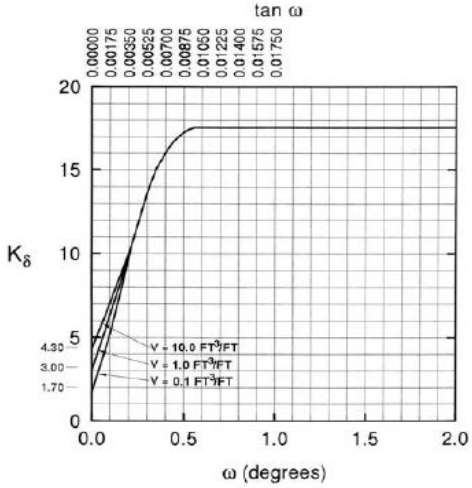


Figure 10.7.3.8.6f-4—Design Curve for Evaluating K_δ for Piles where ϕ_f = 40 degrees (Hannigan et al., 2006 after Nordlund, 1979)

Notes:

a) If the displaced volume is 0.1, 1.0 or 10.0 ft3/ft, which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle is one of those provided, Kδ can be determined directly from the appropriate figure.

b) If the displaced volume is 0.1, 1.0, or 10.0 ft3/ft which correspond to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle is different from those provided, use linear interpolation to determine Kδ for the required ø angle.

c) If the displaced volume is other than 0.1, 1.0 or 10.0 ft3/ft which corresponds to one of the curves provided in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, but the ø angle corresponds to one of those provided, use log linear interpolation to determine Kδ for the required displaced volume.

d) If the displaced volume is other than 0.1, 1.0, or 10.0 ft3/ft which correspond to one of the curves provided in in AASHTO LRFD Bridge Design Specifications Figures 10.7.3.8.6f-1 through 4, and the ø angle does not correspond to one of those provided, first use linear interpolation to determine Kδ for the required ø angle at the displaced volume curves provided for 0.1, 1.0 and 10.0 ft3/ft. Then use log linear interpolation to determine Kδ for the required displaced volume.

Step 4: Determine the correction factor, C_F, to be applied to K_δ, if $\delta \neq \phi$ angle for each soil layer using AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-5. (Example below is for ø=30°). $\delta/\phi_f = 0.761$ is from Step 2b, above.

Soil layer #1: ø=30° $\delta/\phi_f = 0.761$ therefore $C_F = 0.91$
Soil layer #2: ø=32° $\delta/\phi_f = 0.761$ therefore $C_F = 0.90$
Soil layer #3: ø=33° $\delta/\phi_f = 0.761$ therefore $C_F = 0.89$
Soil layer #4: ø=35° $\delta/\phi_f = 0.761$ therefore $C_F = 0.88$
Soil layer #5: ø=36° $\delta/\phi_f = 0.761$ therefore $C_F = 0.87$
Soil layer #6: ø=37° $\delta/\phi_f = 0.761$ therefore $C_F = 0.86$
Soil layer #7: ø=38° $\delta/\phi_f = 0.761$ therefore $C_F = 0.85$

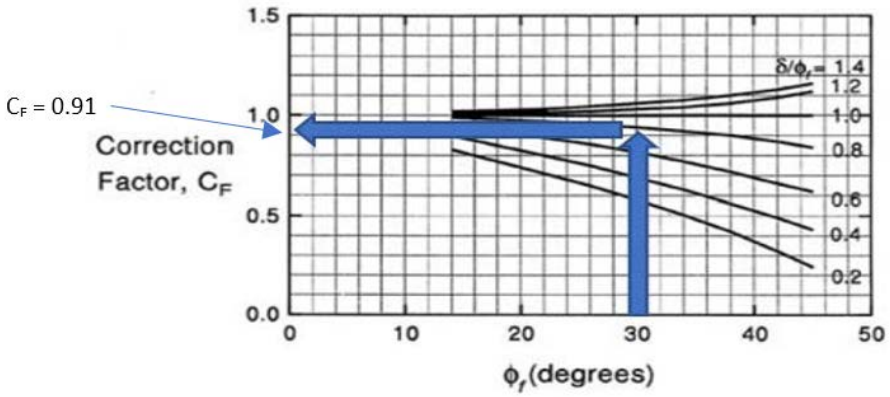


Figure 10.7.3.8.6f-5—Correction Factor for K_δ where $\delta \neq \phi_f$ (Hannigan et al., 2006 after Nordlund, 1979)

Step 5: Compute the vertical effective stress (σ'_v) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Midpoint of Layer #1: Not relevant for this example
Bottom of Layer #1: 5'(γ'=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Midpoint of Layer #2: 5'(γ'=61.6 pcf) + 288 psf = 596 psf = 0.596 ksf
Bottom of Layer #2: 5'(γ'=61.6 pcf) + 596 psf = 904 psf = 0.904 ksf

Layer #3:
Midpoint of Layer #3: 5'(γ'=63.6 pcf) + 904 psf = 1222 psf = 1.22 ksf
Bottom of Layer #3: 5'(γ'=63.6 pcf) + 1222 psf = 1540 psf = 1.54 ksf

Layer #4:
Midpoint of Layer #4: 5'(γ'=65.6 pcf) + 1540 psf = 1868 psf = 1.87 ksf
Bottom of Layer #4: 5'(γ'=65.6 pcf) + 1868 psf = 2196 psf = 2.19 ksf

Layer #5:
Midpoint of Layer #5: 5'(γ'=67.6 pcf) + 2196 psf = 2534 psf = 2.53 ksf
Bottom of Layer #5: 5'(γ'=67.6 pcf) + 2534 psf = 2872 psf = 2.87 ksf

Layer #6:
Midpoint of Layer #6: 5'(γ'=70.6 pcf) + 2872 psf = 3225 psf = 3.23 ksf
Bottom of Layer #6: 5'(γ'=70.6 pcf) + 3225 psf = 3578 psf = 3.58 ksf

Layer #7:
Midpoint of Layer #7: 2.5'(γ'=72.6 pcf) + 3578 psf = 3760 psf = 3.76 ksf
Bottom of Layer #7: 2.5'(γ'=72.6 pcf) + 3760 psf = 3941 psf = 3.94 ksf

Compute the Strength Limit State Compression Design Tip Elevation

STEP 6: Compute the pile side resistance (R_s).

Step 6a: Compute the pile side resistance for each soil layer. (For H-piles in cohesionless soils, the "box" area should generally be used to compute the surface area of the pile side).

$q_s = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})$ (AASHTO 10.7.3.8.6f-1)

$A_s = (C_d)(\Delta d)$

$R_s = (q_s)(A_s) = (K_\delta)(C_F)(\sigma'_v)(\frac{\sin(\delta+\omega)}{\cos\omega})(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2)
- K_δ = Coefficient of lateral earth pressure (from Step 3)
- C_F = Correction factor to be applied to K_δ (from Step 4)
- σ'_v = Vertical effective stress at midpoint of each relevant soil layer (from Step 5) (ksf)
- δ = Friction angle between the pile and soil (from δ in Step 2c)
- ω = Pile taper angle. (In this example no taper, so $\omega = 0^\circ$)
- C_d = Pile perimeter (using "box" H-pile area) = 3.37' for this example
- Δd = Thickness of soil layer (ft)

Layer #1: No side resistance contribution from Layer #1

Layer #2:(K_δ=0.99)(C_F=0.90)(σ'_v=0.596 ksf) ($\frac{\sin(24.4^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=10.0') = 7.3 kips

Layer #3:(K_δ=1.05)(C_F=0.89)(σ'_v=1.22 ksf) ($\frac{\sin(25.1^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=10.0') = 16.1 kips

Layer #4:(K_δ=1.18)(C_F=0.88)(σ'_v=1.87 ksf) ($\frac{\sin(26.6^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=10.0') = 29.4 kips

Layer #5:(K_δ=1.30)(C_F=0.87)(σ'_v=2.53 ksf) ($\frac{\sin(27.4^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=10.0') = 44.3 kips

Layer #6:(K_δ=1.41)(C_F=0.86)(σ'_v=3.23 ksf) ($\frac{\sin(28.2^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=10.0') = 62.0 kips

Layer #7:(K_δ=1.53)(C_F=0.85)(σ'_v=3.76 ksf) ($\frac{\sin(28.9^\circ+0^\circ)}{\cos 0^\circ}$) (C_d=3.37')(Δd=5.0') = 39.5 kips

Step 6b: After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the total pile side resistance (R_s).

ΣR_s = 199 kips

Step 7: Determine the pile tip resistance (Rp):

$q_p = (\alpha_t)(N'_q)(\sigma'_v)$

$R_p = (q_p)(A_p) = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

Where:

- q_p = Nominal unit tip resistance (ksf)
- α_t = Coefficient from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7
- N'_q = Bearing capacity factor from AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8
- σ'_v = Vertical effective stress at the bottom of Layer #7 (at pile tip) (ksf)
- A_p = Area of the H-pile steel section = 16.8 in^2 = 0.117 ft^2

Step 7a: Determine the α_t coefficient by entering the ø angle of soil at the pile tip in Layer #7 into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-7 intersecting the D/b ratio line. For this example, ø=38°; D/b ratio is: Pile Length/Pile Width = 55'/0.85' = 64.71

Therefore, **α_t = 0.72**

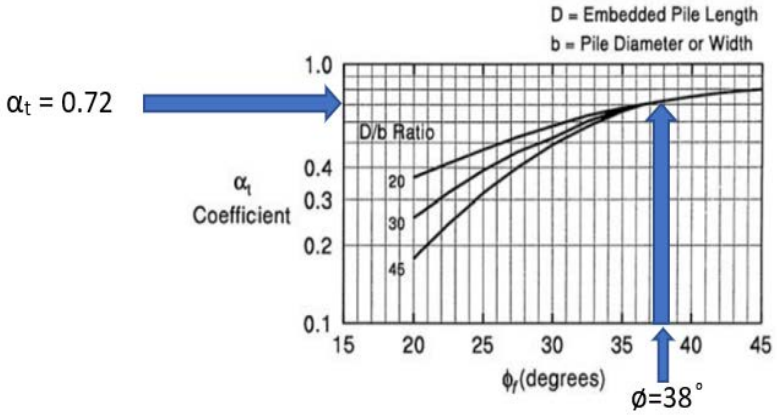


Figure 10.7.3.8.6f-7—α_t Coefficient (Hannigan et al., 2006 modified after Bowles, 1977)

Step 7b: Determine the bearing capacity factor, N'_q , by entering ø angle of soil at the pile tip in Layer #7 into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-8. For this example, ø=38°.

Therefore, **N'_q = 105**

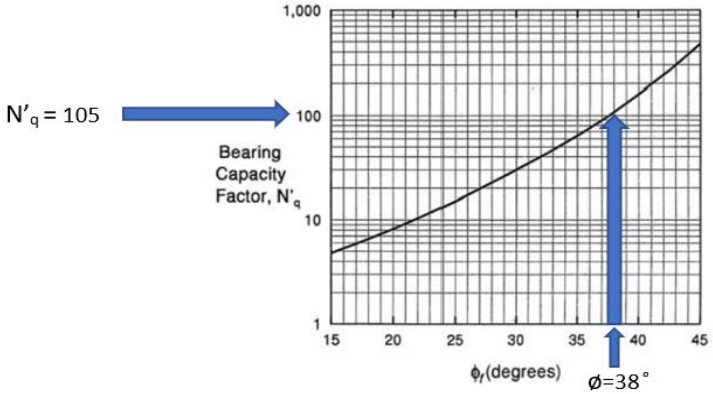


Figure 10.7.3.8.6f-8—Bearing Capacity Factor, N'_q
(Hannigan et al., 2006 modified after Bowles, 1977)

Step 7c: Determine the vertical effective stress (σ'_v) at the pile tip.

The vertical effective stress (σ'_v) at the bottom of Layer #7 = 3.94 ksf. However, AASHTO LRFD Bridge Design Specifications section 10.7.3.8.6f states it must be limited to ≤ 3.2 ksf, therefore:

σ'_v = 3.2 ksf.

Step 8: Compute the pile tip resistance (R_p).

$R_p = (\alpha_t)(N'_q)(\sigma'_v)(A_p)$

R_p = (0.72)(105)(3.2 ksf)(0.117 ft^2) = **28 kips**

Step 8a: Compare the pile tip resistance (R_p) to the limiting unit tip resistance (q_L)

The calculated pile tip resistance must be compared to the limiting unit tip resistance (q_L). The limiting q_L value is obtained from entering the ø angle of soil near the pile tip into AASHTO LRFD Bridge Design Specifications Figure 10.7.3.8.6f-9. For this example, ø=38°, therefore **q_L = 260 ksf**.

$R_{p\ limited} = (q_L)(A_p)$

$R_{p\ limited}$ = (260 ksf) (0.117 ft^2) = **30.4 kips**

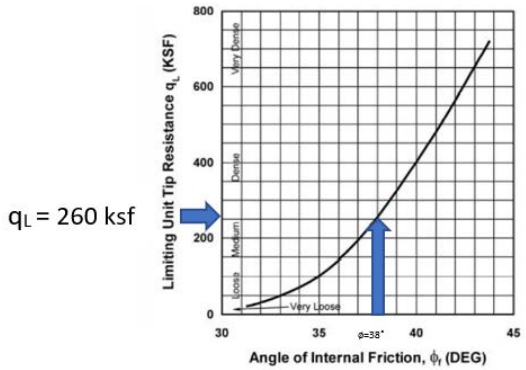


Figure 10.7.3.8.6f-9—Limiting Unit Pile Tip Resistance (Hannigan et al., 2006 after Meyerhof, 1976)

Step 8b: Use the lesser of either the R_p or $R_{p\ limited}$ value: Therefore, use **R_p = 28 kips**

Step 9: Compute the Nominal Pile Resistance (R_n)

R_n = R_s + R_p = 199 kips + 28 kips = **227 kips** at design tip elevation 35.0 feet.

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inch)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (ft)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp (φ_{qs}=1.0) (φ_{qp}=1.0)	Tension (φ_{qs}=1.0)			
Pier 2	HP 10X57 H-Pile	90.0	760	550	1	220	0	120	0	35.0 (a-I)	35.0	227

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Compression	Tension			
Pier 2	HP 10X57 H-Pile	220	0	35.0 (a)	35.0	227

Note: Design tip elevations are controlled by (a) Compression

Appendix C: 2000 American Petroleum Institute (API) Method - Driven CISS Pile

This design example is for Strength Limit State only. A typical pile design includes evaluation of Service Limit State, Strength Limit State, and Extreme Event Limit State. The Extreme Event Limit State analysis is the same as this example, except that it would be designed for the Extreme Event Limit State required nominal resistance and scour percentages.

For pile foundations in cohesive soils or layers and loose granular soils, pile group settlement must be evaluated by using the equivalent footing method for Service Limit State design. Figures that show soil layers and stress distribution for various cases can be found in AASHTO LRFD Bridge Design Specifications Section 10.7.2.3.

In conditions where the piles are driven into very dense layers or rock, and settlement is anticipated to be minimal, Service Limit State calculation may not be needed. However, If the pile is driven into a very dense layer that is underlain by a loose granular or cohesive layer, then settlement should be evaluated.

Lateral design of piles is usually performed by the structure designer, however the Geoprofessional typically provides the structure designer the soil parameters for the L-Pile analysis in Class S2 soils per Seismic Design Criteria 6.1.2.

Information provided by Structure Design

- 24-inch diameter CISS pile is required.
- The structure designer determines the minimum length of the rebar cage. For this example, the structure designer needs 25 feet of concrete and reinforcement from the pile cutoff.
- A ½-inch thick steel section is required.
- Foundation information and loads will be provided from the Structure Designer, as shown in the tables below.

Foundation Design Data Sheet Provided by Structure Designer

Support No.	Pile Type	Finished Grade Elevation (feet)	Pile Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement Under Service Load (inch)	Number of Piles per Support
				B	L		
Pier 2	24" diam. CISS Piles	100.0	90.0	N/A	N/A	1	5

Foundation Factored Design Loads Provided by Structure Designer

Support No.	Service I Limit State (kips)		Strength/Construction Limit State (Controlling Group) (kips)				Extreme Event Limit State (Controlling Group) (kips)			
			Compression		Tension		Compression		Tension	
	Total Load Per Support	Permanent Loads Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Pier 2	1154	730	1811	392	0	0	750	187	0	0

Geotechnical Design Considerations

- The Geoprofessional will take the factored load and apply the resistance factor (0.7 for both side and tip resistance for Strength Limit State, and 1.0 for both side and tip resistance for Extreme Event Limit State) and then round up to the nearest 10 kips. For this example, the Required Nominal Resistance for the Strength Limit State and Extreme Event Limit State are 560 kips and 190 kips, respectively. The Foundation Design Recommendations Table and the Pile Data Table examples are presented at the end of this example.
- If it is anticipated that the contractor will request to use the vibratory method to "seat" the pile prior to impact driving, the design should ignore side resistance throughout that zone of vibratory driving. The depth of "seating" will depend on the pile size and length, and is typically limited to a maximum length of 20% of the pile length or 20 feet. Usage of the vibratory method and "seating" depth used in the pile design should be disclosed in the "Notes for Construction" section of the Foundation Report.
- Due to the CISS pile tip below the groundwater table, a seal course will be required to place the concrete and reinforcement in a dry condition.
- A drivability analysis by Foundation Testing and Instrumentation will determine what minimum thickness of shell is needed based on the subsurface conditions at the site. The structure designer must also be consulted as to what minimum thickness of shell is required structurally for their design. The shell thickness that satisfies both Geotechnical Services and Structure Design requirements must be included in the foundation report.
- Scour information for this example is presented below:

Scour Data Table

Support No.	Long-Term (Degradation and Contraction) Scour Elevation (feet)	Short-Term (Local) Scour Depth (feet)
Pier 2	95.0	20.0

- AASHTO LRFD Bridge Design Specifications – California Amendments, Table 3.7.5-1, provides the percentages of Long-Term (Degradation + Contraction) and Short-Term (Local) scour to be used for each limit state.

Table 3.7.5-1—Scour Conditions for Limit State Load Combinations

Limit State		Degradation/ Aggradation	Contraction Scour	Local Scour
Strength	minimum	0%	0%	0%
	maximum	100%	100%	50%
Service	minimum	0%	0%	0%
	maximum	100%	100%	100%
Extreme Event I	minimum	0%	0%	0%
	maximum	100%	100%	0%

- When calculating nominal resistance:
  1. The vertical effective stress is calculated using 100% of Long-Term (Degradation + Contraction) scour and 0% Short-Term (Local) scour for Strength Limit and Extreme Event Limit states.
  2. Side resistance pile-soil contact is calculated using 100% Long-Term and 50% Short-Term (Local) scour for Strength Limit State, and 100% Long-Term and 0% Short-Term scour for Extreme Event Limit State. See the table below for information.

Effect of Scour on Pile Design				
	Vertical Effective Stress		Side Resistance	
	Long-Term	Short-Term	Long-Term	Short-Term
Service	0%	0%	N/A	N/A
Strength	100%	0%	100%	50%
Extreme	100%	0%	100%	0%

- For this example, the following elevations are used to calculate the Strength Limit State pile design tip elevation:
  1) Elev. 85’ is the top of side resistance zone.
  2) Elev. 95’ is the ground surface for vertical effective stress calculation for both side and tip resistance.
- This example does not calculate the Extreme Event Limit State pile design tip elevation. However, if it did, the following would be used:
  1) Elev. 90’ would be the top of side resistance zone.
  2) Elev. 95’ would be the ground surface for vertical effective stress calculation for both side and tip resistance.

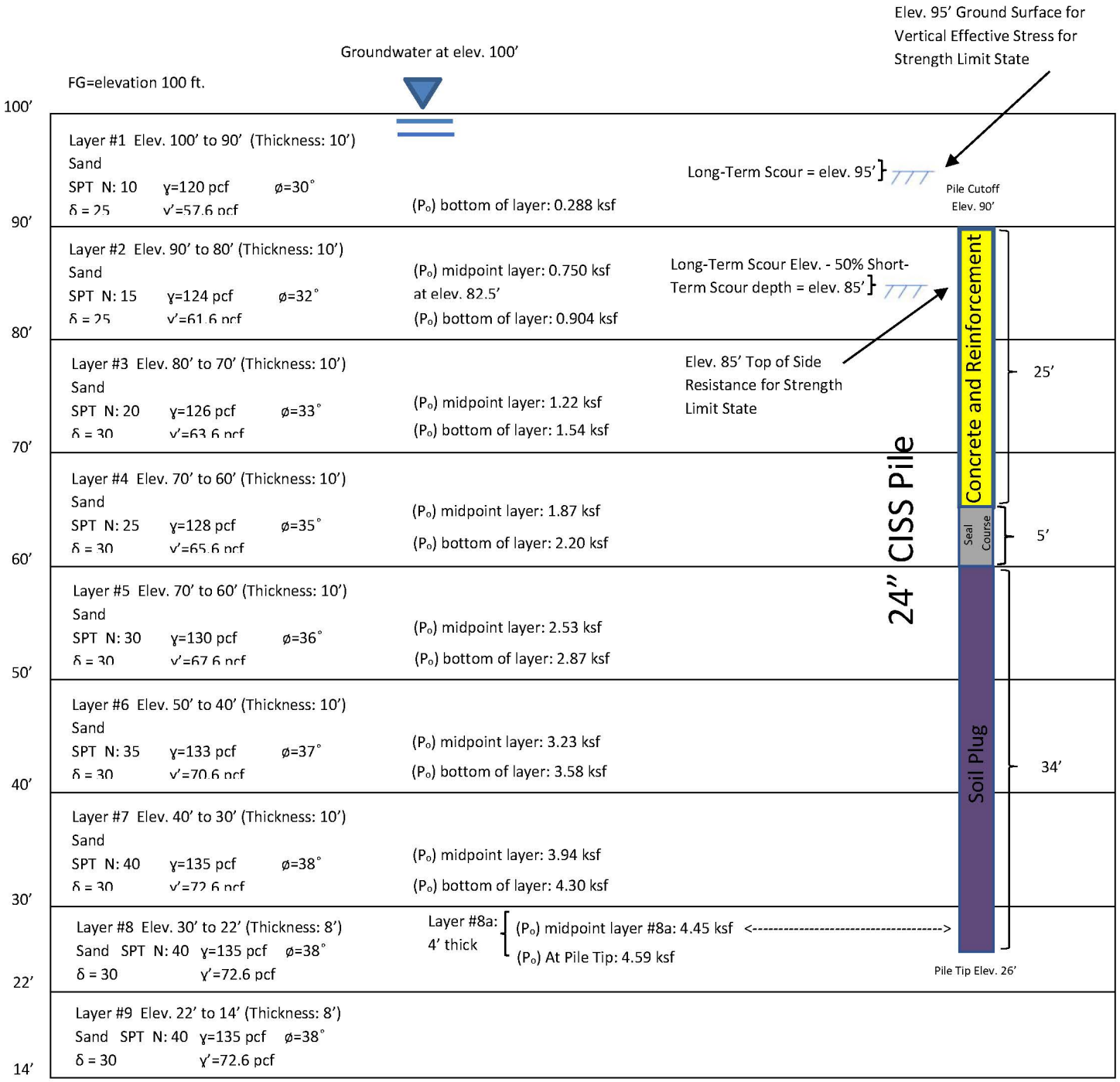
- Table 6.4.3-1 in the API Method is used for determining the Soil-Pile Friction Angle (δ), the limiting skin friction values, Nq, and the limiting unit end bearing values in the design example.

Table 6.4.3-1—Design Parameters for Cohesionless Siliceous Soil*

Density	Soil Description	Soil-Pile Friction Angle, δ Degrees	Limiting Skin Friction Values kips/ft² (kPa)	N_q	Limiting Unit End Bearing Values kips/ft² (MPa)
Very Loose Loose Medium	Sand Sand-Silt** Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt** Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt**	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt**	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

*The parameters listed in this table are intended as guidelines only. Where detailed information such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.
**Sand-Silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

Soil Profile:



California Amendments to AASHTO LRFD Bridge Design Specifications, Section C10.7.3.8.6a, states that the 2000 American Petroleum Institute method should be used to determine the nominal pile resistance (R_n) for steel pipe piles and CISS piles over 18 inches in diameter. The following steps should be used to calculate the nominal pile resistance of an open-ended pile, which is the sum of the outside pile side resistance ($R_{s\ outside}$), the steel shell end area resistance ($R_{p\ steel}$), and the lesser of either the soil plug end area resistance ($R_{p\ soil}$) or internal soil plug side resistance ($R_{s\ inside}$).

Step 1: Divide the soils into layers and determine the ɣ and ø angle for each layer. (See Soil Profile above).

Step 2: Compute the vertical effective stress (P_o) at the midpoint and bottom of each relevant soil layer. (See Soil Profile above).

Layer #1:
Midpoint of Layer #1: Not relevant for this example
Bottom of Layer #1: 5'(ɣ'=57.6 pcf) = 288 psf = 0.288 ksf

Layer #2:
Midpoint of Layer #2: 5'(ɣ'=61.6 pcf) + 288 psf = 596 psf = 0.596 ksf
Bottom of Layer #2: 5'(61.6 pcf) + 596 psf = 904 psf = 0.904 ksf

Layer #3:
Midpoint of Layer #3: 5'(ɣ'=63.6 pcf) + 904 psf= 1222 psf = 1.22 ksf
Bottom of Layer #3: 5'(63.6 pcf) + 1222 psf = 1540 psf = 1.54 ksf

Layer #4:
Midpoint of Layer #4: 5'(ɣ'=65.6 pcf) + 1540 psf = 1868 psf = 1.87 ksf
Bottom of Layer #4: 5'(ɣ'=65.6 pcf) + 1868 psf = 2196 psf = 2.20 ksf

Layer #5:
Midpoint of Layer #5: 5'(ɣ'=67.6 pcf) + 2196 = 2534 psf = 2.53 ksf
Bottom of Layer #5: 5'(ɣ'=67.6 pcf) + 2534 psf = 2872 psf = 2.87 ksf

Layer #6:
Midpoint of Layer #6: 5'(ɣ'=70.6 pcf) + 2872 psf = 3225 psf = 3.23 ksf
Bottom of Layer #6: 5'(ɣ'=70.6 pcf) + 3225 psf = 3578 psf = 3.58 ksf

Layer #7:
Midpoint of Layer #7: 5'(ɣ'=72.6 pcf) + 3578 psf =3941 psf = 3.94 ksf
Bottom of Layer #7: 5'(ɣ'=72.6 pcf) + 3941 psf = 4304 psf = 4.30 ksf

Layer #8a:
Midpoint of Layer #8a : 2'(ɣ'=72.6 pcf) + 4304 psf = 4449 psf = 4.45 ksf
Bottom of Layer #8a: 2'(ɣ'=72.6 pcf) + 4449 psf = 4594 psf = 4.59 ksf

Compute the Strength Limit State Compression Design Tip Elevation

Step 3: Compute the outside pile side resistance ($R_{s\ outside}$) in each soil layer.

$q_s = (K)(P_o)(\tan\delta)$

$A_s = (C_d)(\Delta d)$

$R_{s\ outside} = (q_s)(A_s) = (K)(P_o)(\tan\delta)(C_d)(\Delta d)$

Where:

- q_s = Nominal unit side resistance (ksf)
- A_s = Surface area of pile side (ft^2).
- K = Coefficient of lateral earth pressure. The API method states it is usually appropriate to assume K= 0.8 for both compression and tension loadings for piles driven unplugged. Values of K for full displacement piles (plugged or closed end) may be assumed to be 1.0. The API method states that a pile may be considered plugged or unplugged based on static calculations. It also states that a pile could be driven in an unplugged condition, but act plugged under static loading. Due to the uncertainty of where plugging may occur, a value of K=0.8 was conservatively selected for this example.
- P_o = Vertical effective stress at midpoint of each relevant soil layer (from Step 2).
- δ = Friction angle between steel and soil (from Table 6.4.3-1 API Method). For this example, a δ of 25 was selected for Layer #2, and a δ of 30 was selected for Layers #3 through #8.
- C_d = Outside Pile circumference for 24” dia. CISS pile: (π)(24”) = 75.4” = 6.28 ft. for this example.
- Δd = Thickness of soil layer (ft)

Layer #1: (No side resistance contribution from Layer #1 due to cut-off)

Layer #2: (K=0.8)(P_o=0.59 ksf)(tanδ=tan25=0.466) (C_d=6.28’)(Δd=5’) = 6.9 kips

Layer #3: (K=0.8)(P_o=1.22 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 35.4 kips

Layer #4: (K=0.8)(P_o=1.87 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 54.2 kips

Layer #5: (K=0.8)(P_o=2.53 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 73.3 kips

Layer #6: (K=0.8)(P_o=3.23 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 93.6 kips

Layer #7: (K=0.8)(P_o=3.94 ksf)(tanδ=tan30=0.577) (C_d=6.28’)(Δd=10’) = 114.2 kips

Layer #8: (2.0 ksf) (C_d=6.28’)(Δd=4’) = 50.2 kips

For Layer #8, $q_s = (K)(P_o)(\tan\delta)$ = 2.05 ksf. However, Table 6.4.3-1 in the API method limits the unit skin friction value to 2.0 ksf for dense sand and very dense sand-silt. Therefore, for Layer #8 q_s is limited at 2.0 ksf.

After computing the side resistance in each soil layer, sum the side resistances from each soil layer to obtain the outside pile side resistance.

$\Sigma R_{s\ outside}$ = 428 kips

Pile Tip Resistance (R_p)

The pile tip resistance has two components: the steel shell end area resistance ($R_{p\ steel}$), and the lesser of either the soil plug end area resistance ($R_{p\ soil}$) or the internal soil plug side resistance ($R_{s\ inside}$). The sum of these two components is the total pile tip resistance of the pile.

- $R_p = (R_{p\ steel}) + [\text{Lesser of } (R_{p\ soil}) \text{ or } (R_{s\ inside})]$.

Steps 4 and 5, below, provide examples for calculating the steel shell end area resistance and the soil plug end area resistance, respectively. Step 6 provides an example for calculating the internal soil plug side resistance ($R_{s\ inside}$). Step 7 compares the value of the soil plug end area resistance ($R_{p\ soil}$) to the value of the internal soil plug side resistance ($R_{s\ inside}$) and the lesser of the two values is used.

Step 4: Calculate the steel shell end area resistance.

Where:
- 24” CISS pile outside diameter
- 23” CISS pile inside diameter
- ½” thick steel shell

Step 4a: Calculate the steel shell end area:

Area$_1$ (A_1) : Using 24” CISS outside diameter: $\pi D^2/4 = 3.14(24")^2/4 = 452.16\ in^2 = 3.14\ ft^2$

Area$_2$ (A_2): Using 23” CISS inside diameter: $\pi D^2/4 = 3.14(23")^2/4 = 415.27\ in^2 = 2.89\ ft^2$

Steel shell end area: $\mathbf{A_{p\ steel}} = A_1 - A_2 = 3.14\ ft^2 - 2.89\ ft^2 = \mathbf{0.25\ ft^2}$

Step 4b: Calculate the steel shell end area resistance: $R_{p\ steel} = (q_{p\ steel})(A_{p\ steel})$

$q_{p\ steel} = (P_o)(Nq)$

Where:
- P_o = Vertical effective stress at bottom of Layer #8a = 4.59 ksf
- Nq = Dimensionless bearing capacity factor (from Table 6.4.3-1 API Method)= 40

$q_{p\ steel}$ = (4.59 ksf) (40) = 183.6 ksf
$A_{p\ steel} = 0.25\ ft^2$

$R_{p\ steel}$ = $(q_{p\ steel})(A_{p\ steel})$ = (183.6 ksf) (0.25 ft^2) = **46 kips**

Internal Soil Plug

For steel pipe piles that are installed open ended, during pile driving the soil enters the pile and as pile penetration increases the pile fills with soil. If the interior soil column does not equal the pile penetration, then this is called the plugging effect and the pile may behave during driving as if it was closed-ended where soil does not move into the pipe/shell.

The API method provides minimal information on plugging and unplugging conditions. The API method states it is usually appropriate to assume K=0.8 for both compression and tension loadings for piles driven unplugged. Values of K for full displacement piles (plugged or closed end) may be assumed to be 1.0. The API method states that a pile may be considered plugged or unplugged based on static calculations. It also states that a pile could be driven in an unplugged condition, but act plugged under static loading. Due to the uncertainty of where plugging may occur, a value of K=0.8 was conservatively selected for this example.

The API method compares the soil plug end area resistance and the internal soil plug side resistance. It states that the soil plug end area resistance should not exceed the internal soil plug side resistance, and the lesser of the two values should be used.

Studies have shown that the calculated internal soil plug may be overestimated, so consideration should be given to reducing the side resistance of the internal soil plug. If reduction is utilized, it is recommended that studies with similar site conditions and design methodologies be utilized when adopting such reductions.

Step 5: Calculate the seal course thickness:

For this example, the structure designer needs 25 feet of concrete and reinforcement from the pile cutoff. The Geoprofessional must make an assumed seal course thickness (typically 3-5 feet). For this example, an assumed value of 5 feet is used. A hydrostatic head of 40 feet is used in the calculation for this example, which is from the groundwater surface down to the bottom of the assumed seal course thickness (elev. 100 ft to elev. 60 ft). The bond strength (Q_s) was assumed to be 300 psf for the seal course concrete placed below groundwater.

The seal course thickness (t) is calculated by the following equation: $$t = \frac{\gamma_w Z r}{\gamma_c r + 2Q_s}$$

Where:

- γ_w = Unit weight of water: 62.4 pcf
- Z = Hydrostatic head from the groundwater surface to bottom of seal course: 40 ft this example
- r = Inside pile radius. For this example: r = (12” – ½”)/12 = 0.96 ft.
- γ_c = Unit weight of concrete. For this example: 145 pcf
- Q_s = Bond strength of steel to concrete: 300 psf

Seal Course Thickness: $$t = \frac{(62.4\text{ pcf})(40')(0.96')}{(145\text{ pcf})(0.96') + 2(300\text{ psf})} = 3.2\ feet$$

Due to variable conditions at the site (variations in groundwater, soil type, etc.), it was decided to round up the calculated seal course thickness to 5 feet in this example.

Therefore, the seal course thickness t = 5.0 feet

- The seal course thickness must be reported in the foundation report.

Step 6: Calculate the internal soil plug side resistance ($R_{s\ inside}$).

The soil plug length was determined by taking the required length of the rebar cage (25 feet in this example), then adding the seal course thickness (5 feet in this example), then subtracting that length from the total length of the pile (64 feet). 64 ft – 30 ft = 34 feet. For this example, the vertical effective stress (P_o) for the internal soil plug is calculated from the bottom of the seal course (elev. 60') to the midpoint of Layer #8a (elev. 28').

$q_{s\ inside} = (K)(P_o)(\tan\delta)$

$A_{s\ inside} = (C_d)(\Delta d)$

$R_{s\ inside} = (q_{s\ inside})(A_{s\ inside}) = (K)(P_o)(\tan\delta)(C_d)(\Delta d)$

Where:

- $q_{s\ inside}$ = Internal soil plug nominal unit side resistance
- $A_{s\ inside}$ = Surface area of the inside of the pile (ft^2).
- K = Coefficient of lateral earth pressure. Use K=0.8 as the pile is assumed to drive unplugged.
- P_o = Vertical effective stress from bottom of seal course to midpoint of soil layer #8a = 2.25 ksf.
- δ = Friction angle between steel and soil (from Table 6.4.3-1 API Method). δ=30 degrees for this example.
- C_d = Inside Pile circumference for 23" dia.: (π)(23") = 72.2" = 6.02 ft. for this example.
- Δd = Thickness of internal soil plug. 34 feet for this example.

Step 6a: Calculate Side Resistance for Internal Soil Plug in Layers #5 through #8a (Elev. 60' to 26')

Internal Soil Plug Side Resistance for Layer #5 (Elev. 60' – 50'):

$P_{o\ Midpoint\ Layer\ \#5}$ = (5')(67.6 pcf)= 338 psf = 0.338 ksf

$P_{o\ Bottom\ Layer\ \#5}$ = (5')(67.6 pcf) + 338 psf = 676 psf = 0.676 ksf

$q_{s\ Inside\ Layer\ \#5}$ = (K=0.8) (P_o= 0.338 ksf) (tanδ=tan30=0.577) = 0.156 ksf

$A_{s\ Inside\ Layer\ \#5}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#5}$ = (6.02 ft)(10 ft) = 60.2 ft^2

$R_{s\ Inside\ Layer\ \#5}$ = $(q_{s\ inside})(A_{s\ inside})$ = (0.156 ksf) (60.2 ft^2) = **9.4 kips**

Internal Soil Plug Side Resistance for Layer #6 (Elev. 50' – 40'):

$P_{o\ Midpoint\ Layer\ \#6}$ = (5')(70.6 pcf) + 676 psf = 1029 psf = 1.03 ksf

$P_{o\ Bottom\ Layer\ \#6}$ = (5')(70.6 pcf) + 1029 psf = 1382 psf = 1.38 ksf

$q_{s\ Inside\ Layer\ \#6}$ = (K=0.8) (P_o=1.03 ksf) (tanδ=tan30=0.577) = 0.48 ksf

$A_{s\ Inside\ Layer\ \#6}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#6}$ = (6.02 ft)(10 ft) = 60.2 ft^2

$R_{s\ Inside\ Layer\ \#6}$ = ($q_{s\ inside}$)($A_{s\ inside}$) = (0.48 ksf) (60.2 ft^2) = **28.9 kips**

Internal Soil Plug Side Resistance for Layer #7 (Elev. 40' – 30'):

$P_{o\ Midpoint\ Layer\ \#7}$ = (5')(72.6 pcf) + 1382 psf = 1745 psf = 1.74 ksf

$P_{o\ Bottom\ Layer\ \#7}$ = (5')(72.6 pcf) + 1745 psf = 2108 psf = 2.11 ksf

$q_{s\ Inside\ Layer\ \#7}$ = (K=0.8) (P_o=1.74 ksf) (tanδ=tan30=0.577) = 0.80 ksf

$A_{s\ Inside\ Layer\ \#7}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#7}$ = (6.02 ft)(10 ft) = 60.2 ft^2

$R_{s\ Inside\ Layer\ \#7}$ = ($q_{s\ inside}$)($A_{s\ inside}$) = (0.80 ksf) (60.2 ft^2) = **48.2 kips**

Internal Soil Plug Side Resistance for Layer #8a (Elev. 30' – 26'):

$P_{o\ Midpoint\ Layer\ \#8a}$ = (2')(72.6 pcf) + 2108 psf = 2253 psf = 2.25 ksf

$P_{o\ Bottom\ Layer\ \#8a}$ = (2')(72.6 pcf) + 2253 psf = 2398 psf = 2.40 ksf

$q_{s\ Inside\ Layer\ \#8a}$ = (K=0.8) (P_o=2.25 ksf) (tanδ=tan30=0.577) = 1.04 ksf

$A_{s\ Inside\ Layer\ \#8a}$ = Surface area of the pile side (ft^2): $A_{s\ Inside\ Layer\ \#8a}$= (6.02 ft)(4 ft) = 24.1 ft^2

$R_{s\ Inside\ Layer\ \#8a}$ = ($q_{s\ inside}$)($A_{s\ inside}$) = (1.04 ksf) (24.1 ft^2) = **25.0 kips**

$R_{s\ Inside\ Soil\ Plug}$ = 9.4 kips + 28.9 kips + 48.2 kips + 25.0 kips = **112 kips**

Step 7: Calculate the internal soil plug end area resistance: $R_{p\ soil}$ = ($q_{p\ soil}$)($A_{p\ soil}$)

$q_{p\ soil}$ = (P_o)(Nq)

Where:

- P_o = Vertical effective stress at the bottom of Layer #8a = 4.59 ksf
- Nq = Dimensionless bearing capacity factor (from Table 6.4.3-1 API Method)= 40

$q_{p\ soil}$ = (4.59 ksf) (40) = 183.6 ksf

(Note: Table 6.4.3-1 in the API method limits the unit end bearing value to 200 ksf for dense sand and very dense sand-silt).

$A_{p\ soil}$ = 23" circular end area (using inside diameter of pile): $\pi D^2/4$ = 3.14(23^2)/4 = 415.27 in^2 = 2.89 ft^2

$R_{p\ soil}$ = ($q_{p\ soil}$)($A_{p\ soil}$) = (183.6 kips/ft^2) (2.89 ft^2) = **530 kips**

Step 8: Determining the lesser of either ($R_{s\ inside}$) or ($R_{p\ soil}$)

Based on the 2000 API method, the soil plug end area resistance ($R_{p\ soil}$) should not exceed the internal soil plug side resistance ($R_{s\ inside}$). Therefore, the two values are compared, and the lesser of the two values is used.

Internal soil plug side resistance for 34 feet long soil plug: $R_{s\ inside}$ = 112 kips

Soil plug end area resistance = $R_{p\ soil}$ = 530 kips

Therefore, using the lesser of the two values: **112 kips ($R_{s\ inside}$)**

Step 9: Compute the Nominal Pile Resistance (R_n):

R_n = $R_{s\ outside}$ + $R_{p\ steel}$ + $R_{s\ inside}$ = 428 kips + 46 kips + 112 kips = **586 kips** at tip elevation 26.0 feet.

Step 10: Compute the Nominal Driving Resistance:

The Nominal Driving Resistance includes driving through the first five feet of Layer #2, from elevation 90' to 85', plus the calculated resistance from elevation 85' to the pile tip at elevation 26'. For this example, it was assumed that no center relief drilling was performed during driving.

Side resistance from 90' to 85': (K=0.8)(P_o=0.44 ksf)(tanδ=tan25=0.466) (C_d=6.28')(Δd=5') = 5.2 kips

Total resistance from 85' to 26' = 586 kips. Nominal Driving Resistance: 586 + 5.2 = **591.2 kips (round to 591 kips).**

Once the pile design tip elevation is calculated, it must be presented in both the Foundation Design Recommendations Table and the Pile Data Table in the Foundation Report. Example tables are shown below.

Foundation Design Recommendations

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Required Nominal Resistance (kips)				Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
						Strength Limit		Extreme Event				
			Total	Permanent		Comp. (φ_{qs}=0.7) (φ_{qp}=0.7)	Tension (φ_{qs}=0.7)	Comp. (φ_{qs}=1.0) (φ_{qp}=1.0)	Tension (φ_{qs}=1.0)			
Pier 2	24" diam. CISS Pile	90.0	1160	730	1	560	0	190	0	26.0 (a-I)	26.0	591

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit)

Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elevation (feet)	Specified Tip Elevation (feet)	Required Nominal Driving Resistance (kips)
		Nominal Resistance (kips), Compression: Compression	Nominal Resistance (kips), Tension: Tension			
Pier 2	24” diam. CISS Pile	560	0	26.0 (a)	26.0	591

Note: Design tip elevations are controlled by (a) Compression

Cast-In-Drilled-Hole (CIDH) Pile Foundations

This document presents the design methods and communication steps between Structure Design (SD) and Geotechnical Services (GS) for the load and resistance factor design (LRFD) of Cast-In-Drilled-Hole (CIDH) pile foundations used for support of bridges, retaining walls, non-standard walls, signs, and other structures. Also included are the procedures for documenting design information that might be needed to evaluate the impact of an anomaly (i.e., construction defect), and related analytical procedures. The Appendices include three design calculation examples.

Standards relating to CIDH pile foundation investigations, design, and reporting are:

- Caltrans Seismic Design Criteria (SDC)
- AASHTO LRFD Bridge Design Specifications with CA Amendments (AASHTO)
- Drilled Shafts: Construction Procedures and Design Methods (O'Neill & Reese, 1999)
- Caltrans Standard Specifications, Standard Plans, Bridge Standard Detail Sheets (XS Sheets)
- Bridge Memos to Designers (MTD) 3-1, *Deep Foundations*
- Bridge Design Aids
- Bridge Construction Records and Procedures Manual, Volume II
- Caltrans Geotechnical Manual
    - *Foundation Reports for Bridges*
    - *Geotechnical Investigations*

Geotechnical Service's role in CIDH pile foundation design is to provide the Structure Designer with a Foundation Report addressing the following:

- Design Tip Elevations for piles for Service, Strength and Extreme Event Limit State.
- The Controlling Design Tip Elevation.
- The Steel Casing Specified Tip Elevation (if applicable).
- Recommendations relating to specifications and construction.

The Structure Designer's role in CIDH pile design includes:

- Providing the project schedule including due dates for reports.
- Providing the foundation design data and factored design load information.
- Providing the latest plan sheets pertinent to foundation design (e.g. General Plan, Foundation Plan, Foundation Detail Sheets, etc).

Terminology

1. Cast-In-Drilled Hole (CIDH) concrete piles: CIDH concrete piles, also known as drilled shafts, can be used as smaller-diameter piles that are connected to a pile cap supporting a column or as a larger pile (typically 5 feet or larger) that directly supports a column and is either a Type I or Type II shaft (as determined by the Structure Designer). Standard Plan CIDH concrete piles are either 16 or 24 inches in diameter, whereas special design CIDH concrete piles range from 30 inches and greater. Piles placed in wet conditions must be at least 24 inches in diameter to accommodate inspection pipes for acceptance testing. CIDH concrete pile lengths should be limited to 30 times the pile diameter to help ensure constructability and quality.
    i. Type I Shaft: the reinforcement consists of one continuous cage that extends from the pile tip to the bent cap.
    ii. Type II Shaft: the reinforcement consists of one cage that extends from the pile tip to the pile cut-off elevation. The column cage is a smaller-diameter cage that extends into the CIDH concrete pile reinforcement cage to form a lap splice. For a 5-foot diameter or larger Type II shaft, a construction joint is mandatory at the bottom of the column rebar cage elevation. The construction joint requires the placement of a permanent steel casing/shell in the hole to allow workers to clean and prepare the joint.
2. Rock Socket: a pay item for the length of a CIDH concrete pile that is constructed in rock that requires a core barrel, cluster hammer, or other hard rock tool for excavation. The rock is usually stronger than concrete, and typically the side resistance is controlled by the compressive strength of concrete, not the rock strength.
3. Driven Steel Shell: Used for structural capacity and/or geotechnical resistance. Must be installed with an impact hammer.
4. Permanent Casing can be either:
    i. Permanent Smooth-wall Steel *Casing:* Used for constructability. May be used for structural capacity. Not used for geotechnical resistance. Unacceptable methods of installation must be listed in the Foundation Report, Notes for Specifications.
    ii. Corrugated Metal Pipe (CMP) *Casing:* Used for constructability. May be used for limited geotechnical resistance. Not used for structural capacity. A CMP is placed in an excavated hole and the annular space backfilled with grout (SS 49-3.02B(5), 49-3.02C(6)). If the conditions are dry, the geotechnical resistance of the CMP can be utilized in the pile design, however limit side resistance to the uppermost 20 feet.

Investigations

The goal of the geotechnical investigation for a CIDH pile foundation is to determine the properties and behaviors of the soil and/or rock, and the groundwater condition that can affect foundation design and construction. All subsurface conditions that might influence the foundation design and performance should be investigated.

Perform a literature search (see *Geotechnical Investigations*) to gather all relevant information related to site geology, strength of soil and rock, and geologic hazards. Then, develop a prudent exploration plan considering site constraints, geologic variability, and available resources. Borings should be located as close as possible to the proposed foundation.

The exploration plan should include:

- An appropriate number of exploratory borings and/or cone penetration tests (CPT) to develop the design soil profile (AASHTO Table 10.4.2-1).
- An appropriate depth of exploration for the borings or CPT. The depth of exploration should generally extend below the anticipated pile tip elevation a minimum of 20 feet, or a minimum of two times the maximum pile group dimension, whichever is deeper (AASHTO Table 10.4.2-1).
- Standard penetration tests (SPT). When SPTs are to be performed, sampling intervals should be limited to no more than 5 feet.
- Groundwater measurements.
- Soil and water samples for corrosion testing in accordance with current *Caltrans Corrosion Guidelines*.
- Adequate samples for laboratory testing (e.g. classification tests, consolidation test, soil strength parameters required for design).

Design

The following provides design methodologies used to calculate settlement (Service-I Limit State) and pile resistance (Strength and Extreme Event Limit States) in accordance with AASHTO 10.8.1. For appropriate resistance factors refer to AASHTO Table 10.5.5.2.4-1.

Soil properties used for design should come from: (1) SPT correlations (see *Soil Correlations* module) and/or (2) results of laboratory tests under similar field conditions.

The design must also account for geologic hazards such as:

- Liquefaction (see *Liquefaction* module)
- Lateral spreading (see *Lateral Spreading* module)
- Scour: Foundations that are constructed in a watercourse must meet AASHTO guidelines regarding scour depths (AASHTO C2.6.4.4.2). The top of the pile cap must be below the degradation plus contraction scour depth. The bottom of the pile cap must be below the degradation plus contraction plus local pier scour depth.

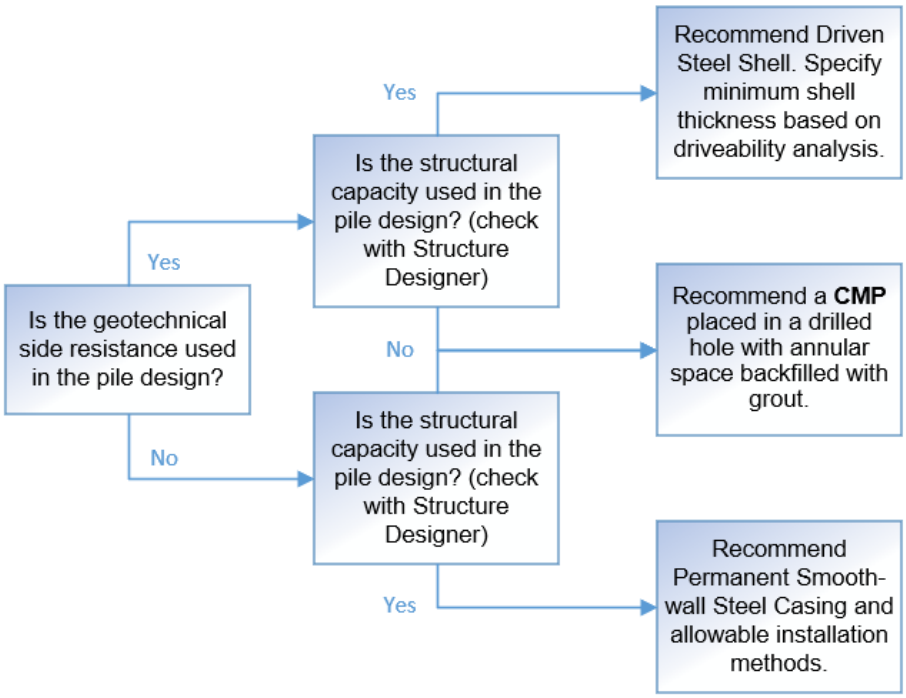
CIDH Concrete Pile Design Considerations

The following table presents options for selection of a permanent smooth-wall steel casing, driven steel shell, or CMP casing.

Permanent Smooth-wall Steel Casing, CMP, or Steel Shell in CIDH Concrete Pile Design

Types	Can it be used for Constructability?	Can it be used for Structural Capacity?	Can it be used for Geotechnical Resistance?	Installation Method
Permanent Smooth-wall Steel Casing	Yes	Yes	No	Drilled, vibrated, oscillated/rotated into place, or placed in drilled hole and annular space backfilled with grout
Corrugated Metal Pipe (CMP) Casing	Yes	No	No	Placed in a drilled hole and annular space backfilled with grout
Driven Steel Shell	Yes	Yes	Yes	Driven

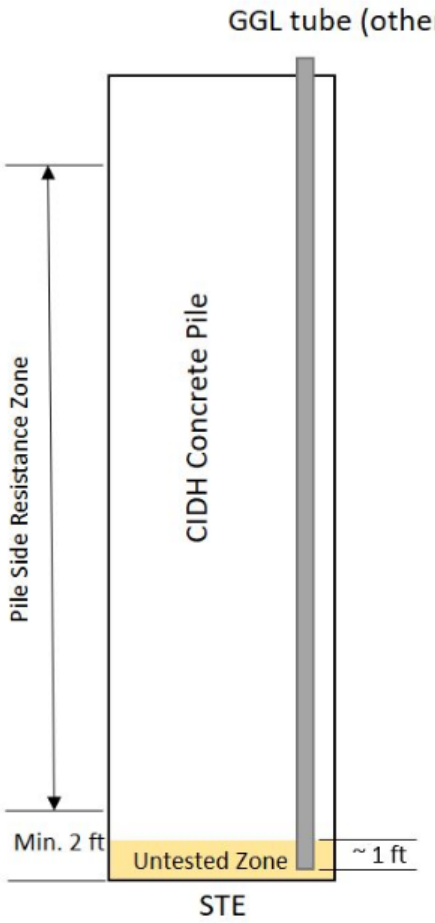
The flow chart shows the most cost-effective options for use of a permanent smooth-wall steel casing, shell or CMP.



Side Resistance Considerations

If it is anticipated that the slurry displacement method will be used for concrete placement, CIDH pile acceptance testing in accordance with California Test (CT) 233, "*Method of Ascertaining the Homogeneity of Concrete in CIDH Piles Using the Gamma-Gamma Test Method*" will be required. Due to the limitations of the gamma-gamma logging (GGL) equipment, there is a zone of untested concrete at the bottom the pile. To account for this zone of untested concrete, the specified tip elevation must be lowered a minimum of two feet below the bottom of the side resistance zone (Figure 1).

Figure 1: Untested Concrete Zone at Bottom of CIDH Pile



Tip Resistance Considerations

If the slurry displacement construction method is anticipated, tip resistance may only be used in rock and with certain considerations, as directed by the OGDPP.

Design Information and Communication (Preliminary Foundation Report)

After the field investigation and testing has been completed, the Geoprofessional must review the design information provided by the Structure Designer which should include:

- General Plan
- Preliminary Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Table X: Preliminary Foundation Design Data Sheet

Support Location	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (Strength Limit State) (kips)
Abutment 1		
Pier 2		
Abutment 3		

Design Process (Preliminary Foundation Report)

Complete the CIDH pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Calculate the Preliminary Tip Elevations

- Calculate the preliminary tip elevations meeting the controlling compression and tension requirements for the Strength Limit State at each support location.

Step 3: Complete Preliminary Foundation Design Recommendations table

- Present the tip elevations for compression in the Preliminary Foundation Design Recommendations table under the Preliminary Tip Elevation column in the Preliminary Foundation Report.

Step 4: Reporting

- Complete the Preliminary Foundation Report according to the *Foundation Reports for Bridges* module.

Design Information and Communication (Foundation Report)

The Geoprofessional must review the design information provided by the Structure Designer, which should include:

- General Plan
- Foundation Plan
- Scour Data Table (if scour potential exists, MTD 3-1, Attachment 1) or Hydraulics Report
- Foundation Design Data Sheet (MTD 3-1, Attachment 1)
- Foundation Factored Design Loads information (MTD 3-1, Attachment 1)

Table X: Foundation Design Data Sheet (MTD 3-1, Attachment 1)

Foundation Design Data Sheet							
Support No.	Pile Type	Finished Grade Elevation (feet)	Cut-off Elevation (feet)	Pile Cap Size (feet)		Permissible Settlement under Service Load (inches)	Number of Piles per Support
				B	L		
Abut 1	30-in diam. CIDH						
Pier 2	96-in diam. CIDH			N/A	N/A		
Abut 3	30-in diam. CIDH						

Table X: Foundation Factored Design Loads (MTD 3-1, Attachment 1)

Foundation Factored Design Loads										
Support No.	Service-I Limit State (kips)		Strength/Construction Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load per Support	Permanent Load per Support	Compression		Tension		Compression		Tension	
			Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1							N/A	N/A	N/A	N/A
Pier 2										
Abut 3							N/A	N/A	N/A	N/A

Design Process (Foundation Report)

Complete the CIDH pile foundation design process by following the steps below:

Step 1: Evaluation of Support Location and Foundation Type

- Verify that the foundation location and type is acceptable considering the known subsurface information and geological hazards (e.g., scour, lateral spreading, liquefaction).

Step 2: Determine the Specified Tip Elevation for the Steel Casing (if applicable)

(Commentary: Depending on the method of excavation, the diameter of the rock socket may need to be sized at least 8 inches smaller than the nominal casing size to permit seating of casing and insertion of rock drilling equipment (AASHTO 2017, C10.8.1.3).

Step 3: Calculate the Design Tip Elevations for the Piles

- Calculate the design tip elevations meeting the controlling compression and tension requirements for the Strength Limit State and the Extreme Event Limit State at each support location.
- Calculate the design tip elevations meeting the permissible settlement criteria for Service-I Limit State.

  (Commentary: Pile design should ensure that strength limit state considerations are satisfied before checking service limit state considerations. For piles embedded adequately into dense granular soils such that the equivalent footing is located on or within the dense granular soil, and furthermore are not subjected to downdrag loads, a detailed assessment of the pile group settlement may be waived. If the design tip for service limit state is waived, then a note should be placed at the bottom of the pile data table).
- Using the design tip elevation from Strength or Extreme Limit State, verify the settlement under the Service-I Limit State load is less than the permissible settlement.

Step 4: Determine the Controlling Design Tip Elevation

- The deepest design tip elevation in compression, tension, and settlement determined by Geotechnical Services will be the controlling design tip elevation, and shown in the Foundation Design Recommendations and Pile Data Tables. The Structure Designer will need to compare their design tip elevation for lateral loading to the GS Controlling Design Tip Elevation. The lowest elevation will become the final Specified Tip Elevation to be shown on the contract plans.

Step 5: Provide Foundation Recommendations Table and Pile Data Table

- Refer to *Foundation Reports for Bridges* for the applicable tables.

Step 6: Lateral Tip Considerations

- Prior to signing and stamping the report, obtain the lateral tip elevation from the Structure Designer and verify that it is higher than the Specified Tip Elevation.
    - If the lateral tip is higher than or equal to the specified tip elevation then there is no action required by the GP.
    - If the lateral tip is lower, the GP must verify that the pile can be installed to the lateral tip elevation and that all other recommendations in the report are correct (e.g., pile tip is now below groundwater, pile tip is now in rock).

Step 7: Reporting

- Once the design of the CIDH pile foundation is completed, the Geoprofessional may proceed with completing the Foundation Report according to the *Foundation Report for Bridges* guidelines.

Design Data Documentation

If the contractor uses either the slurry displacement method to place concrete or temporary casing to aid in dewatering the drilled hole, the CIDH pile will be inspected in accordance with California Test (CT) 233, "*Method of Ascertaining the Homogeneity of Concrete in CIDH Piles Using the Gamma-Gamma Test Method*". If anomalies are detected in the pile, the Foundation Testing Branch will issue a *Pile Acceptance Report* stating that the pile be rejected. Geotechnical Services will then participate in a process, initiated by Structure Construction, to evaluate the impact of the anomaly on the pile's capacity and determine a path forward.

To meet the time requirements in the Standard Specifications for evaluating the effect of anomalies on the geotechnical capacity of CIDH piles, pertinent geotechnical design information must be readily accessible to the GP and BC. The following CIDH pile information must be retained in the Geotechnical Design Office's *Electronic Project File Storage System* (EPFSS).

- Foundation Report
- Log of Test Borings, (LOTB)
- Geological profile used for each CIDH pile design
- Soil and or rock strength parameters used in the design of each support/pile
- Calculations and/or computer/spreadsheet outputs used to determine each pile SPTE

Evaluation of Anomalous CIDH Concrete Piles

When a pile is rejected, the State has limited time per Standard Specification section 49-3.02A(4)(d)(iv) *Rejected Piles*, to determine which of the following options is available to the contractor:

1. The pile must be supplemented or replaced.
2. The pile must be repaired.
3. The pile is adequate with the anomaly left in place.

The *Pile Design Data Form* (PDDF) is used by Structure Construction with input by the Foundation Testing Branch, Structure Design, GS, and the METS Corrosion Branch to determine acceptable options for a rejected pile. The FTB will complete Part 1 of the PDDF, which will identify the location and extent of the anomaly, and attach the PDDF to the *Pile Acceptance Report*. A copy of the *Pile Acceptance Report* is sent to the Geoprofessional and the Chair of the CIDH Pile Mitigation Committee. SC will request that SD, GS and Corrosion complete their respective sections and return the PDDF to SC. The information completed by the FTB for Part 1 is used by the Geoprofessional to complete Part 2, SD to complete Part 3, and the Corrosion to complete Part 4 of the PDDF. See MTD 3-7, *Design Data Documentation and Evaluation of Anomalous Concrete Shafts,* for details.

The FTB produces a plot on the PDDF (Part 1) that identifies the top and the bottom of the anomaly and the percentage of the pile affected. Using the original design calculations and assumptions (found in the EPFSS), calculate:

- the area of the anomaly on the outside of the pile and the skin friction contribution of that area.

  <*Note: The geotechnical evaluation assumes that the anomaly does not affect the ability of the pile to transmit load to the pile below the anomaly*.>

- the area of the anomaly at the pile base and the end bearing contribution of that area.

If the required nominal resistance exceeds the anomaly-reduced nominal resistance, the pile is unacceptable. If the pile is determined to be adequate with the anomaly in place, then the contractor may either repair the pile and receive full payment or leave the anomaly in place and incur an administrative deduction as specified in the contract.

Complete Part 2 of the PDDF using the results of the analysis and return the form to SC.

If the capacity of the pile is determined to be inadequate by GS, SD or Corrosion, then the anomaly mitigation process will initiate and the FTB, SC, GS, and SD will collectively determine if the pile can be repaired, supplemented, or replaced. The standard repair techniques are excavation, removal and replacement for anomalous concrete near the top of the pile, or grouting repair to lower potions of the pile. If the standard repair methods are not feasible, SC will hold a *CIDH Pile Non-Standard Mitigation Meeting*, per *Bridge Construction Memo BCM 130-21* to determine an acceptable mitigation strategy.

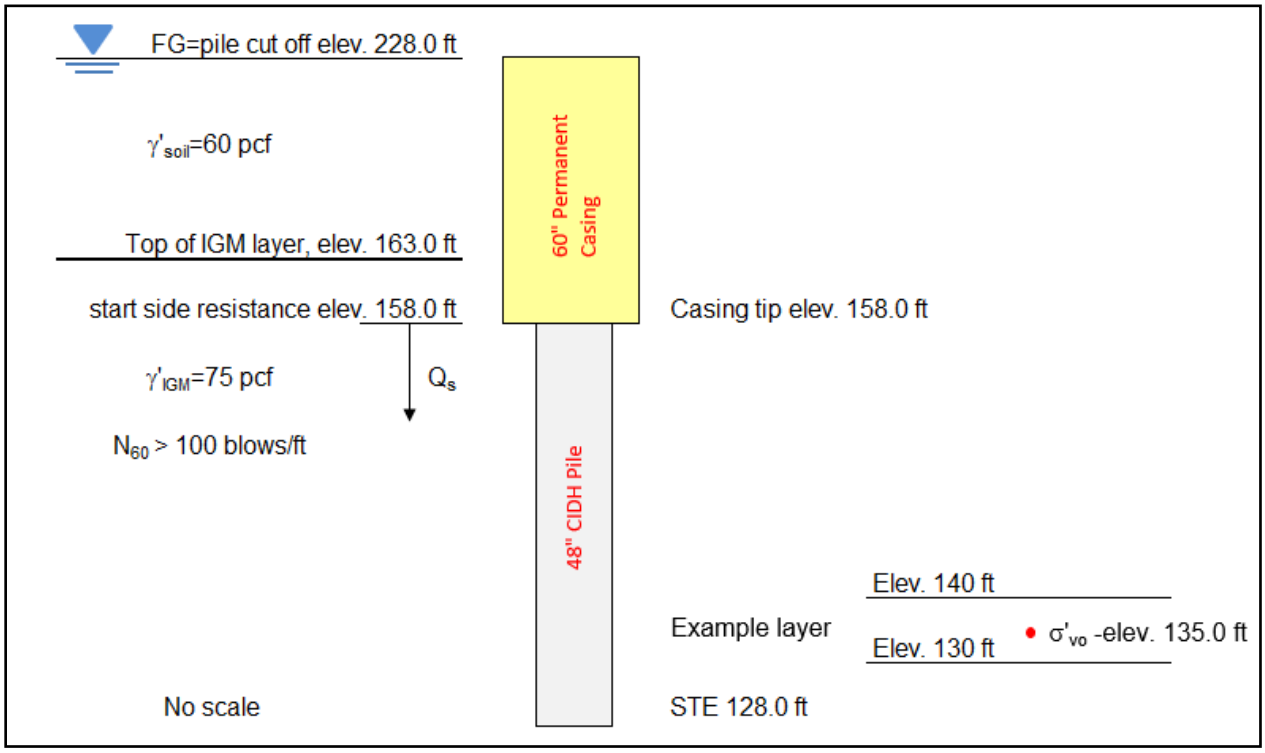
Attachments

- Appendix A: Example Design Calculations for Cohesionless IGM
- Appendix B: Example Design Calculations for Rock
- Appendix C: Example Calculations for Anomaly Evaluation Process

Appendix A: CIDH Pile Design in Cohesionless Intermediate Geomaterial (IGM)

The following presents the design calculations for a 10-foot layer (Elevation 140 feet to 130 feet) located at an abutment. Figure A1 shows the CIDH pile and soil/rock layers.

Figure A1: Simplified Soil/Rock Profile for Abutment 1 Design



Information provided by Structure Design

- 60-inch Permanent Steel casing is required.
- 48-inch CIDH pile (below casing tip).
- Controlling Factored Design Load = 1146 kips (Strength Limit State).
- The pile cutoff elevation is 228.0 ft.

Geotechnical Design Considerations

- The side resistance of the permanent casing is not used in the design.
- Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity of the piles is based on side resistance of 48-inch diameter CIDH piles and no tip resistance was considered. The design pile side resistance starts at elevation 158.0 ft.
- The required nominal resistance is factored design load divided by the resistance factor of side resistance or 1146 kips / 0.7 = 1637 kips, round up to 1640 Kips.

- Cohesionless IGM design method (for granular geomaterials with SPT N_{60} value greater than 50) will be used to determine pile side resistance.
- Based on the LOTB, representative N_{60} value of 100 was selected for cohesionless IGM design method. N_{60} values should be limited to 100 or less (O'Neill et al., 1996, p.32).
- Assume groundwater surface is at elevation 228.0 ft.

Step 1: Determine the Preconsolidation Pressure (σ'_p):

$$\sigma'_p = 0.2\, N_{60}\, \sigma_p$$

Where:

N_{60} = representative SPT blow count corrected for hammer efficiency effect

σ_p = atmospheric pressure taken as 2120 psf

$$\boldsymbol{\sigma'_p} = 0.2 * 100 * 2120 \text{ psf} = \mathbf{42400\ psf} \quad (N_{60} \leq 100)$$

Step 2: Determine the Overconsolidation Pressure (OCR):

$$OCR = \frac{\sigma'_p}{\sigma'_{vo}}$$

Where:

σ'_{vo} = effective overburden pressure at mid layer

σ'_{vo} = (60 pcf)(elev. 228 ft -163 ft) + (75 pcf)(elev. 163 ft- 158 ft) + (75 pcf)(elev. 158 ft – 140 ft) + (75 pcf)(10 ft/2) = 6000 psf

$$\mathbf{OCR} = \frac{42400 \text{ psf}}{6000 \text{ psf}} = \mathbf{7.07}$$

Step 3: Determine the Effective Friction Angle (ϕ'):

$$\phi' = \arctan\left\{\left[\frac{N_{60}}{12.2 + 20.3\left(\frac{\sigma'_{vo}}{\sigma_p}\right)}\right]^{0.34}\right\}$$

$$\boldsymbol{\phi'} = \arctan\left\{\left[\frac{100}{12.2+20.3\left(\frac{6000 \text{ psf}}{2120 \text{ psf}}\right)}\right]^{0.34}\right\} = \mathbf{48.5^o}$$

Step 4: Determine the Coefficient of Horizontal Earth Pressure (K_o):

$$K_o = (1 - \sin\phi')\ OCR^{\sin\phi'}$$

$$\mathbf{K_o} = (1 - \sin(48.5°))7.07^{\sin(48.5°)} = \mathbf{1.086}$$

Step 5: Determine the Unit Side Resistance (q_s):

$$q_s = K_o \tan\phi'\ \sigma'_{vo}$$

As the "wet method" (slurry displacement method) is anticipated for the concrete placement, a reduction factor of 0.75 is applied to the effective internal friction angle, ϕ' (O'Neill et al., 1996, p.102).

$\mathbf{q_s} = (1.086)\tan(0.75 * 48.5°)\ 6000\ \text{psf} = \mathbf{4800\ psf}$ ("wet method")

Step 6: Determine the Side Resistance (R_s) *of the layer:*

$R_s = q_s\ \pi\ D\ L$

Where:

D = pile diameter

L = layer thickness

R_s = (4800 psf)(3.14)(4 ft)(10 ft) = 602880 lbs = **603 kips**

Table A1 presents side resistance values for the other IGM layers.

Table A1: Side Resistance

Bottom Elevation (feet)	Layer Thickness (feet)	γ' (pcf)	$\sigma'_{v(mid\ layer)}$ (psf)	N_{60}	σ'_p (psf)	OCR	Φ' (degrees)	K_o	q_s (psf)	R_s (kips)
158	70	0	4275[1]	0	0	0	0	0	0	0
150	8	75	4575	100	42400	9.27	50.6	1.27	4530	455
140	10	75	5250	100	42400	8.08	49.6	1.17	4663	586
130	10	75	6000	100	42400	7.07	48.5	1.09	4800	603

1: total overburden Total = 1645 kips

Design Summary

Additional pile length should be added to the specified tip to account for the untested zone at bottom of pile. For this example, additional 2 ft was added to the required pile length.

Design Tip Elev. = 130 ft (from calculations)
- 2 ft (additional length for untested zone)

Specified Tip Elev. = 128 ft

Foundation Reports for Bridges Section 3.14.3, "Notes for Construction (CIDH Piles)" requires reporting of "*how the geotechnical resistance is derived*." Table A2 presents what would be reported for this example.

Table A2: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	Side Resistance Start Elevation (feet)	Side Resistance End Elevation (feet)	Specified Tip Elevation (feet)
Abutment 1	158.0	130.0	128.0

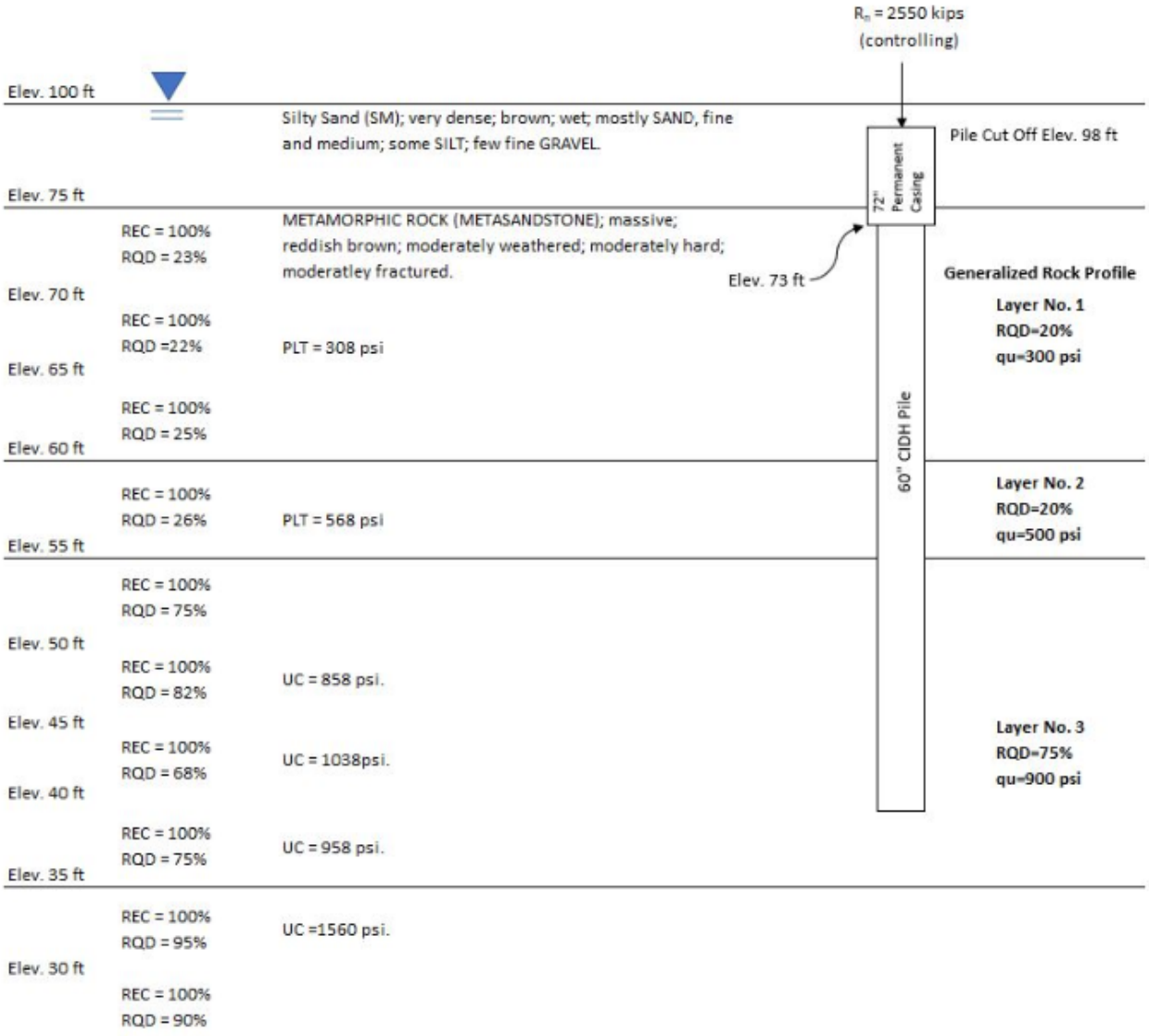
References

O'Neill M., Townsend F., Hassan, K., Buller, A, and Chan P., "Load Transfer for Drilled Shafts in Intermediate Geomaterials," FHWA-RD-95-172, Federal Highway Administration, McLean, VA, 1996

Appendix B: CIDH Pile Design in Rock

Appendix B presents the design calculations for a 60-inch diameter CIDH pile in rock located at Bent 2. Figure B1 shows the CIDH pile and soil/rock layers.

Figure B1: Simplified Soil/Rock Profile Design



Information provided by Structure Design

- 72-inch Permanent Steel casing is required.
- 60-inch CIDH pile (below casing tip).
- At Bent 2, the Controlling Factored Design Load = 1783 kips (Strength Limit State)
- The pile cutoff elevation is 98.0 ft.

Geotechnical Design Considerations

- The side resistance of 72-inch diameter permanent casing is not used in the design.
- Due to the anticipation that concrete placement for the CIDH piles will require slurry displacement methods, the calculated geotechnical capacity is based on side resistance of 60-inch diameter CIDH piles and no tip resistance was considered in this example.
- The controlling design load is 1783 kips (Strength Limit State). The required nominal resistance is 1783 kips /0.7 = 2547 kips, round up to 2550 Kips.
- The rock joints are closed.

Step 1: Determine the Factored Nominal Resistance (R_R)

$$R_R = \varphi_{qp} R_p + \varphi_{qs} R_s \qquad \text{(AASHTO 2017, 10.8.3.5-1)}$$

Where:

R_p = nominal tip resistance

R_s = nominal side resistance

φ_{qp} = resistance factor for tip resistance

φ_{qs} = resistance factor for side resistance

Tip resistance is not used in this design example, therefore:

$$R_R = \varphi_{qs} R_s$$

Step 2: Determine the Unit Side Resistance (q_s) *of CIDH pile in rock*

Due to the low RQD values, assume temporary casing may be used during construction.

$$\frac{q_s}{P_a} = 0.65 \alpha_E \sqrt{\frac{q_u}{P_a}} \qquad \text{(AASHTO 2017, 10.8.3.5.4b-2)}$$

Where:

P_a = atmospheric pressure taken as 2.12 ksf

α_E = joint modification factor (AASHTO 2017, Table 10.8.3.5.4b-1)

q_u = uniaxial compressive strength of rock in ksf

Equation 10.8.3.5.4b-2 becomes:

$q_s = 0.946\alpha_E\sqrt{q_u}$ (q_s, q_u are in ksf)

$q_s = 2.5\alpha_E\sqrt{q_u}$ (q_s, q_u are in psi)

$q_s = 30\alpha_E\sqrt{q_u}$ (q_s, q_u are in psf)

Figure B2: AASHTO Table 10.8.3.5.4b-1

Table 10.8.3.5.4b-1—Estimation of α_E (O'Neill and Reese, 1999)

RDQ (%)	Joint Modification Factor, α_E	
	Closed Joints	Open or Gouge-Filled Joints
100	1.00	0.85
70	0.85	0.55
50	0.60	0.55
30	0.50	0.50
20	0.45	0.45

For Layer No. 1:

Representative rock properties from elev. 73 ft to elev. 60 ft:

RQD = 20 %, use α_E = 0.45 for closed joints.

q_u = 300 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.45)\sqrt{300\text{psi}} = 19.5 \text{ psi}$$

$$R_s = 19.5 \text{ psi } (3.14)(60 \text{ in})(\text{elev. } 73 \text{ ft} - \text{elev. } 60 \text{ ft})\left(\frac{12 \text{ in}}{1 \text{ ft}}\right)\left(\frac{1 \text{ kip}}{1000 \text{ lb}}\right) = 573 \text{ kips}$$

If the RQD values are below 20% for most of the pile length, an alternative design method, such as the cohesionless Intermediate Geo Material method, should be used. Do not combine side resistances from IGM and rock design methods.

For Layer No. 2:

Representative rock properties from elev. 60 ft to elev. 55 ft:

RQD = 20 %, use α_E = 0.45 for closed joints

q_u = 500 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.45)\sqrt{500\text{psi}} = 25.2 \text{ psi}$$

$$R_s = 25.2 \text{ psi } (3.14)(60 \text{ in})(\text{elev. } 60 \text{ ft} - \text{elev. } 55 \text{ ft})(\frac{12 \text{ in}}{1 \text{ ft}})(\frac{1 \text{ kip}}{1000 \text{ lb}}) = 285 \text{ kips}$$

For Layer No. 3:

Representative rock properties from from elev. 55 ft to elev. 35 ft:

RQD = 75 %, interpolate between RQD = 70% & 80% for α_E

$\alpha_E = 0.85 + \frac{(1.00-.85)}{(100-70)}(75-70) = 0.88$ for closed joints

q_u = 900 psi

Pile nominal side resistance, R_s, calculation:

$$q_s = 2.5\alpha_E\sqrt{q_u} = 2.5(0.88)\sqrt{900\text{psi}} = 66 \text{ psi}$$

Required Nominal Resistance at top of pile is 2550 kips. At top of layer no. 3, elev. 55 ft, the total pile side resistance is 573 kips + 285 kips = 858 kips. The pile design still needs 1692 kips (= 2550 kips – 858 kips) side resistance. Check for the minimum pile length (L_{min}) required in layer no. 3 to meet the 1692 kips in side resistance.

$$L_{min} = \frac{1692 \text{ kips}}{(66 \text{ psi})(3.14)(60 \text{ in})(\frac{12 \text{ in}}{\text{ft}})(\frac{1 \text{ kip}}{1000 \text{ lb}})} = 11.4 \text{ ft, say } 12 \text{ ft}$$

Design pile tip elevation is Elev. 55 ft -12 ft = 43 ft

Design Summary

Additional pile length should be added to the specified tip to account for the untested zone at bottom of pile. For this example, additional 2 ft was added to the required pile length.

Design Tip Elev. = 43 ft (from calculations)
- 2 ft (additional length for untested zone)
Specified Tip Elev. = 41 ft

Foundation Reports for Bridges Section 3.14.3, “Notes for Construction (CIDH Piles)” requires reporting of “*how the geotechnical resistance is derived*.” Table A2 presents what would be reported for this example.

Table B1: CIDH Concrete Pile Side Resistance Zone Elevations

Support Location	Side Resistance Start Elevation (feet)	Side Resistance End Elevation (feet)	Specified Tip Elevation (feet)
Bent 2	73.0	43.0	41.0

Appendix C: Evaluating an Anomalous CIDH Concrete Pile

1. Assemble required design information, and the *Pile Acceptance Report*.
2. Report the Required Nominal Resistances shown in contract plans.
3. Determine the Remaining Required Nominal Resistance ($RRNR_{Ct}$ and $RRNR_{Tt}$) to be developed below the top of each anomalous section.
4. Determine the Remaining Required Nominal Resistance ($RRNR_{Cb}$ and $RRNR_{Tb}$) to be developed below the bottom of each anomalous section.
5. Determine and report the "as-designed" resistance over the entire pile surface from the top elevation to the bottom elevation of each anomaly.
6. Determine and report the calculated capacity loss in each of the anomalous lengths.
7. Report the soil and/or rock type.
8. Determine and report if each anomalous section is geotechnically acceptable.

If a pile has multiple anomalies and various combinations exist in a manner that some anomalies could be marked either "acceptable" or "unacceptable", include a note in the *Comments* section of Part 2 of the PDDF that SD, GS, SC and Corrosion must discuss and develop a repair strategy.

If additional sources of resistance are available, they should be considered to contribute to the available resistance. These include:

- Tip resistance if not used in the pile design.
- Side resistance near the bottom of the pile not used in the pile design.
- Pile resistances increases due to reevaluation of the pile design calculations, e.g., the soil parameters may be reevaluated to see if using higher (but still realistic) values will result in increased nominal resistance.
- The structural designer may also check the actual demand of the anomalous pile. In pile groups the actual demands for interior piles may be considerably less than what is shown on the plans.
- The structural designer may accept additional settlement in lieu of the required capacity.

Definitions

Refer to Figure 2 for the following:

- R_{NC} = Nominal Resistance in compression.
- R_{NT} = Nominal Resistance in tension.
- STE = Specified Tip Elevation.
- Z_t = Side resistance zone start elevation.
- Z_b = Side resistance zone end elevation.
- AE_t = Top of anomaly elevation.

- AE_b = Bottom of anomaly elevation.
- L = Length of side resistance zone (Z_t - Z_b).
- $RRNR_{Ct}$ / $RRNR_{Tt}$ = Remaining Required Nominal Resistance at top of Anomaly for compression or tension, respectively
- $RRNR_{Cb}$ / $RRNR_{Tb}$ Remaining Required Nominal Resistance at bottom of Anomaly for compression or tension, respectively
- AF=Affected Pile Surface Area (%), see Part 1 of Pile Design Data Form.
- $R_{s\text{-}add}$ = Pile nominal resistance below the side resistance zone.

Example Calculations

Project Information

Bridge Name:	Deepwater Creek Bridge (Replace)
Bridge #:	53C-XXXX
Pile Diameter:	10 feet
Nominal Resistance:	R_{NC} = 6,600 kips
	R_{NT} = 0 kips
Pile Cutoff El.:	5.8 feet
Specified Pile Tip El.:	-133.0 feet
Ground Water El.:	2.31 feet

GGL Results (Figure 3)

Pile C at Bent 12 has two anomalous sections:

- Section A-A: Elevation 4.6 ft to 3.6 ft
- Section B-B: Elevation -10.8 ft to -16.0 ft

Geotechnical Design of Pile C at Bent 12

The Foundation reports states that:

- The geotechnical capacity of the CIDH pile is based on side resistance (R_s) only.
- The geotechnical capacity was calculated from one pile diameter (10 ft) below the pile cutoff elevation to one pile diameter (10 ft) above the STE.

Analysis Method

Step 1: Assemble and Review Pertinent Information

- From the *Pile Acceptance Report* and/or the PDDF (Figure 1) determine the location and size of anomalous zones.
    - Section A-A: from El. 4.6 ft to 3.6 ft
    - Section B-B: from El. -10.8 ft to -16.0 ft.
- Review the Foundation Report and calculations to determine where and how the geotechnical capacity was obtained. In this example, the geotechnical capacity is from skin friction only:

- Z_t = Pile Cutoff El. – One Pile Dia. = El. 5.8ft – 10ft = El. -4.2ft
- Z_b = SPTE + One Pile Dia. = El. -123.0 ft

- Plot Remaining Required Nominal Resistance vs. Elevation using the design calculations (Figure 4).

Step 2: Report Required Nominal Resistances Shown in Contract Plans

- Compression, R_{NC} = 6,600 kips
- Tension, R_{NT} = 0 kips.

Steps 3-6: Determine the "as-designed" nominal resistance over the entire pile surface from the top elevation to the bottom elevation of each anomaly and the calculated capacity loss within each anomaly length

(There is no tension demand in this example problem, therefore, there is no tension demand calculation shown.)

Anomaly A-A

Anomaly A-A is above the design side resistance and therefore does not reduce the CIDH pile's nominal capacity. Report the "as-designed" nominal resistance over the entire pile surface from the top elevation to the bottom elevation of each anomaly and the capacity loss within the anomaly length (kips), in Part 2 of the PDDF:

Section A-A: Compression: 0 / 0 Tension: 0 / 0

Anomaly B-B

Determine the "As-designed" Nominal Resistance as follows:

- $RRNR_{Ct} - RRNR_{Cb}$

  Where $RRNR_{Ct}$ = 6,600 - 367 Kips = 6,233 kips (Figure 4)

  $RRNR_{Cb}$ = 6,600 - 656 Kips = 5,944 kips (Figure 4)

- 6,233 kips – 5,944 kips = 289 kips (or 656 kips – 367 kips = 289 kips)

Calculate the capacity loss as follows:

- $(RRNR_{Ct} - RRNR_{Cb}) \times AF/100$

  Where AF = 100% (Part 1 of PDDF, 12 of 12 inspection tubes)

- (6,233 - 5,944) x 100/100 = 289 kips

Anomaly B-B is within the design side resistance zone and therefore reduces the CIDH pile's nominal capacity.

Report the "as-designed" nominal resistance over the entire pile surface from the top elevation to the bottom elevation of each anomaly and the capacity loss within the anomaly length (kips), in Part 2 of the PDDF:

Section B-B: Compression: 289 / 289 Tension: 0 / 0

Step 7: Report the soil and/or rock type

Using the LOTB (Figure 4), report the soil/rock type in the anomalous zone(s) in Part 2 of the PDDF:

Section A-A: Soil and/or Rock Type: Silty Sand

Section B-B: Soil and/or Rock Type: Silty Sand and Fat Clay

Step 8: Determine if each anomaly section is geotechnically acceptable

Anomaly A-A is located above the design side resistance zone and has no adverse impact on the geotechnical capacity of the CIDH pile. Check the appropriate boxes in the PDDF:

Section A-A: Section is Geotechnically: [X] Acceptable [ ] Unacceptable

Anomaly B-B results in a potential loss of 289 kips of side resistance. However, the geotechnical design of the pile did not include side resistance in the bottom 10 feet of the pile. This additional side resistance may be used to counter the capacity loss within the Anomaly B-B.

Determine the pile nominal resistance below the side resistance zone, from Elev. -123.0 ft to Elev. -133.0 ft.

$R_{s\text{-}add}$ = (7155-6600) = 555 kips

Because the excess of 555 kips is greater than the potential loss of 289 kips, the pile is considered to be acceptable. Check the appropriate boxes in the PDDF:

Section B-B: Section is Geotechnically: [X] Acceptable [ ] Unacceptable

Attachments

- Figure C1: Pile Design Data Form: Anomaly A-A
- Figure C2: Pile Design Data Form: Anomaly B-B
- Figure C3: Example Diagram
- Figure C4: GGL Plot
- Figure C5: Log of Test Borings (LOTB)

References

1. MTD 3-7 – Design Data Documentation and Evaluation of Anomalous Concrete Shafts (April 2012)
2. BCM 130-10 – Testing of CIDH Piling (June 2014)
3. BCM 130-20 – Cast-In-Drilled-Hole (CIDH) Pile Preconstruction Meeting (December 2011)
4. BCM 130-21 – CIDH Pile Non-Standard Mitigation Meeting (June 2014)
5. California Test (CT) 233 – Method of Ascertaining the Homogeneity of Concrete in Cast-In-Drilled-Hole (CIDH) Piles Using the Gamma-Gamma Test Method (November 2005)

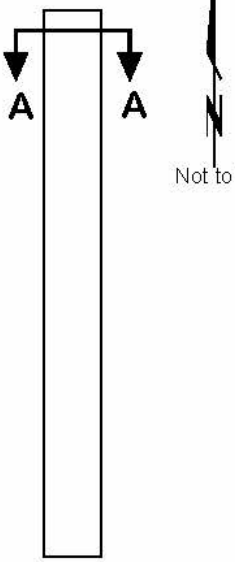
Pile Design Data Form

Figure C1

1 Foundation Testing

Name: GS-FTB Engr
Phone: 916-227-xxxx
Date:

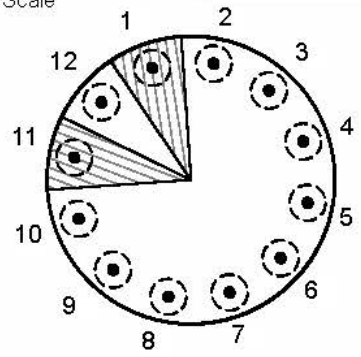
Testing Performed: X GGL CSL





Section A - A

Elev: 4.6 to 3.6 ft
Depth: 1.2 to 2.2 ft
Diameter 10 ft
Depth Ref: Plan Pile Cut-Off Elev.



Provide the relevant elevations.

Plan Pile Cut-off Elev: 5.75 ft
Plan Const. Joint Elev: NA
Plan Top of Pile Pedestal Elev: NA
Reported Casing Tip Elev: NA
Reported Pile Tip Elev: -133.00 ft

Anomaly Description:
Section A - A: Anomaly was detected in two (2) GGL inspection pipes. May affect up to 17% of shaft cross-section at this location.

2 Geotechnical
(See CT Geotechnical Manual)

Name:
Phone:
Date:

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 6600 Tension (kips): 0

Required Nominal Resistance at top of Anomaly:
Compression (kips): 6600 Tension (kips): 0

"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 0 / 0 Tension (kips): 0 / 0

Soil and/or Rock Type: Silty Sand

Section is geotechnically: ☑ **Acceptable** ☐ **Unacceptable**

Comments:
The anomalous section is above the pile side resistance zone.

3 Structural
(See MTD 3-7)

Name:
Phone:
Date:

As-Designed Capacity of Shaft at Anomaly
Shear: Moment:

Maximum Demand of Shaft at Anomaly
Shear: Moment:

Note: Section shall also be evaluated for axial capacity at anomaly.

Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**

Comments:

4 Corrosion

Comments:

Section repair is: ☐ **Required** ☐ **Not Required**

Name:
Phone:
Date:

The groundwater elevation must be assessed from the *Geotechnical Report* and *Section 7.6 Cast-in-Drilled-Hole (CIDH) Pile Anomalies* in the most current *California Department of Transportation Corrosion Guidelines*.

5 Oversight Engineer Concurrence

Geotech: Struct: Corr:

6 Construction

Section is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable; Mitigation is Required

Comments:

Structure Rep:
Phone:
Date:

Structure: Struct. No: Support:
Dist-Co-Rte-PM: EA / EFIS: Pile:

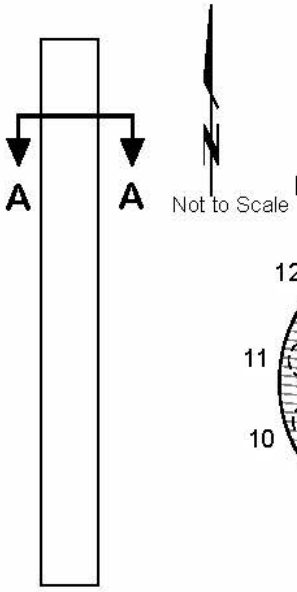
Pile Design Data Form

Figure C2

1 Foundation Testing

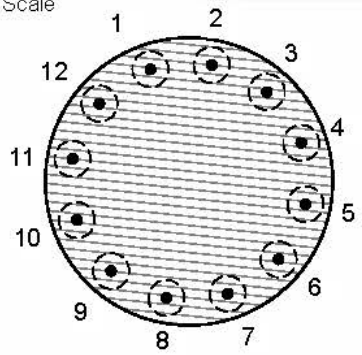
Name: GS-FTB Engr
Phone: 916-227-xxxx
Date:

Testing Performed: X GGL CSL



Section A - A

Elev: 4.6 to 3.6 ft
Depth: 1.2 to 2.2 ft
Diameter 10 ft
Depth Ref: Plan Pile Cut-Off Elev.



Provide the relevant elevations.

Plan Pile Cut-off Elev: 5.75 ft
Plan Const. Joint Elev: NA
Plan Top of Pile Pedestal Elev: NA
Reported Casing Tip Elev: NA
Reported Pile Tip Elev: -133.00 ft

Anomaly Description:
Section A - A: Anomaly was detected in two (2) GGL inspection pipes. May affect up to 17% of shaft cross-section at this location.

2 Geotechnical
(See CT Geotechnical Manual)

Name:
Phone:
Date:

Required Nominal Resistance at top of Shaft (per contract plans):
Compression (kips): 6600 Tension (kips): 0

Required Nominal Resistance at top of Anomaly:
Compression (kips): 6600 Tension (kips): 0

"As-Designed" nominal resistance over entire pile surface from the top to bottom elev. of anomaly/capacity loss within anomaly length (kips):
Compression (kips): 0 / 0 Tension (kips): 0 / 0

Soil and/or Rock Type: Silty Sand

Section is geotechnically: ☑ **Acceptable** ☐ **Unacceptable**

Comments:
The anomalous section is above the pile side resistance zone.

3 Structural
(See MTD 3-7)

Name:
Phone:
Date:

As-Designed Capacity of Shaft at Anomaly
Shear: _____ Moment: _____

Maximum Demand of Shaft at Anomaly
Shear: _____ Moment: _____

Note: Section shall also be evaluated for axial capacity at anomaly.

Section is structurally: ☐ **Acceptable** ☐ **Unacceptable**

Comments:

4 Corrosion

Comments:

Section repair is: ☐ **Required** ☐ **Not Required**

Name:
Phone:
Date:

The groundwater elevation must be assessed from the *Geotechnical Report* and *Section 7.6 Cast-in-Drilled-Hole (CIDH) Pile Anomalies* in the most current *California Department of Transportation Corrosion Guidelines*.

5 Oversight Engineer Concurrence

Geotech: _____ Struct: _____ Corr: _____

6 Construction

Section is: ☐ Acceptable with Administrative Deduction ☐ Unacceptable; Mitigation is Required

Comments:

Structure Rep:
Phone:
Date:

Structure: _____ Struct. No: _____ Support: _____
Dist-Co-Rte-PM: _____ EA / EFIS: _____ Pile: _____

Example Diagram

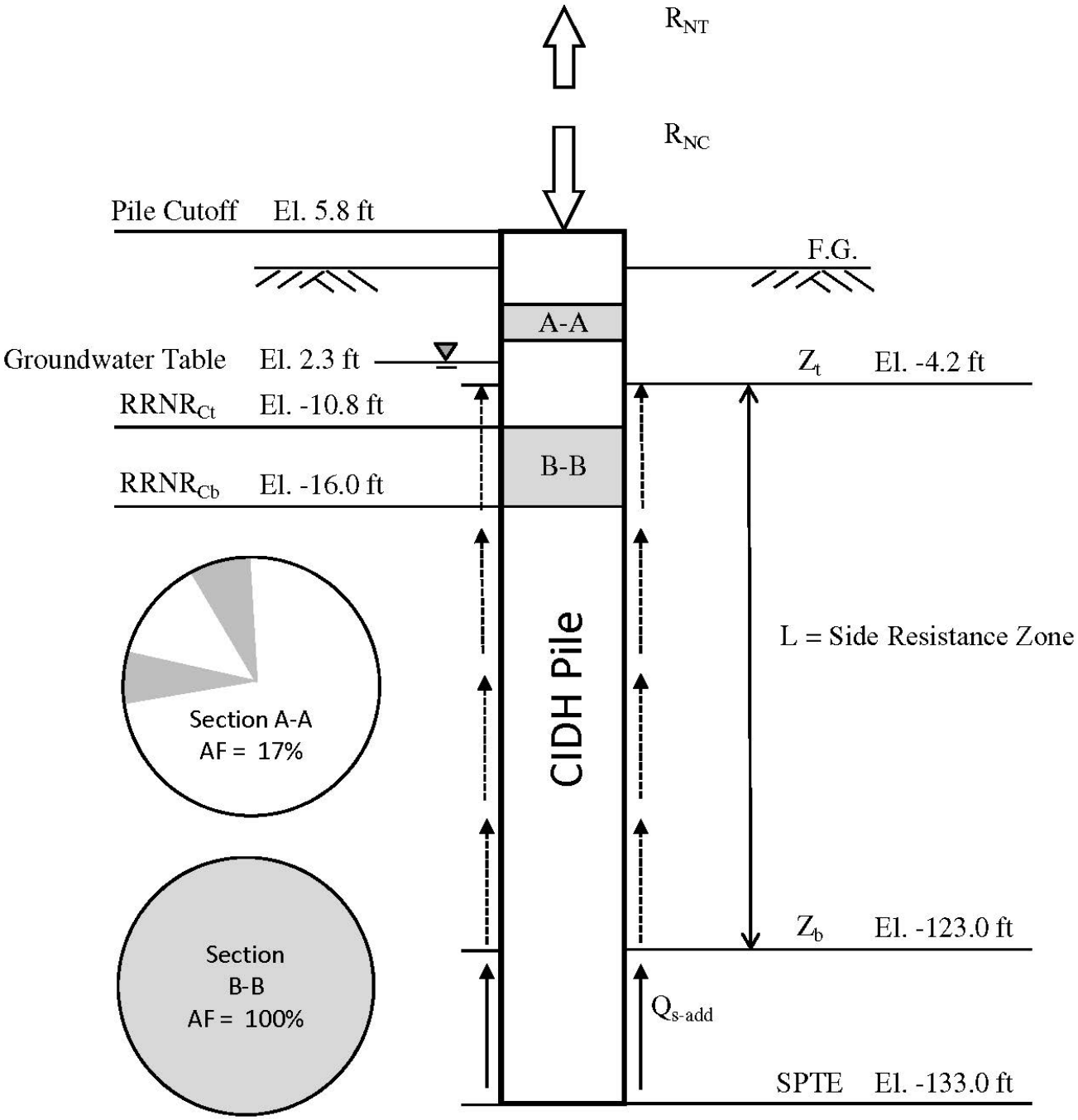
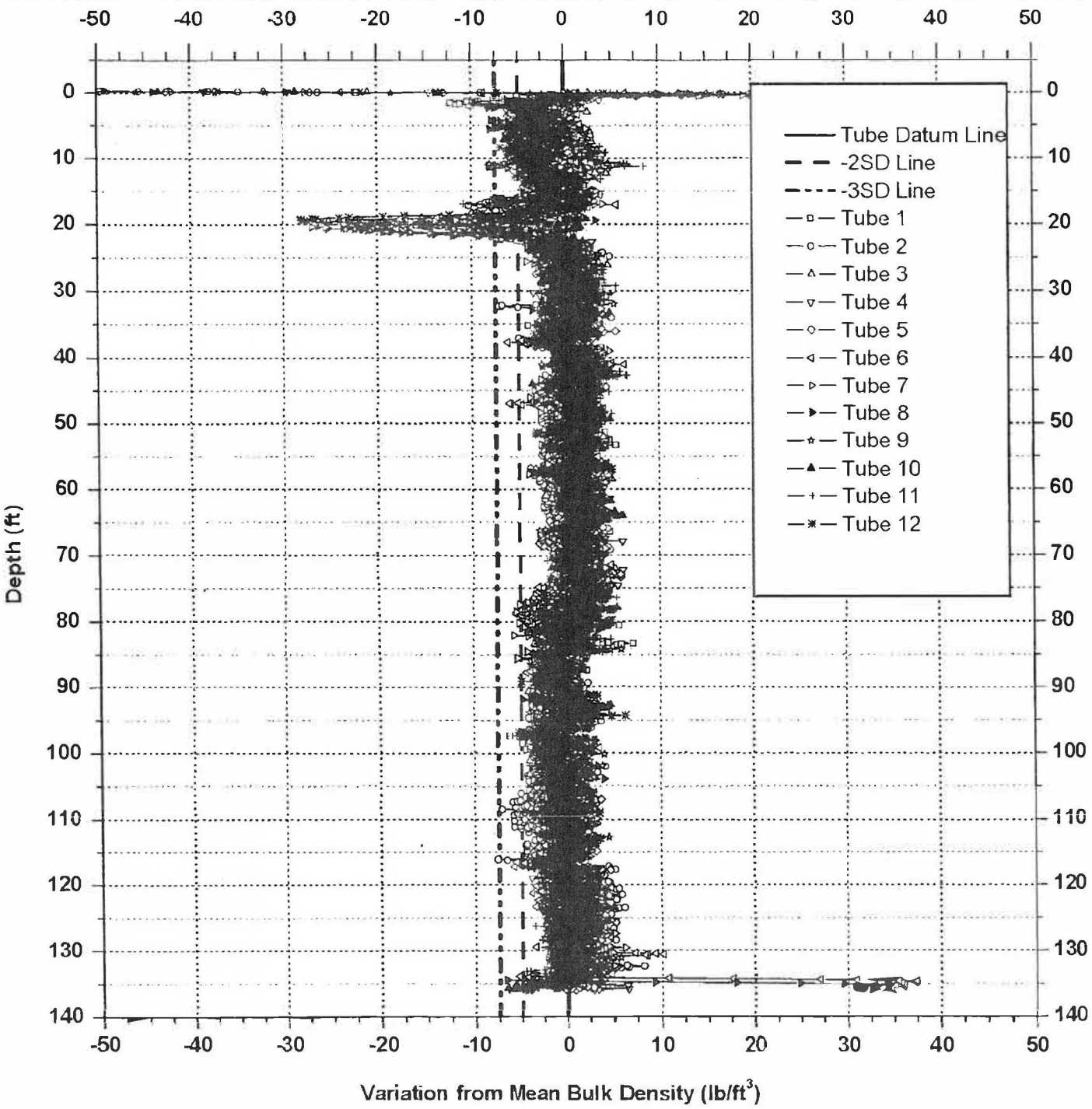


Figure C3



GAMMA-GAMMA LOGGING ACCEPTANCE TEST RESULTS

Deepwater Creek Bridge (Replace)

CIDH Pile C at Bent 12

EA 07-XXXX
Bridge Number 53C-XXXX
07-LA-47-
Date Tested: 01/20/2021

10-ft dia. CIDH Pile
Reported Top of Concrete Elev.= +5.75 ft
Reported Tip Elev.= -133.0 ft
Winch-Probe-Source Number: 1273-2575-568

CALIFORNIA FTB Improving Mobility

The mean and standard deviation were calculated using density readings for all inspected tubes, excluding portions significantly impacted by reinforcement, anomalies, and water, as appropriate.

Figure C4

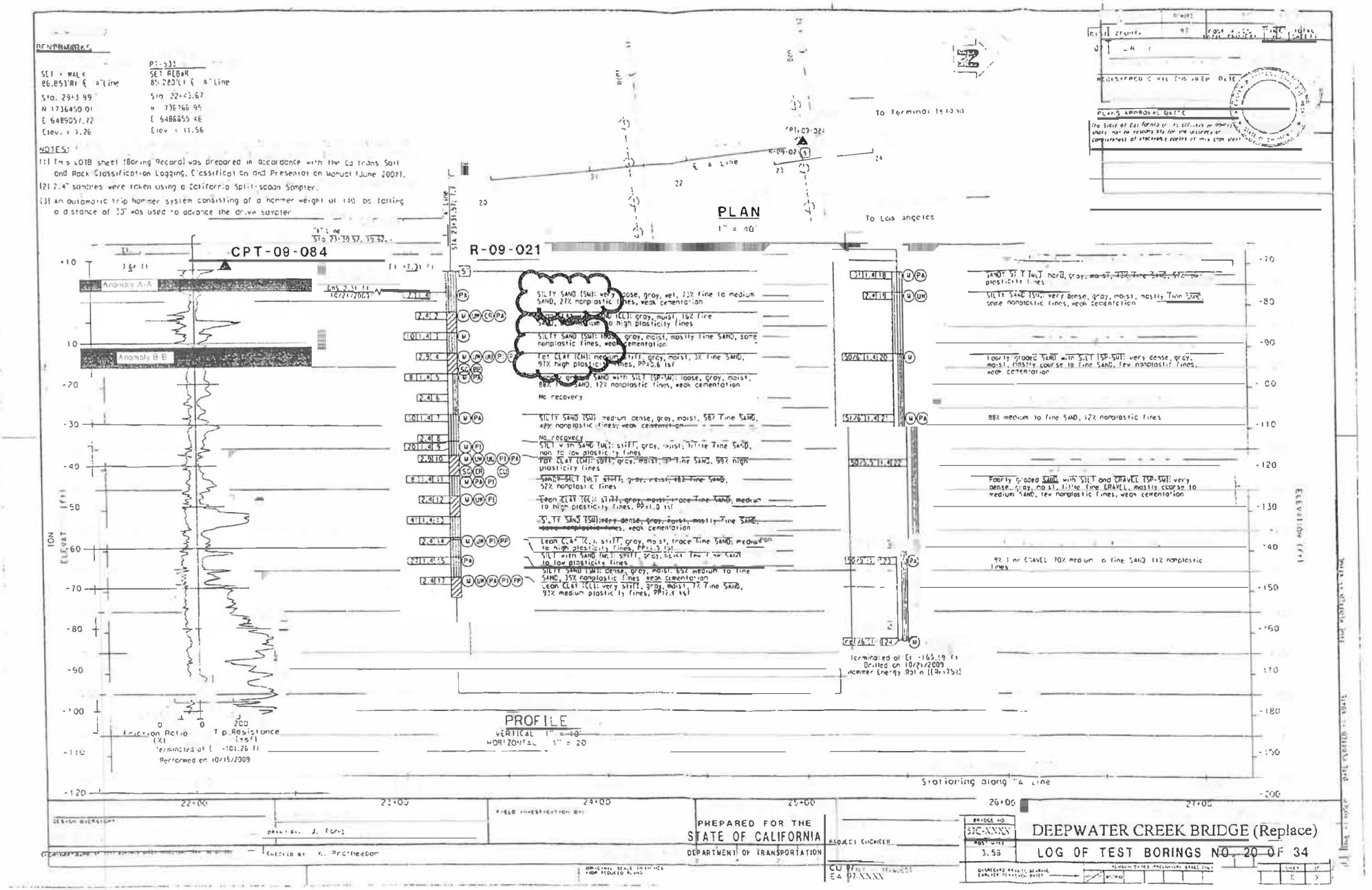

PLAN
1" = 40'
CPT-09-084
R-09-021
PROFILE
VERTICAL 1" = 10'
HORIZONTAL 1" = 20
Stationing along "A" Line
Anomaly A-A
Anomaly B-B
Friction Ratio (%)
Tip Resistance (tsf)
Performed on 10/15/2009
Drilled on 10/21/2009
PREPARED FOR THE
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DEEPWATER CREEK BRIDGE (Replace)
LOG OF TEST BORINGS NO. 20 OF 34

Figure C5

Liquefaction-Induced Downdrag

This module presents a simplified practical analysis procedure for the geotechnical seismic design of a pile foundation subjected to the effects of soil liquefaction, including liquefaction-induced downdrag. Downdrag refers to the phenomenon in which a pile (or a pile-group) foundation is subjected to *negative (downward) side shear stresses* imposed by the grounds surrounding the pile length when settling (or moving downward) more than the pile. Liquefaction-induced downdrag occurs due to ground settlement caused by soil liquefaction and affects the axial behaviors of pile foundations immediately after the cessation of shaking.

Unless permitted otherwise in a Project Specific Seismic Design Criteria (PSDC), the scope of this module is limited to the bridges designed in accordance with the Caltrans Seismic Design Criteria (SDC). It is assumed that the reader is thoroughly familiar with the Caltrans current Load and Resistance Factor Design (LRFD) of bridge foundation, including AASHTO LRFD Bridge Design Specifications with California Amendments (Caltrans LRFD BDS), Memo to Designers (MTD 3-1) and Geotechnical Manual (GM).

Caltrans' Safety Evaluation Earthquake (SEE) design requires that axially loaded pile foundations are stable against bearing capacity type failure. Noting that for SEE seismic design of bridges Caltrans uses a value of 1.0 for all load and resistance factors, based on the schematic shown in Figure 1, the general bearing stability requirements for a single pile during seismic events can be expressed as follows:

$$\left[R_N = (R_s + R_p) \right] \geq Q_{EQ} \qquad (1)$$

Where, R_s = Total pile nominal side resistance in compression from soil layer (kips)

R_p = Pile nominal tip (bearing) resistance in compression, (kips)

R_N = Total pile nominal (bearing) resistance in axial compression (kips)

Q_{EQ} = Factored seismic design load in axial compression (kips).

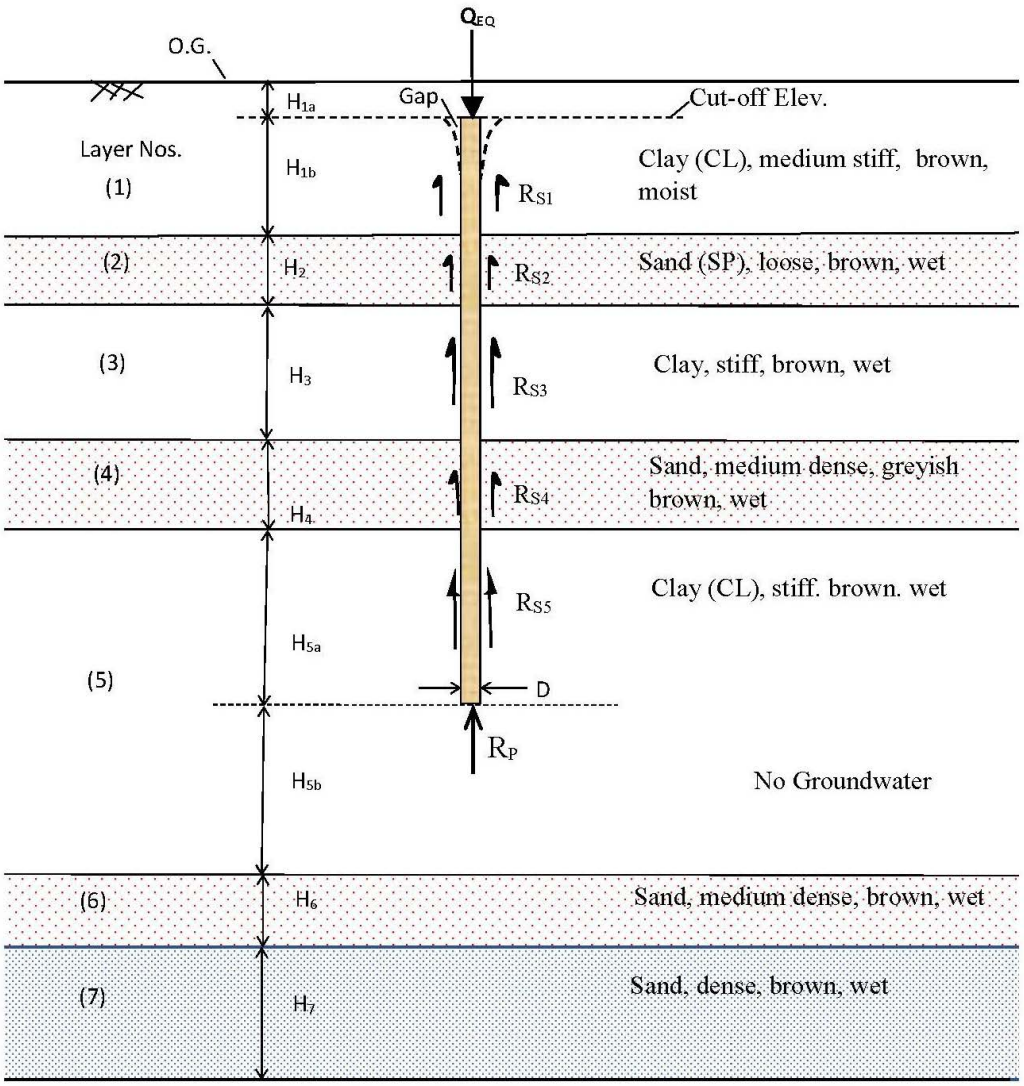
In general, the factored seismic design load in axial compression (QEQ) for a pile can be written as follows:

$$Q_{EQ} = (Q_{EQ})_{Perm} + (Q_{EQ})_I + W'p \qquad (2)$$

Where, the component (QEQ)perm is due to the permanent gravity load from the superstructure. The component, (QEQ)I, is a live load resulting from the ground motion induced inertia of the superstructure. The parameter W'p is the effective weight of the pile, a permanent load.

Based on the foundation factored design load data table in MTD 3-1, Structure Designer (SD) provides two different seismic (Extreme Event) factored design loads: a "Max. Per Pile" Load and a "Max. Permanent Per Pile" Load. As designated above, QEQ = "Max. Per Pile" load (kips) and $(Q_{EQ})_{Perm}$ = "Max. Permanent Per Pile" load (kips).

In the absence of soil liquefaction, geotechnical seismic design of an axially loaded pile in compression involves determination of pile design tip elevation (DTE) that will satisfy the bearing stability criterion expressed by Equation 1, given the maximum per pile seismic factored design loads QEQ. It requires calculations of pile nominal side and tip bearing resistances (left side of Equation 1) that are available to resist QEQ (right side of Equation 1). In MTD 3-1 this pile tip elevation is designated as the DTE for Compression (Extreme Event).



Legends: H_i = Thickness of Soil Layer i (ft), D = Pile Diameter (ft), R_{Si} = Nominal Side Resistance from Soil Layer No. i (kips), R_p= Nominal Tip Resistance (kips)

Figure 1: Schematic of a Pile Foundation for Seismic Axial Bearing Stability Design

Soil liquefaction if predicted to occur as shown in Figure 2 for the same soil profile as in Figure 1 but includes shallow groundwater conditions, must be considered when determining the pile DTE based on Equation 1.

The potential effects of soil liquefaction on the pile includes reduction in the nominal side resistance from the (1) liquefied soil layers due to reduction in the shear strength, (2) disintegrated non-liquefied soil layers, if any, overlying liquefied soils when surface manifestation of liquefaction occurs, and (5) upper part of the pile due a gap that may form between the pile and the near surface ground due to cyclic loading. This usually occurs for cohesive soils.

A reduction in the nominal tip resistance can occur if liquefied soils are present within the zone of significant influence for pile tip resistance. A liquefied soil layer is considered a soft or weak soil layer characterized by a cohesion equal to its undrained residual or steady state shear strength (Sr). For seismic downdrag design, it can be assumed that this zone of influence extends from 1.5D above to 1.5D below the pile tip elevation.

One of the most significant effects of soil liquefaction on the pile bearing stability is the imposition liquefaction-induced downdrag load. Downdrag load occurs when liquefaction-induced soil settlements cause the grounds surrounding the pile length to settle more relative to the pile. Ground settlements occur when the excess porewater pressure generated in the liquefied soil during shaking dissipates.

However, dissipation of the excess porewater pressure generated in completely liquefied soils, and hence liquefaction-induced downdrag, does not occur until the cessation of ground shaking. Therefore, effects of liquefaction-induced downdrag on the pile need not be considered in combination with the inertial component (QEQ)I that occur during shaking. Thus, in liquefied soils a pile foundation needs to be designed to satisfy the seismic axial bearing stability requirements in compression (Equation 1), for the following two different factored design load (demand) and factored pile axial resistance (capacity) combinations:

1) ***Inertial (Extreme Event-I Compression) Loading Combination:*** This load combination occurs during ground shaking when the pile is subjected to the design load component $(Q_{EQ})_I$ due to the inertia of the superstructure in addition to the ever-present design permanent load component $(Q_{EQ})_{Perm}$. The concurrently available total pile nominal bearing resistance (R_N) corresponds to the completely liquefied soil conditions. Reductions in the pile nominal side and tip resistances as discussed above need to be considered in determining R_N. The required pile tip elevation for this loading and resistance case is designated as the DTE for Compression (Extreme Event) in MTD 3-1.

2) ***Downdrag (Extreme Event -I Downdrag) Load Combination:*** This load combination occurs soon after the cessation of ground shaking when the pile is subjected to the maximum possible liquefaction-induced downdrag load, DD_{max}, in addition to the ever-present permanent load $(Q_{EQ})_{Perm}$. That is, $Q_{EQ} = [(Q_{EQ})_{Perm} + DD_{max} + W'p]$ in Equation 1. The available total pile nominal bearing resistance (R_N) corresponding to the soil conditions that exists at the time the maximum downdrag load occurs. This resistance will depend on the magnitude of the total ground settlement relative to the pile that causes downdrag, and the depths and extent of soil liquefaction. The pile tip elevation necessary to satisfy the axial bearing stability

requirement corresponding to this load and resistance combination is designated as the DTE for Compression (Extreme Event) in MTD 3-1.

A simplified step by step procedure is presented below to determine the above pile DTEs. Due to the complexities involved, the procedure is presented with the aid of a generalized schematic shown in Figure 2. It is assumed that:

- The necessary general pile design information and all LRFD factored design loads, as per MTD 3-1 are available.
- Geotechnical subsurface information necessary to perform the LRFD Service State and Strength Limit State designs and soil liquefaction hazard analysis has been obtained.
- The ground surface and groundwater elevations appropriate for seismic analysis and design have been established.
- A representative subsurface soil profile, such as that shown in Figure 2 as an example, has been developed for the pile support location.
- The pile DTEs corresponding to the LRFD Service and Strength Limit States have been evaluated and are available.

Step by Step Procedure

Step 1: Perform Soil Liquefaction Hazard Analysis

Perform soil liquefaction hazard analysis in accordance with the procedure contained in the "Liquefaction Evaluation" module of the Caltrans GM. Identify the liquefied soil layers as shown in Figure 2(a).

Step 2: Evaluate and Summarize Design Soil Parameters

The following empirical correlations proposed by Kramer and Wang (2015) can be used to evaluate the undrained residual shear strengths (S_r) of liquefied soils:

$$S_r(\text{psf}) = 2116 \exp\left\{-8.444 + 0.109(N_1)_{60} + 5.379\left(\frac{\sigma'_{vo}}{2116}\right)^{0.1}\right\} \qquad (3)$$

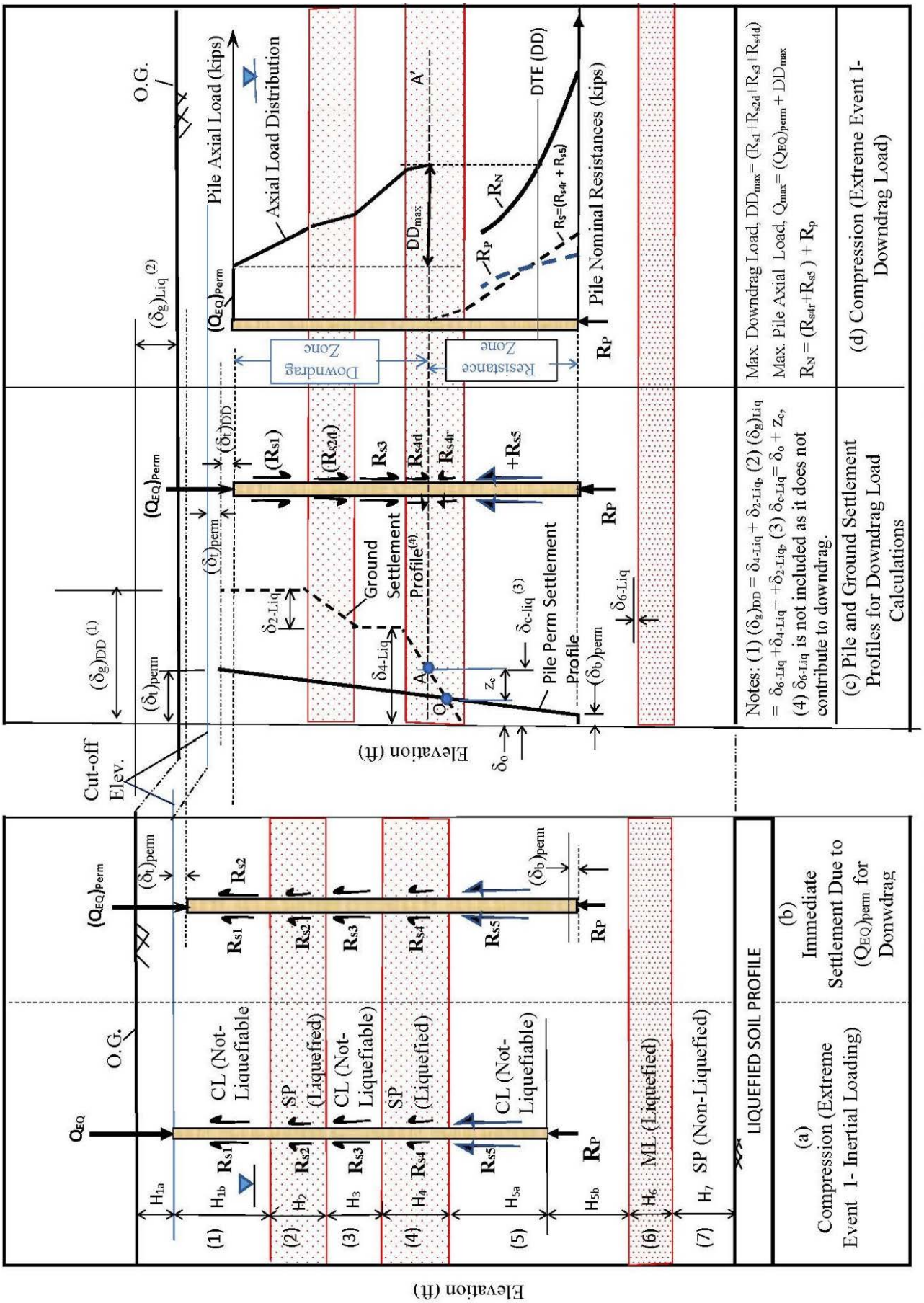


Figure 2. Seismic Design of a Pile Foundation at Liquefied Sites (Schematic).

Where, $(N_1)_{60}$ = energy corrected and overburden pressure normailzed SPT blow count at the depth under consideration. The field measured SPT blow count (N_m) at a given depth is corrected to standard 60% hammer engery and normailzed to an initial effective overburden stress equal to one (1.0) atmohspheric pressure ($\cong$ 2116 psf). See "Liquefaction Module" of the Geotechncial Manual for a detailed calculation procedure for $(N_1)_{60}$. Note that no fine content correction is used in Eqaution 3. And, σ'_{vo} = initial in-situ effective overburden stress at the depth under consideration evaluated based on the ground surface and groundwater elevations applicable to seismic design (Extreme Event-I Limit State).

The empirical correlation proposed by Robertson (2010) can be used to evaluate the undrained residual shear strengths (Sr) of liquefied soils using CPT data.

For non-liquefiable clayey soils, assume ϕ=0 condition and use undrained shear strength (S_u). For non-liquefied coarse-grained and fine-grained soils with plasticity index (PI)<7, both cohesion (c) and friction angle (ϕ) may be used for seismic design when these parameters are measured directly for the in-situ state conditions using appropriate field or laboratory test. Cohesion should not be used for these soils if the friction angle is estimated indirectly, such as using empirical correlations with SPT or CPT data. For non-liquefied soils, the current practice is to use the same shear strength parameters in the seismic analysis as those used in the analyses performed to determine the pile DTEs for the LRFD Strength Limit State.

Summarize the assigned design soil parameters, including soil total/effective unit weight and the shear strength parameters, in a table.

Step 3: Designing for the Inertial Load Combination, QEQ = (QEQ)Perm + (QEQ)I +W'P

Step 3-1: Calculate Pile Nominal Side and Tip Resistances

Calculate pile nominal side and tip bearing resistances for the liquefied soil profile, such as the one shown in Figure 2(a), based on the design soil parameters determined and assigned in Step 2. See the deep foundation modules in the Caltrans' Geotechnical Manual for details on how to calculate pile nominal side and tip bearing resistances for a given soil profile once the appropriate soil design parameters are assigned to the soil layers.

Step 3-2: Evaluate Potential for Surface Manifestation of Liquefaction

For the soil layers identified as liquefiable, evaluate starting from the top whether the combined thickness of the overlying soil layers is adequate to prevent surface manifestation of liquefaction based on Figure 3 (Modified after Ishihara,1985). See Figure 4 for the definitions and symbols to be used for the overlying soil layer thickness (Hs) and the liquefied soil layer thickness (HL) in Figure 3. Identify the lowest liquefied soil layer that is likely to cause surface manifestation of liquefaction.

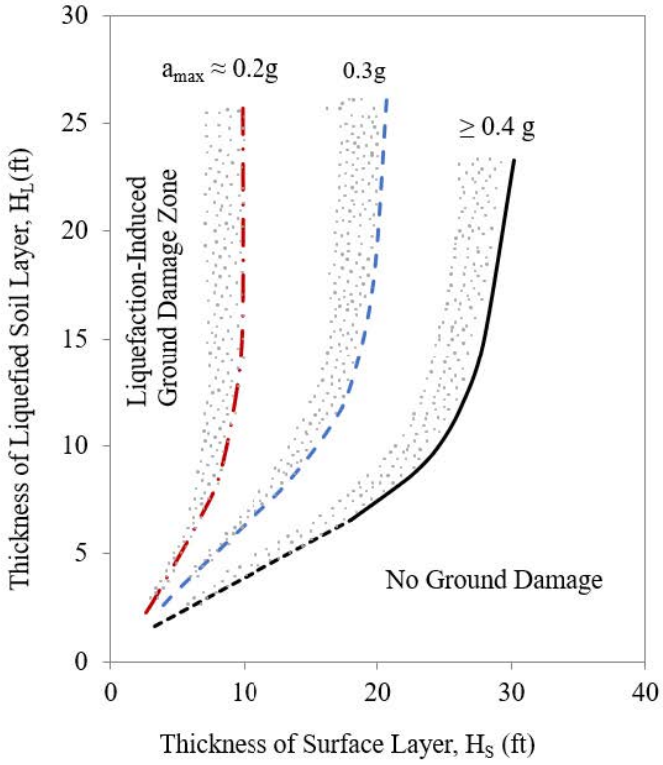


Figure 3. Boundaries for Surface Manifestation of Soil Liquefaction (Modified After Ishihara, 1985)

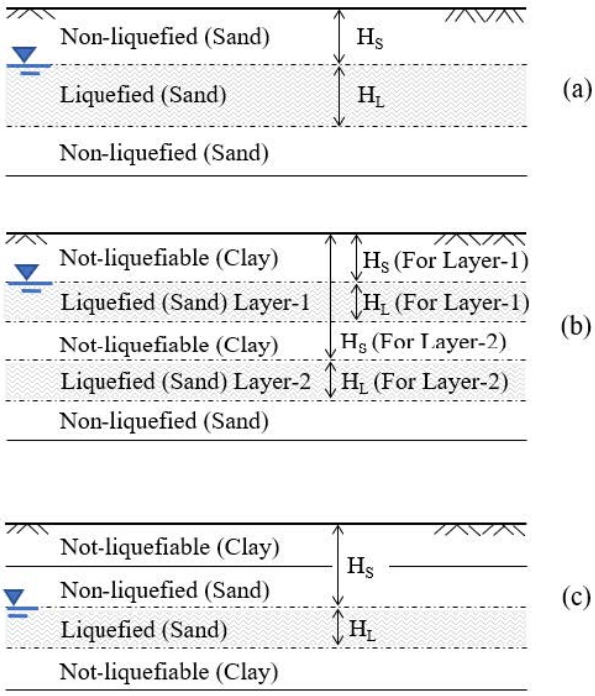


Figure 4. Definitions for Thickness of Surface Layer (H_s) and Thickness of Liquefiable Sand Layer (H_L) in Figure 3

Step 3-3: Determine Pile DTE for Compression (Extreme Event I - Compression)

Determine the pile DTE for Compression (Extreme Event) or the "Inertial Loading Combination", based on the seismic factored design load Q_{EQ} and the available pile total nominal resistance (R_N) calculated using the pile nominal side and tip bearing resistances from Step 3-1, except:

1) Ignore the pile nominal resistances calculated for the upper one pile diameter length of the pile if located in cohesive soils.
2) If surface manifestation of liquefaction is predicted in Step 3-2, ignore all pile nominal resistances from the lowest causative liquefied soil layer and the overlying soil layers, if any.

Step 4: Designing for Downdrag Load Combination, $Q_{EQ} = (Q_{EQ})_{Perm} + DD_{max} + W'_p$

The purpose of this step is to determine the pile DTE for Compression (Extreme Event-Downdrag), as designated in MTD 3-1. This DTE will be referred to hereafter simply as DTE(DD).

Settlements of the soil layers above the pile cut-off elevation or below the pile tip settlement zone of influence do not affect downdrag and need not be considered in the following analysis. Such ground settlements, if any occurs, will contribute to the total

ground surface and bridge support settlement, and may also contribute to the differential settlements between adjacent supports. The objective of the following analysis, however, is to analyze and design the pile foundation for axial bearing stability in compression only.

It is not necessary to consider live load in downdrag analysis. Once downdrag load occurs, it is considered a permanent axial load on the pile. Use a load factor of 1.0 for liquefaction-induced downdrag load.

Step 4-1: Select the Preliminary Pile Tip Elevation for Downdrag Load Analysis

Determine the pile DTE that would be the minimum (i.e., shall not be raised above) tip elevation based on all other applicable design requirements (i.e., Service, Strength and Extreme Event -Compression), as per MTD 3-1, except, if not available, the DTE for lateral load. Use this minimum pile DTE as the preliminary pile tip elevation to evaluate the maximum downdrag load (DD_{max}) and the DTE(DD) as per the following steps.

Step 4-2: Determine Pile Settlement at the Onset of Ground Settlement

With the pile tipped at the lowest DTE as determined in Step 4-1, and for the same liquefied soil profile and assigned soil parameters as used in Step 3, estimate the pile immediate settlements (pseudo-static) of the pile top and tip, $(\delta_t)_{perm}$ and $(\delta_b)_{perm}$, respectively, due to the pile axial load $(Q_{EQ})_{perm}$ applied at the pile top, as shown in Figure 2(b). In determining these pile settlements, ignore any resistances to pile settlement from the soil layers that were ignored in determining the pile DTE for Compression (Extreme Event - I) in Step 3.

Pseudo-static pile load-immediate settlement curves can be developed using available computer programs for soil conditions represented by Figure 2(b) to estimate $(\delta_t)_{perm}$ and $(\delta_b)_{perm.}$ For additional information, see specifications and commentaries related to the immediate settlement of bridge foundations in Section 10 of Caltrans LRFD BDS.

Step 4-3: Estimate Liquefaction-Induced Ground Settlements

Estimate liquefaction-induced settlement of the identified liquefied soil layers in accordance with the procedures recommended by Tokimatsu and Seed (1987). Zhang et al (2002) may be used to estimate liquefaction-induced ground settlement using CPT data. In Figure 2(c), $\delta_{2\text{-}Liq}$, $\delta_{4\text{-}Liq}$ and $\delta_{6\text{-}Liq}$ represents the estimated total settlements of the liquefied soil layers 2, 4 and 6, respectively.

Liquefaction-induced total ground settlement, $(\delta_g)_{Liq,}$ at the ground surface elevation is equal to the sum of the total settlements of all the liquefied soil layers. In Figure 2(c), $(\delta_g)_{Liq} = [\delta_{2\text{-}Liq}) + \delta_{4\text{-}Liq} + \delta_{6\text{-}Liq}]$. However, settlement of the liquefied soil layer 6 does not contribute to the development of downdrag load on the pile. As shown in Figure 2(c), the settlements of the soil layers 2 and 4 only can cause downdrag load on the pile. Thus, the total ground settlement at the pile cut-off elevation that contributes to the development of downdrag load on the pile, $(\delta_g)_{DD} = (\delta_{2\text{-}Liq} + \delta_{4\text{-}Liq})$.

Step 4-4: Plot Pile and Ground Settlement Profiles

Plot pile settlement profile as shown in Figure 2(c) based on $(\delta_t)_{perm}$ and $(\delta_b)_{perm}$ determined in Step 4-2. Also plot the ground settlement profile based on the settlements evaluated in Step 4-3 based on the settlement of the liquefied soil layers located within the pile embedment length (i.e., Layers 2 and 4 only in Figure 2(c). Unless noted otherwise, ground settlements in the following steps refer to this ground settlement profile.

Step 4-5: Determine the Location of the Maximum Downdrag Load (DDmax)

Calculate the pile-ground relative settlement z_{max} at which the maximum negative side shear stresses will be developed on the pile. The parameter z_{max} depends on several factors, including the pile type, diameter, length, axial stiffness, and the soil type and state (e.g., API, 2014, O'Neil and Reese, 1999; Karlsrud, 2014, Coyle and Sullivan, 1967). For evaluating the maximum downdrag load on the pile, calculate the parameter z_{max} based on the simplified correlations in Table 1.

As shown in Figure 2(c), determine the ground settlement, δ_o, at Point O, where the pile settlement and the ground settlement profiles intersect each other. Determine the critical ground settlement, $\delta_{c\text{-}Liq}$, by adding δ_o and z_{max}. This is the total ground settlement above which the negative skin friction will be mobilized on the pile. If the total ground settlement at the surface elevation, $(\delta_g)_{DD}$, is less than or equal to $\delta_{c\text{-}Liq}$, it can be assumed that the pile will not experience any downdrag load, and no further analysis is necessary.

Draw a horizontal line AA' passing through the Point A. For design, assumed that the portion of the pile embedded in soil above the elevation of the line AA' will be subjected to the maximum negative side shear stresses or downdrag load.

Table 1: Parameter z_{max} for Various Pile and Soil Types

Pile Type	Soil Type	$\frac{z_{max}}{D}$
Driven-Closed End	Sand or Clay	0.03
Driven – Open Ended	Clay	0.02
	Sand	0.03
Drilled Shafts	Clay	0.008
	Sand	0.01

Step 4-6: Calculate Maximum Downdrag Load (DDmax)

Calculate the maximum negative side shear stresses that can be mobilized along the pile embedment length above the elevation of line AA'. Use the same shear strength parameters as those assigned in Step 2, except for the liquefied soil layers.

The maximum negative side shear stress from liquefied soils will not occur until the excess porewater pressures generated during ground shaking completely dissipate.

Thus, downdrag loads from the liquefied layers should be calculated based on the shear strengths of the resettled liquefied soils. In the absence of measured shear strength parameters, the maximum downdrag load from the resettled loose liquefied soils can be calculated using:

- c=0 and ϕ=30^o for coarse-grained liquefied soil (e.g., SW, SP, SM) or,
- c=0 and ϕ=25^o for fine-grained liquefied soils (e.g., ML)

Determine the nominal downdrag loads from each of the soil layers located above line AA' based on the calculated maximum (or nominal) negative side shear stresses.

Calculate the total axial load on the pile at any depth (or elevation) below the pile cut-off elevation as the sum of $(Q_{EQ})_{perm}$ and the total downdrag load at the same depth (or elevation).

Plot the total pile axial load as a function of the elevation and calculate the maximum total downdrag load, DD_{max}, on the pile which occurs at the elevation of the line AA'.

In Figure 2(d), $DD_{max} = R_{s1}+R_{s2d}+R_{s3}+R_{s4d}$. Here the subscript "d" is used to denote the downdrag loads from the liquefied soil layer.

The maximum axial load on pile is equal to $[(Q_{EQ})_{perm} + DD_{max}+W'_p)$, which occurs at the same elevation as the maximum downdrag load, DD_{max}.

Step 4-7: Calculate Pile Nominal Resistances (RN)

Calculate the available pile nominal side and tip bearing resistances from the soil layers below line AA' based on the soil shear strength parameters assigned in Step 2, except for liquefied soils. For the resettled liquefied soils, use the shear strength parameters recommended in Step 4-6.

Plot the total pile nominal side resistance, Rs), and the total pile nominal tip resistances, R_P, as a function of elevation as shown in Figure 2(d).

Calculate the available total pile nominal bearing resistance, R_N, in axial compression by adding R_S and R_P. Plot R_N as a function of elevation, as shown in Figure 2(d).

Step 4-8. Determine pile DTE for Compression (Extreme Event -Downdrag)

Determine the pile DTE(DD), as the elevation at which $R_N = [(Q_{EQ})_{perm} + DD_{max} + W'_p]$. If the DTE (DD) is located at or above the preliminary tip elevation used in the above downdrag load calculations, no further analysis is necessary.

If R_N at the pile preliminary tip elevation is less than the factored load, $[(Q_{EQ})_{perm} + DD_{max} W'_p]$, extend calculations of R_S and R_P from underlying the preliminary tip the calculated R_N value exceeds the total factored seismic design load. Determine the DTE(DD) based on $R_N = [(Q_{EQ})_{perm} + DD_{max}+ W'_p]$.

Reporting

As per MTD 3-1, the following information need to be included in the Foundation Report for Extreme Event-I Limit State design:

- Extreme Event (Compression) case: Geotechnical Nominal Resistance Per Pile and Nominal Tip Resistance (Compression), and the corresponding DTE (Extreme Event)
- Extreme Event (Downdrag) case: Geotechnical Nominal Resistance Per Pile and Nominal Tip Resistance (Compression with Downdrag), and the corresponding DTE (Extreme Event-Downdrag)
- Maximum downdrag load per pile (DD_{max})
- The downdrag zone, i.e., the top and bottom elevations of the downdrag zone
- Ground settlement $(\delta_g)_{DD}$ at the pile cut-off elevation due to the settlement of liquefied soil layers located within the pile embedment length
- The total ground settlements $(\delta_g)_{Liq}$ at the ground surface elevation due to all liquefied soil layers

If requested by the Structure Designer (SD), provide data for the unit side resistance (t) versus pile-soil relative settlement (z) or t-z curves, and the unit tip bearing resistance (q) versus tip settlement (w) or q-w curves for both the inertial loading (Step 3) and liquefaction-induced downdrag loading (Step 4) combinations. Figures 2(a) and 2(d) represent the corresponding soil profiles and conditions, respectively.

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Appendix A

Liquefaction Induced Downdrag Example Problem

Limit State: Extreme Event -I (Seismic)

Appendix A presents an example geotechnical analysis and design of a pile foundation subjected to liquefaction-induced downdrag. The example involves designing a Cast-in-Drilled Hole (CIDH) pile or drilled shaft supporting a bridge bent. Figure A1 presents a schematic illustration of the example.

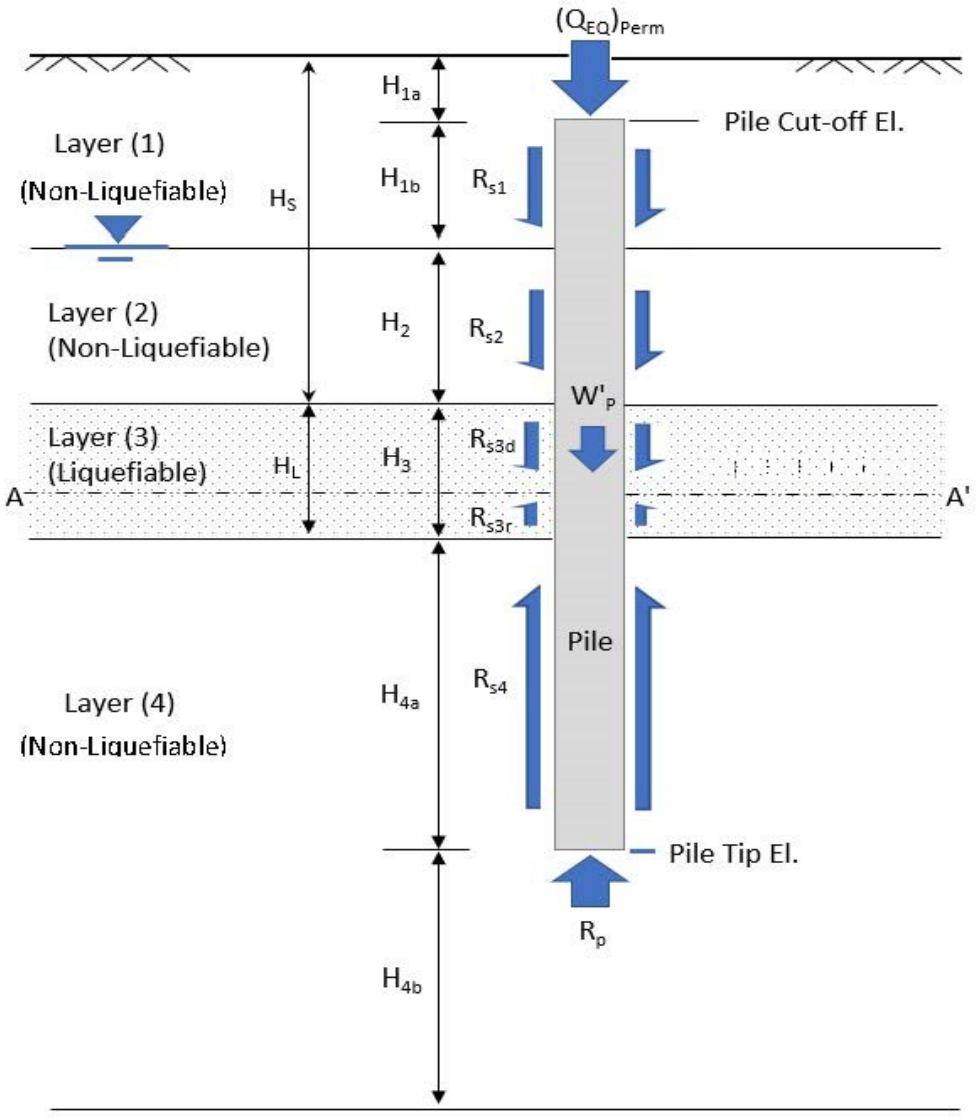


Figure A1: Components of Pile Foundation Design for Liquefaction-Induced Downdrag

Notations and definitions are:

H_i : thickness of the soil layer no. i
H_S : overlying soil layer thickness for surface manifestation of liquefaction
H_L : liquefied soil layer thickness for surface manifestation of liquefaction
A-A′: maximum downdrag load elevation due to liquefaction-induced ground settlement
R_{si} : nominal side resistance along the pile within the soil layer i.
R_{sid} : nominal side resistance along the pile above A-A' line within the liquefied soil layer i.
R_{sir} : nominal side resistance along the pile below A-A' line within the liquefied soil layer i.
R_p : nominal tip resistance
W'_P: effective weight of the pile
$(Q_{EQ})_{Perm}$: maximum permanent load per pile.

The pile properties, the general foundation design information and the factored design loads provided by the Structure Designer (SD) are:

- Pile type: CIDH
- Pile Diameter: 66-inch
- Unit weight of the CIDH pile: 150 pcf
- Elastic Modulus for Concrete: $\mathrm{E} = 3.6 \times 10^6$ psi

Table A1: General Foundation Design Information

Support Location	Pile Type	FG Elev. (ft)	Pile Cut-off Elevation (ft)	Pile Cap Size		Permissible Settlement under Service Load (in)	Number of Piles per Support
				B	L		
Bent	66-inch dia. CIDH	+0.0	- 5.0	N/A	N/A	1.0	1

Table A2: Foundation Factored Design Loads

Support Location	Service-1 Limit State (kips)		Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)				
	Total Load Per Support	Perm. Load Per Support	Compression		Tension		Compression			Tension	
			Per Support	Max Per Pile	Per Support	Max Per Pile	Per Support	Max Per Pile	Max Perm Per Pile	Per Support	Max Per Pile
Bent	1115	886	1860	1860	0	0	1135	1135	886	0	0

Step by Step Procedure

This example problem demonstrates the step-by-step procedure for the design of pile foundations subject to liquefaction-induced downdrag.

Step 1: Perform Soil Liquefaction Hazard Analysis

The design ground motion parameters corresponding to a 975-year return period for soil liquefaction evaluation are as follows:

- Design horizontal Peak Ground Acceleration (PGA) = 0.7g
- Moment magnitudue of the design (mean) earthquake event, M =7.3

Based on the procedure in the *Liquefaction Evaluation* module, it is indicated the soil layer from 20 feet to 30 feet below the ground surface is liquefiable during the design ground motion event (Figure A2).

Step 2: Evaluate and Summarize Design Soil Parameters

The soil profile and the relevant engineering soil parameters are summarized in Table A3 based on available existing geotechnical data, a current site investigation performed and interpretation of field and laboratory test data. A representative idealized cross section of the design soil profile including the pile is shown in Figure A2.

Table A3: Design Soil Parameters for the Example Downdrag Analysis

Layer No.	Elevation Range (feet)	Soil Description	$(N_1)_{60}$	Total Unit Weight, γ_t (pcf)	Shear Strength Parameters	
					Friction Angle, Φ (degrees)	Cohesion (c) or Undrained Shear Strength (S_u or S_r) (psf)
1	0 to -10	Silty Sand (SM), Medium dense, fine-to coarse-grained sands	20	120	34	c = 100
2	-10 to -20	Silty Clay (CL), Soft to medium stiff	-	108	0	S_u = 1500
3	-20 to -30	Sand (SP), fine to medium-grained, loose to low medium dense, 9% fines, non-plastic, wet, (Liquefied Layer)	12	110	32	c = 0
					0	S_r = 340
4	-30 to -80	Silty Sand (SM) with gravel, dense, fine to coarse-grained sand, fine gravel	36	130	38	c = 0

Note: S_r represents undrained residual or steady state shear strength of liquefied soil.

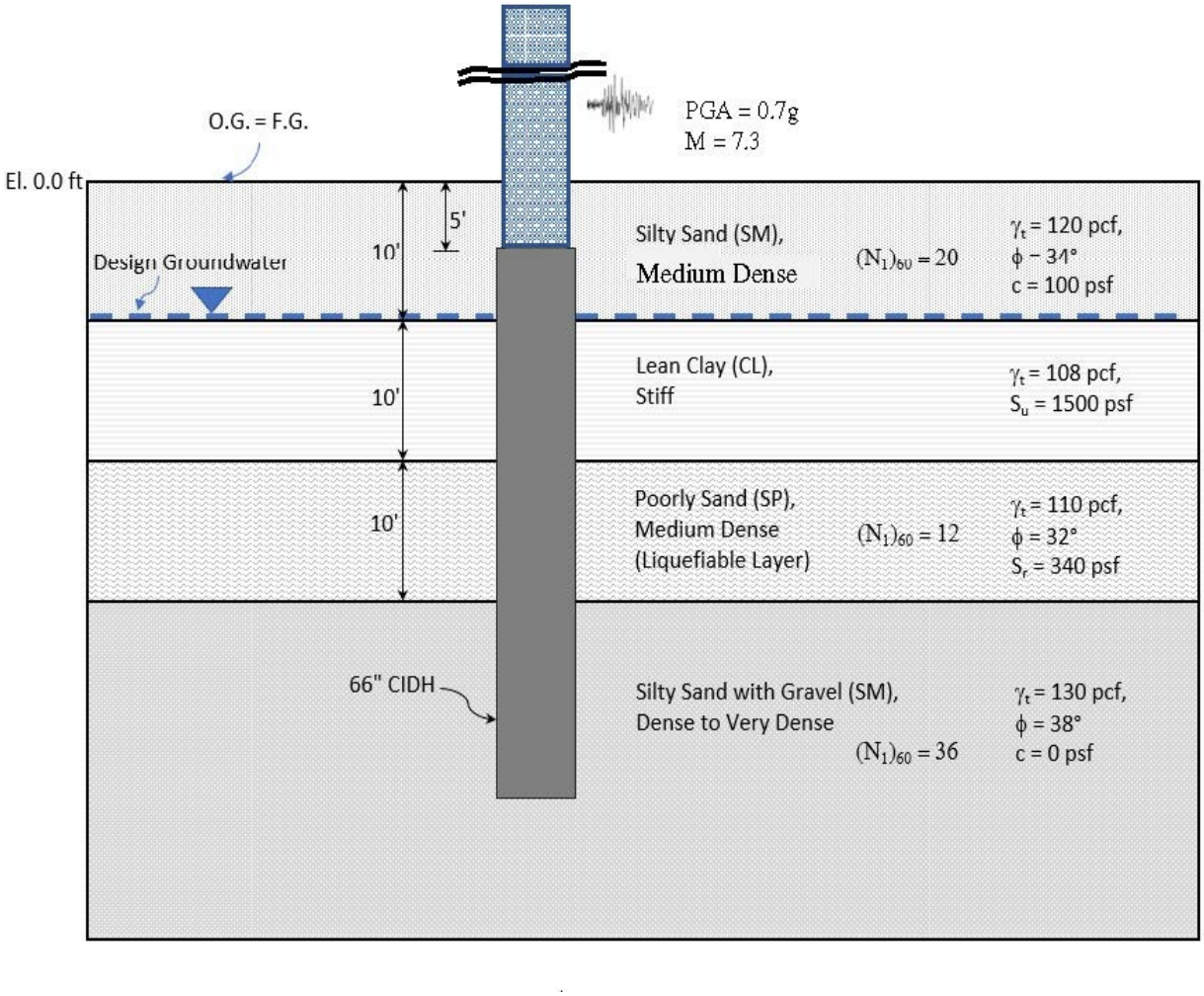


Figure A2: Soil Profile and Pile Model for Liquefaction-Induced Downdrag Analysis

Step 3: Designing for the Inertial Loading Combination (Extreme Event-I Compression)

Step 3-1: Evaluate Potential for Surface Manifestation of Liquefaction

From the soil profile in Figure A2, the overlying non-liquefiable soil layers (H_S) is 20 feet thick, and the liquefiable soil layer (H_L) is 10 feet thick. Based on Figure A3, the two overlying non-liquefiable layers are not thick enough to prevent surface manifestation of liquefaction. Therefore, pile nominal resistances from these two overlying layers and the liquefiable layer are ignored in the pile axial capacity analysis under this loading case.

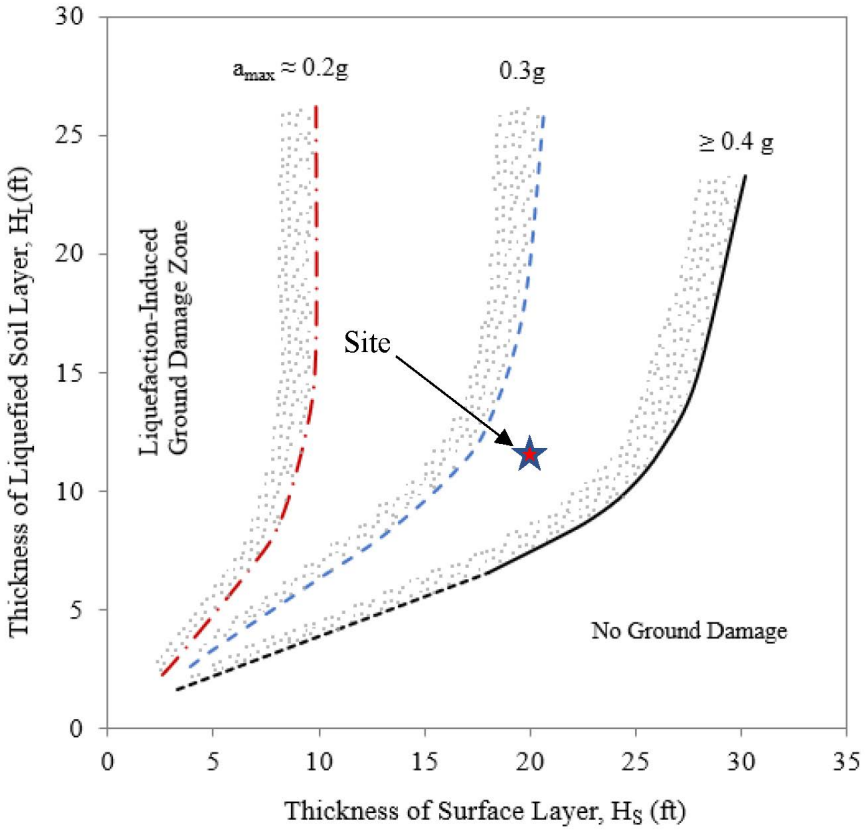


Figure A3: Evaluation of Surface Manifestation (Modified After Ishihara 1985)

Step 3-2: Calculate Pile Nominal Side and Tip Resistances

Based on the design soil parameters summarized in Table A3, the pile nominal side and tip resistances under the extreme event limit state are calculated using SHAFT software (Ensoft, 2012). The outputs using SHAFT are summarized in Table A4 starting from the pile embedment of length of 26 feet (= 31 feet – Depth to pile cut-off elevation 5.0 feet). Note that "LENGTH" in the Table A4 refers to pile length from the finish grade.

Table A4: Summary of Predicted Outputs from SHAFT (Extreme Event-I Limit State)

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	LRFD QS (TONS)	LRFD QB (TONS)	LRFD QU (TONS)
31.0	27.28	13.68	320.21	333.89	13.68	320.21	333.89
32.0	28.16	27.57	328.59	356.16	27.57	328.59	356.16
33.0	29.04	41.66	336.97	378.63	41.66	336.97	378.63
34.0	29.92	55.94	345.35	401.29	55.94	345.35	401.29
35.0	30.80	70.40	353.73	424.13	70.40	353.73	424.13
36.0	31.68	85.03	362.10	447.14	85.03	362.10	447.14
37.0	32.56	99.82	370.48	470.31	99.82	370.48	470.31
38.0	33.44	114.76	378.86	493.62	114.76	378.86	493.62
39.0	34.32	129.84	387.24	517.08	129.84	387.24	517.08
40.0	35.20	145.05	395.62	540.67	145.05	395.62	540.67
41.0	36.08	160.38	404.00	564.38	160.38	404.00	564.38
42.0	36.96	175.83	412.38	588.20	175.83	412.38	588.20
43.0	37.84	191.37	420.75	612.13	191.37	420.75	612.13
44.0	38.72	207.02	429.13	636.15	207.02	429.13	636.15
45.0	39.60	222.74	436.64	659.38	222.74	436.64	659.38
46.0	40.48	238.55	443.20	681.74	238.55	443.20	681.74
47.0	41.36	254.42	448.73	703.15	254.42	448.73	703.15
48.0	42.24	270.35	453.18	723.52	270.35	453.18	723.52
49.0	43.12	286.33	456.46	742.78	286.33	456.46	742.78
50.0	44.00	302.35	458.64	760.99	302.35	458.64	760.99
51.0	44.88	318.41	459.95	778.36	318.41	459.95	778.36
52.0	45.76	334.49	460.61	795.10	334.49	460.61	795.10
53.0	46.64	350.60	460.83	811.42	350.60	460.83	811.42
54.0	47.52	366.71	460.83	827.53	366.71	460.83	827.53
55.0	48.40	382.82	460.83	843.64	382.82	460.83	843.64
56.0	49.28	398.92	460.83	859.75	398.92	460.83	859.75
57.0	50.16	415.01	460.83	875.84	415.01	460.83	875.84
58.0	51.04	431.08	460.83	891.91	431.08	460.83	891.91
59.0	51.92	447.12	460.83	907.95	447.12	460.83	907.95
60.0	52.80	463.12	460.83	923.95	463.12	460.83	923.95

Step 3-3: Determine Pile Design Tip Elevation (DTE) for Extreme Event-I (Compression)

In this example the upper 25 feet of the CIDH pile has no nominal side resistances because of the surface manifestation of liquefaction (Step 3-1) and the 10 feet thick liquefied soil layer. When surface manifestation occurs, it is assumed that the overlying non-liquefied soils are disintegrated along the embedded pile length to provide any side resistance. Plots of the pile side, tip and total factored nominal resistances versus lengths (from the cut-off) are shown in Figure A4.

Since the effective pile weight is considered in the load combination, use of a trial and error method is necessary to determine the DTE.

For first trial, the maximum factored design load per pile from Table A2 (1145 kips rounded to the nearest 10 kips) will be used to estimate the preliminary DTE. Based on Figure A4, for this factored design load, the preliminary DTE is about -42 feet (pile length = 37 feet).

The effective weight for the 37 feet of pile = 92 kips. The revised factored design load (1135 kips +92 kips) = 1227 feet. Rounded to the nearest 10 kips, the factored design load = 1230 kips. As shown in Figure A4, the estimated DTE for this factored load is -43.5 feet (pile length = 38.5 feet).

For the above pile DTE, the ratio of the pile embedment depth into the bearing strata (measured from the bottom of the liquefied soil layer) to the pile diameter is 3.0. This ratio, being greater than 1.5, indicate no reduction is required in the pile tip nominal resistance due to the liquefied soil layer. More accurately, based data in Table A4, for the pile tip elevation – 43.5 feet, the estimated available total geotechnical nominal resistance per pile, RN = 1250 kips, and the corresponding estimated pile tip nominal resistance component, Rp= 850 kips.

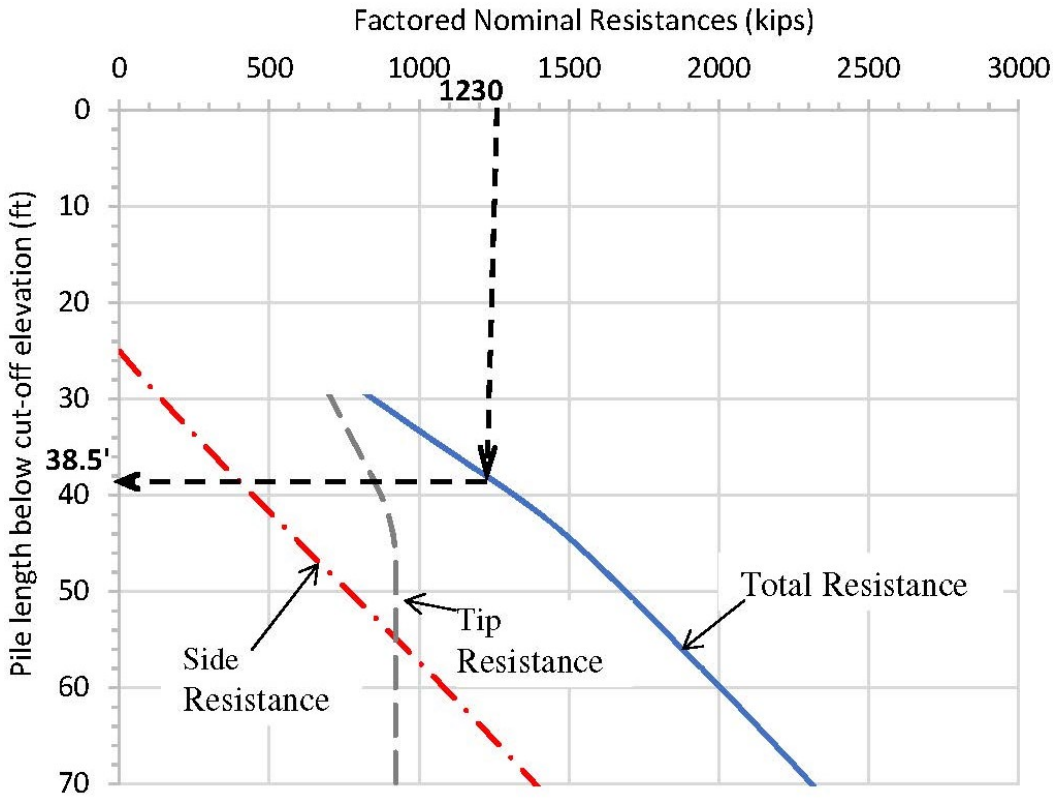


Figure A4: Factored Nominal Resistances with Pile Length for Extreme Limit State

Step 4: Designing for Downdrag Load Combination (Extreme Event I- Downdrag)

Step 4-1: Select the Preliminary Pile Tip Elevation for Downdrag Load Analysis

Based on the pile design tip elevations evaluated for Service and Strength limit states, and the above Extreme Event-I (Compression) design, the minimum (i.e., not to be raised above) tip elevation would be at -79 feet. (pile length = 74 feet. This design tip evaluation (-79 feet) is associated with the Strength Limit State. Note the DTE for lateral load should also be considered in this step, if available. However, in this example case, the lateral design tip elevation is not expected to be lower than -79 feet. This tip elevation will be used as the preliminary pile tip elevation for the subsequent downdrag analysis.

Step 4-2: Calculate Pile Settlements at Onset of Liquefaction-Induced Ground Settlement

Based on the preliminary tip elevation of -79 feet from Step 4-1, and the soil parameters used in Step 3, Table A.5 presents SHAFT output data for the pile axial load at the top versus pile top settlement, and the transferred axial load at the pile tip versus pile tip settlement. The corresponding plots are presented in Figure A5. Due to the seismic design permanent load of 890 kips per pile (886 kips in Table A2, rounded to the nearest 10 kips), the estimated pile top settlement $(\delta_t)_{perm}$ = 0.20 inches and the corresponding pile tip settlement $(\delta_b)_{perm}$ = 0.13 inches.

Table A5: Pile Axial Load-Settlement (Averaged) resulted from SHAFT

RESULT FROM TREND (AVERAGED) LINE

TOP LOAD ton	TOP MOVEMENT IN.	TIP LOAD ton	TIP MOVEMENT IN.
0.4315E-01	0.1455E-04	0.2444E-02	0.1000E-04
0.2157E+00	0.7277E-04	0.1222E-01	0.5000E-04
0.4315E+00	0.1455E-03	0.2444E-01	0.1000E-03
0.2157E+02	0.7277E-02	0.1222E+01	0.5000E-02
0.3236E+02	0.1092E-01	0.1833E+01	0.7500E-02
0.4315E+02	0.1455E-01	0.2444E+01	0.1000E-01
0.1081E+03	0.3640E-01	0.6109E+01	0.2500E-01
0.2164E+03	0.7282E-01	0.1222E+02	0.5000E-01
0.3081E+03	0.1077E+00	0.1833E+02	0.7500E-01
0.3863E+03	0.1411E+00	0.2444E+02	0.1000E+00
0.6456E+03	0.3203E+00	0.6109E+02	0.2500E+00
0.8206E+03	0.5917E+00	0.1210E+03	0.5000E+00
0.8672E+03	0.7585E+00	0.1590E+03	0.6600E+00
0.1001E+04	0.1769E+01	0.2949E+03	0.1650E+01
0.1172E+04	0.3445E+01	0.4677E+03	0.3300E+01

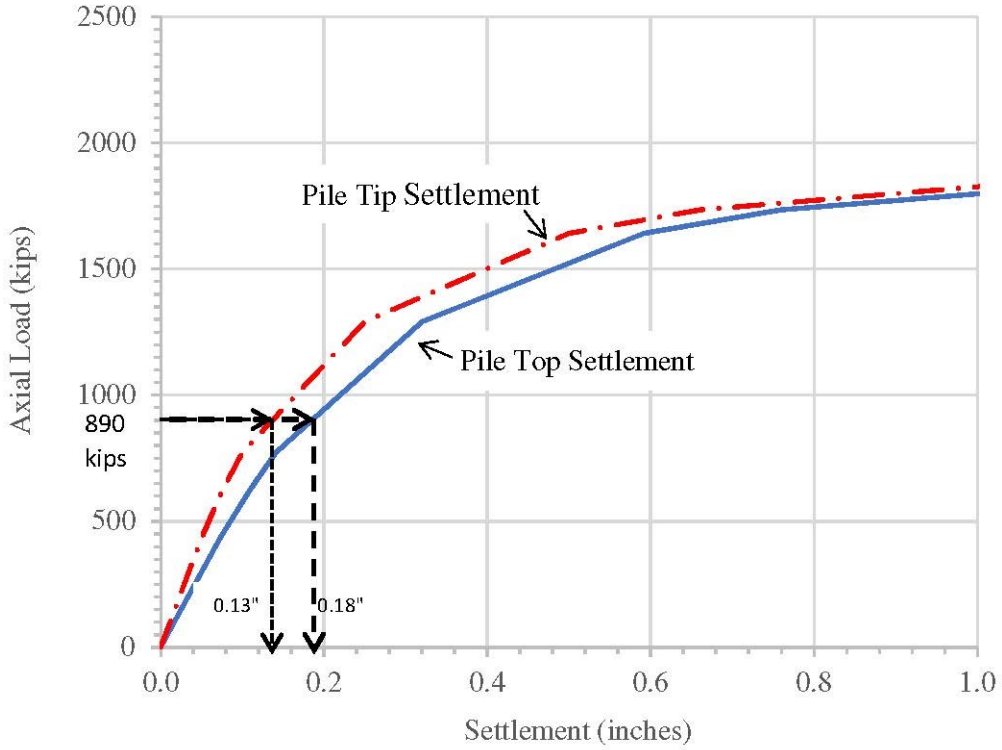


Figure A5: Pile Load-Settlement Curves

Step 4-3: Calculate Liquefaction-Induced Ground Settlements

Liquefaction-induced ground settlement for the liquefied soil layer (No. 3), $\delta_{3\text{-Liq}}$ was calculated using SPT blow counts and using the procedure in Tokimatsu and Seed (1987).

At the depth of 25 feet below the ground surface:

Total vertical overburden stress, σ_o(psf)

$$= 120 \text{ pcf} \times 10 \text{ ft} + 108 \text{ pcf} \times 10 \text{ ft} + 110 \text{ pcf} \times 5 \text{ ft} = 2830 \text{ (psf)}$$

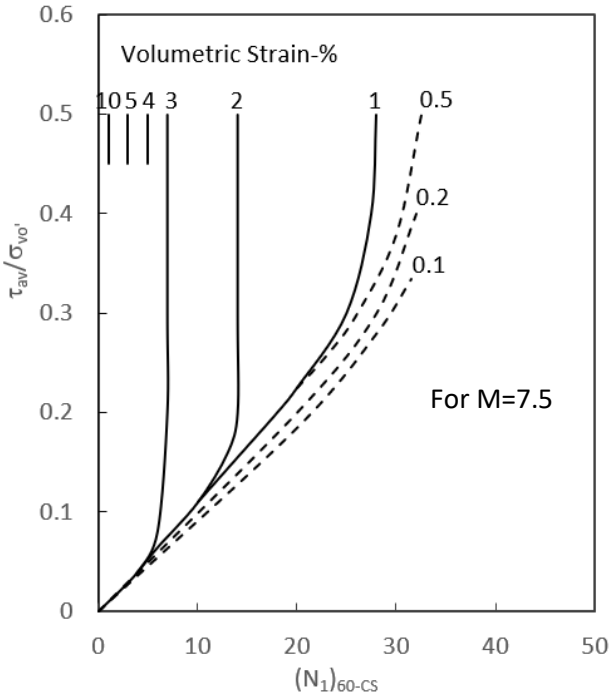
Effective vertical overburden stress, σ'_o(psf)

$$= 120 \text{ pcf} \times 10 \text{ ft} + (108 - 62.4) \text{ pcf} \times 10 \text{ ft} + (110 - 62.4) \text{ pcf} \times 5 \text{ ft}$$

$$= 1894 \text{ (psf)}$$

Stress reduction factor due to soil flexibility, r_d = 0.942 (Youd et. al, 2001)
Clean sand equivalent normalized SPT blow count, $(N_1)_{60\text{-CS}}$ = 13

Cyclic Shear Stress Ratio, $\left(\frac{\tau_{av}}{\sigma'_o}\right)_{M=7.5} = 0.65 \frac{a_{max}}{g} \frac{\sigma_o}{\sigma'_o} r_d \frac{1}{r_m} = 0.65 \times \frac{0.7\,g}{g} \times \frac{2830 \text{ psf}}{1894 \text{ psf}} \times 0.942 \times \frac{1}{r_m} = \frac{0.64}{r_m} 0.62$



Earthquake magnitude, M	Scaling factor for stress ratio, r_m
8.50	0.89
7.50	1.00
6.75	1.13
6.00	1.32
5.25	1.50

Figure A6: Determination of Volumetric Strain of Liquefied Soils (Modified after Tokimatsu and Seed, 1987)

Step 4-4: Plot Pile and Ground Settlement Profiles

Figure A7 shows the pile settlement (Step 4-2) and liquefaction-induced ground settlement (Step 4-3) profiles.

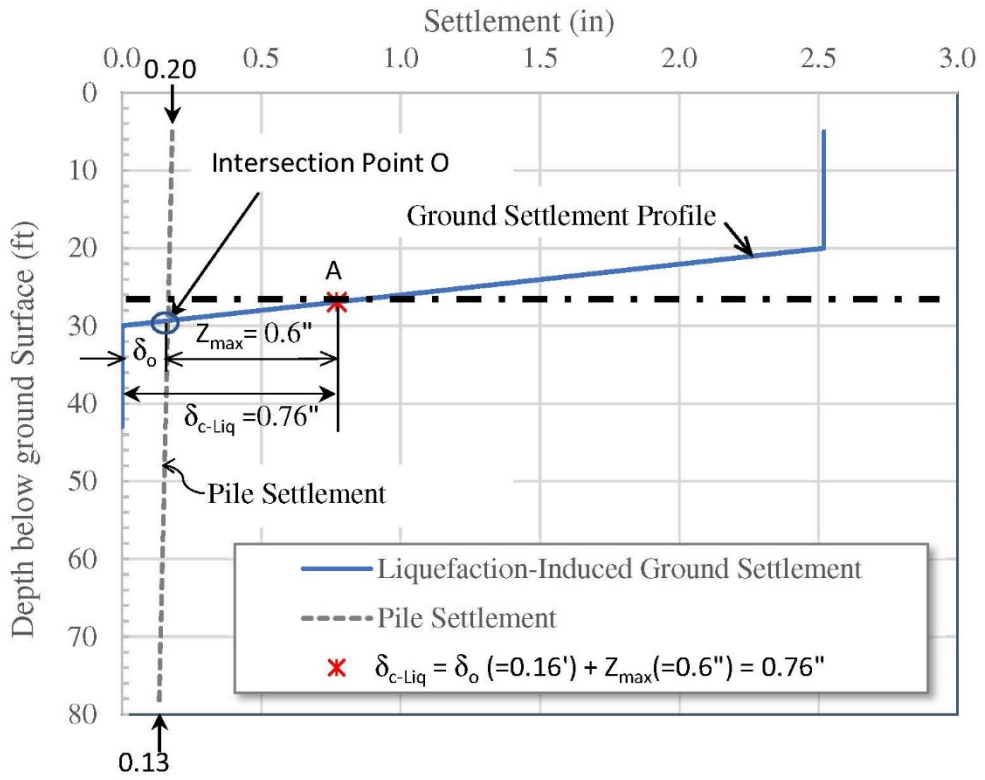


Figure A7: Plot of Pile and Liquefaction-Induced Ground Settlements

Step 4-5: Determine Location of the Maximum Downdrag Load (DDmax)

The relative pile-soil settlement, z_{max}, at which the unit side resistance is fully mobilized is calculated as about 0.9 percent of the pile diameter or 0.60 inches. The 0.9 percent was calculated by using an equivalent depth scheme for the clay and sand layers exerting downdrag load on the pile (i.e. 10 feet times 0.8 percent for the clay layer plus 10 feet times 1.0 percent for the sand layers, then divided by 20 feet).

The pile and ground settlement at the intersection point O (δ_o) is 0.16 inches. The critical ground settlement that will generate the maximum negative side resistance or downdrag load on the pile, $\delta_{c\text{-}Liq}$ = (δ_o) + z_{max} = (0.16 inches + 0.60 inches) = 0.76 inches.

As shown in Figure A7, locate the Point A on the ground settlement profile representing a ground settlement equal to $\delta_{c\text{-}Liq}$ = 0.76 inches. Draw a horizontal line AA' passing through the Point A. The pile length at the elevation of the line AA' is about 22 feet,

corresponding to the ground elevation of -27 feet. The soil layers above the A-A' line exert full downdrag loads on the pile and those below contribute to the pile axial nominal resistance.

Step 4-6: Calculate Maximum Downdrag Load (DDmax)

The nominal downdrag load due to liquefaction-induced settlement was calculated based on a soil friction angle of 30 degrees for the liquefiable soil layer (No. 3), and the soil shear strength parameters in Table A3 for the non-liquefiable soil layers (Nos. 1 and 2). Table A8 presents the output data from SHAFT.

Along the upper 22 feet pile length (from cut-off to A-A' line), the pile nominal side resistance for each soil layer are:

- Layer 1, R_{s1} = 43.3 tons x 2 = 86.6 kips
- Layer 2, R_{s2} = (114.6 tons – 43.3 tons) x 2 = 142.6 kips
- Layer 3 above A-A' line, R_{s3d} = [(206.92 tons – 193.36 tons) x 0.94 + 193.36 tons - 114.57 tons] x 2 = 183.07 kips.

Therefore, the maximum downdrag load, $DD_{max} = (R_{s1} + R_{s2} + R_{s3d})$ = 86.6 + 142.6+ 183.1 = 412 kips

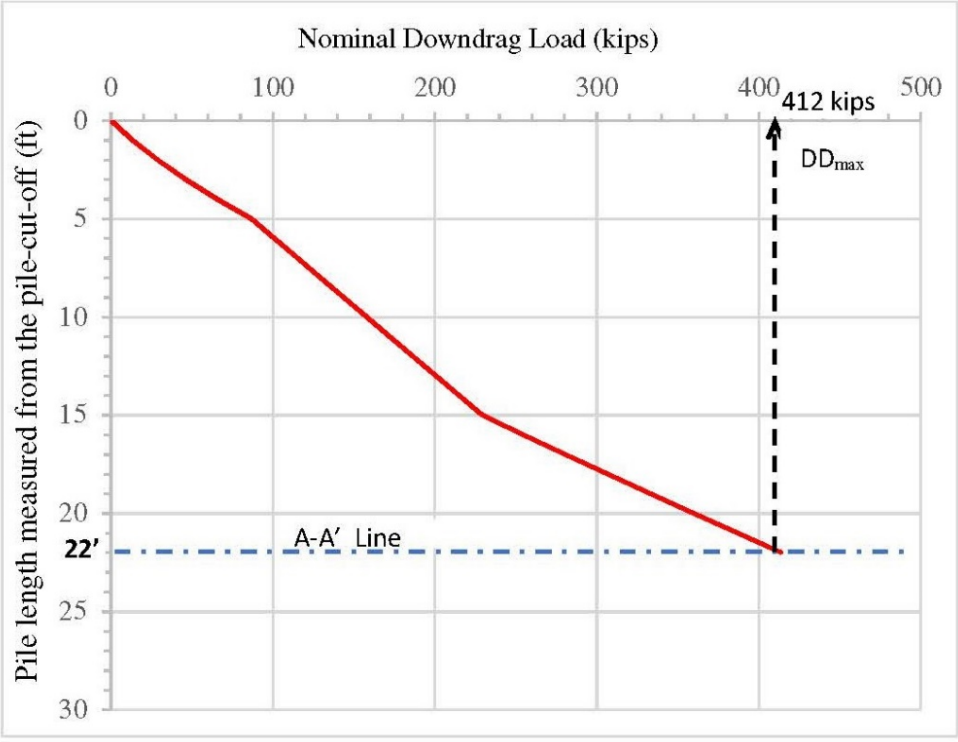


Figure A8: Nominal Downdrag Load vs. Pile Length

Table A8: Summary of Predicted Outputs from SHAFT (Extreme Event-I Downdrag)

LENGTH (FEET)	VOLUME (CU.YDS)	QS (TONS)	QB (TONS)	QU (TONS)	LRFD QS (TONS)	LRFD QB (TONS)	LRFD QU (TONS)
6.0	5.28	6.67	132.93	139.60	6.67	132.93	139.60
7.0	6.16	14.37	143.23	157.60	14.37	143.23	157.60
8.0	7.04	23.07	150.52	173.59	23.07	150.52	173.59
9.0	7.92	32.72	154.87	187.59	32.72	154.87	187.59
10.0	8.80	43.29	139.44	182.73	43.29	139.44	182.73
11.0	9.68	50.41	122.45	172.87	50.41	122.45	172.87
12.0	10.56	57.54	103.89	161.44	57.54	103.89	161.44
13.0	11.44	64.67	83.74	148.42	64.67	83.74	148.42
14.0	12.32	71.80	62.00	133.80	71.80	62.00	133.80
15.0	13.20	78.93	41.46	120.39	78.93	41.46	120.39
16.0	14.08	86.06	24.95	111.01	86.06	24.95	111.01
17.0	14.96	93.19	12.51	105.70	93.19	12.51	105.70
18.0	15.84	100.31	4.18	104.50	100.31	4.18	104.50
19.0	16.72	107.44	0.00	107.44	107.44	0.00	107.44
20.0	17.60	114.57	27.10	141.67	114.57	27.10	141.67
21.0	18.48	127.36	57.34	184.70	127.36	57.34	184.70
22.0	19.36	140.30	90.78	231.08	140.30	90.78	231.08
23.0	20.24	153.38	127.50	280.88	153.38	127.50	280.88
24.0	21.12	166.58	167.57	334.16	166.58	167.57	334.16
25.0	22.00	179.91	206.55	386.47	179.91	206.55	386.47
26.0	22.88	193.36	239.85	433.21	193.36	239.85	433.21
27.0	23.76	206.92	267.24	474.17	206.92	267.24	474.17
28.0	24.64	220.59	288.52	509.11	220.59	288.52	509.11
29.0	25.52	234.35	303.45	537.81	234.35	303.45	537.81
30.0	26.40	248.21	311.83	560.04	248.21	311.83	560.04
31.0	27.28	262.21	320.21	582.42	262.21	320.21	582.42
32.0	28.16	276.42	328.59	605.01	276.42	328.59	605.01
33.0	29.04	290.82	336.97	627.79	290.82	336.97	627.79
34.0	29.92	305.41	345.35	650.76	305.41	345.35	650.76
35.0	30.80	320.18	353.73	673.90	320.18	353.73	673.90
36.0	31.68	335.10	362.10	697.21	335.10	362.10	697.21
37.0	32.56	350.19	370.48	720.67	350.19	370.48	720.67
38.0	33.44	365.41	378.86	744.27	365.41	378.86	744.27
39.0	34.32	380.78	387.24	768.02	380.78	387.24	768.02
40.0	35.20	396.27	395.62	791.88	396.27	395.62	791.88
41.0	36.08	411.87	404.00	815.87	411.87	404.00	815.87
42.0	36.96	427.59	412.38	839.96	427.59	412.38	839.96
43.0	37.84	443.40	420.75	864.15	443.40	420.75	864.15
44.0	38.72	459.30	429.13	888.44	459.30	429.13	888.44
45.0	39.60	475.29	436.64	911.92	475.29	436.64	911.92
46.0	40.48	491.34	443.20	934.54	491.34	443.20	934.54
47.0	41.36	507.46	448.73	956.19	507.46	448.73	956.19
48.0	42.24	523.64	453.18	976.81	523.64	453.18	976.81
49.0	43.12	539.86	456.46	996.31	539.86	456.46	996.31
50.0	44.00	556.12	458.64	1014.76	556.12	458.64	1014.76
51.0	44.88	572.41	459.95	1032.36	572.41	459.95	1032.36
52.0	45.76	588.72	460.61	1049.32	588.72	460.61	1049.32
53.0	46.64	605.04	460.83	1065.87	605.04	460.83	1065.87
54.0	47.52	621.37	460.83	1082.20	621.37	460.83	1082.20
55.0	48.40	637.70	460.83	1098.52	637.70	460.83	1098.52

Step 4-7: Calculate Pile Nominal Resistances in Compression (RN) below the A-A' Line

The total pile nominal side resistance below the A-A' line (for 22 feet pile length) is calculated utilizing the information in Table A8. The nominal resistance for any soil layer along the pile length can be obtained by subtracting the total nominal side resistance at the top of the layer from that at the bottom of the layer. In the example case, the total nominal side resistance of the pile located below the A-A' line includes the nominal side resistance from the portion of the soil layer no. 3 below the A-A' line (R_{s3r}) and the nominal side resistance from the soil layer no. 4 (R_{s4}). Figure A9 presents plots of the pile nominal side and tip resistances below the A-A' line.

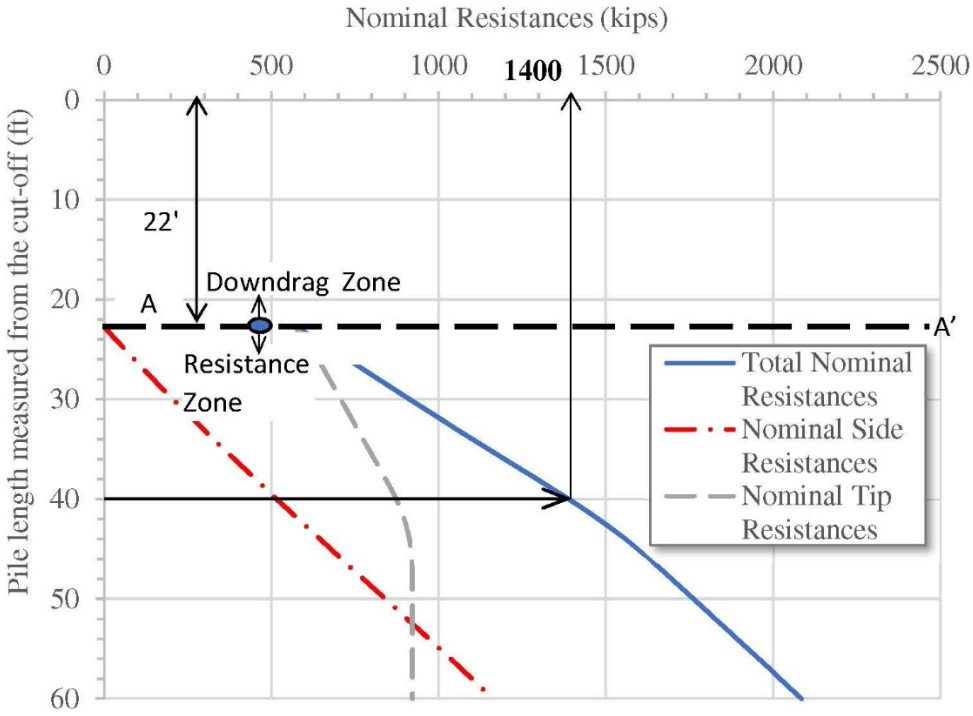


Figure A9: Nominal Resistances with Pile Length for Extreme Event-I Downdrag

Step 4-8: Determine Pile DTE for Compression (Extreme Event-I Downdrag)

For the Extreme Event Limit-I Downdrag case, the factored total seismic design load per pile = (factored max. permanent load per pile) + (factored DD_{max} per pile) + (Factored effective pile weight of the pile). = (886 kips x 1.0) + (412 kips x 1.0) + (W'p kips x 1.0) = 1298 kips + W'p (kips). Note all load and resistance factors are taken as equal to 1.0.

Due to the inclusion of the effective pile weight in the factored design, a trial and error method is necessary to determine the pile DTE from Figure A9. A preliminary estimate of the required pile length = 37 feet can be made based on a factored design load = 1298 kips (i.e. excluding effective weight of the pile).

Trial 1: Assuming the required preliminary pile length = 37 feet, the available total factored resistance can be estimated to about 1270 kips from Table A8 or Figure 9.

The total effective pile weight for the 37 feet long pile, W'p (kips) = 85 kips. In this case, the total factored design load on the pile for the downdrag case = 1298 kips + 85 kips = 1383 kips. This factored design load is greater than the available total factored resistance of 1270 kips for the 37 feet long pile. Therefore, additional pile length is required to support the factored design load.

Trial 2.: Assume pile length = 40 feet. From Figure A9 and Table A8, the available total factored nominal resistance in axial compression for 40 feet long pile is estimated to be about 1400 kips. This total factored nominal resistance consists of total factored nominal side resistance, R_s=530 kips and total nominal pile tip resistance R_p = 870 kips.

The revised total factored load, including effective weight if the pile, for the 40 feet long pile is estimated to be about 1380 kips which is slightly lower than the available total factored resistance of 1400 kips. This pile length is thus adequate to support the applicable factored design load for the Extreme Event (Downdrag) limit state.

Therefore, the design DTE for the Extreme Event (Downdrag) is -45 feet.

Reporting

Based on the above results, for the example case:

- Extreme Event (Compression) case: Geotechnical Nominal Resistance per Pile = 1250 kips, Nominal Tip Resistance = 850 kips and the design DTE = - 43.5 feet.
- Extreme Event (Downdrag) case: Geotechnical Nominal Resistance per Pile = 1400 kips, Nominal Tip Resistance = 873 kips and the design DTE =-42 feet.
- Maximum downdrag load on the pile, DD_{max}= 412 kips.
- The downdrag zone: Top Elevation = - 5 feet and Bottom Elevation = -27 feet.
- Seismic ground settlement at the cut-off elevation, $(\delta_g)_{DD}$ = 2.5 inches.
- Total Ground Settlement $(\delta_g)_{Liq}$ = 2.5 inches.
- No t-z and q-w curves are not requested.

Additional Information

Table A9 presents a partially completed foundation design recommendations table for the example.

Extreme Table A9: Foundation Design Recommendations

Support Location	Pile Type	Cut-off Elev. (feet)	Service-1 Limit State Load per Support (kips)		Total Permissible Support Settlement (inches)	Geotechnical Nominal Resistance (kips)					Design Tip Elev. (feet)	Specified Design Tip Elev. (feet)
						Strength Limit		Extreme Event Limit (All Resistance Factors = 1.0)				
			Total	Perm.		Comp. (Φqs=0.7) (Φqp=0.5)	Tension (Φqs=0.7)	Comp.	Comp. with Downdrag	Tension		
Bent	66" dia. CIDH	-5.0	8916	7085	1.0	2920	0	1140	1390	0	-79 (a-I) -43.5 (a-II) -45 (a-III) -34.3 (c)	xx

Note: *Design tip elevations are controlled by, (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (a-III) Compression (Extreme Event-Downdrag), (c) Settlement.*

Additional References

Ensoft, Inc (2012), SHAFT A Program for the Study of Drilled Shafts Under Axial Load, Austin, Texas.

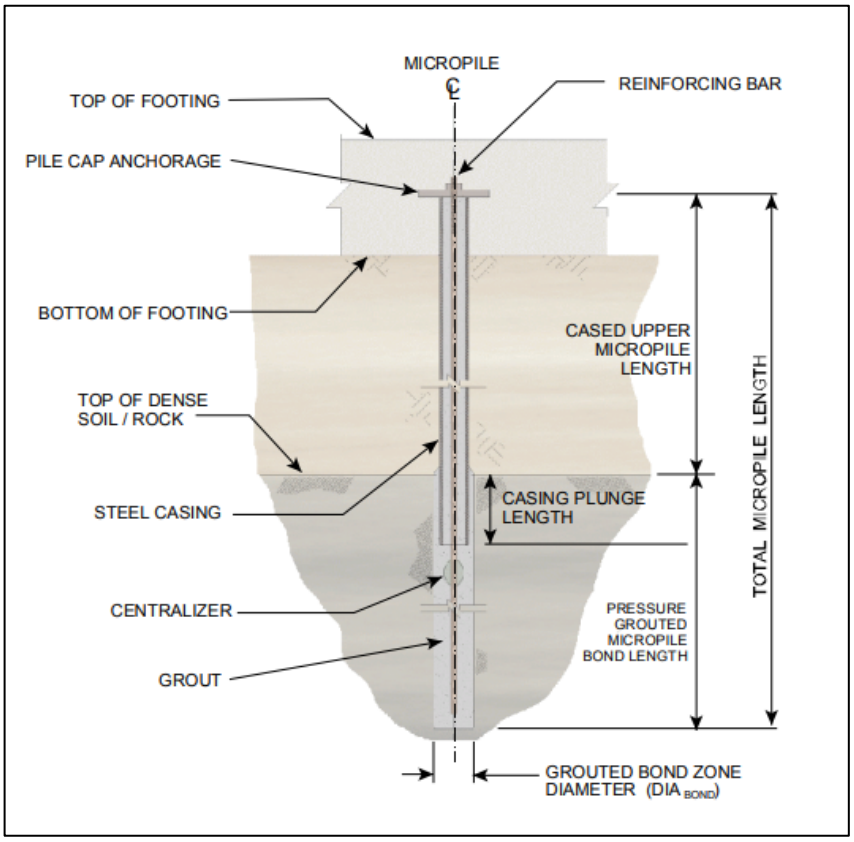
Micropiles for Structure Foundations (Category A Micropiles)

Micropiles are an option for use as foundation support elements or as in-situ reinforcements to provide stabilization of slopes and excavations. The intent of this module is to provide geotechnical design guidelines when micropiles are to be used as foundation support elements (**Category A** micropiles, as defined in the Micropile Classifications section of this module). The design of micropiles for slope stabilization (**Category L** micropiles) is not presented in this module.

A micropile is a relatively small diameter (typically less than 12 inches) grouted pile with internal reinforcement. A micropile is constructed by drilling a borehole (typically using temporary casing), placing steel reinforcement inside, and grouting the hole. Micropile reinforcement may consist of a single reinforcing bar, a group of reinforcing bars, and/or a steel casing. The use of a single reinforcing bar and steel casing is most commonly used. The steel casing is sometimes referred to as a pipe or hollow structural section (HSS). A micropile utilizing partial length permanent casing and reinforcement is called a composite micropile. A typical composite micropile cross section used for structural foundation support is shown on Figure 1 below.

Micropiles are usually designed in small clusters or groups with each pile typically carrying an equal amount of load. They can be designed to resist a combination of compression, tension, and lateral forces.

Figure 1: Details of a Composite Reinforced Micropile (from FHWA NHI-05-039)



Caltrans typically requires that a composite micropile must be used for structure foundation support. The composite micropiles must consist of an inner steel reinforcing element and outer steel casing. Structure Designer (SD) is responsible for specifying the length of the steel casing. The steel casing is typically extended to the lateral tip elevation. Because of their high slenderness ratio (length/diameter), micropiles must not be used in soils susceptible to liquefaction, lateral spreading, or scour. For new bridges, use of micropiles must be limited to Class S1 Soil as defined by Seismic Design Criteria v2.0, Section 6.1.2. Micropiles may be used in Class S2 Soil if the micropiles are not the primary supporting elements. An example of micropiles that are not primary supporting elements may be in seismic retrofit application where the existing piles are the primary supporting elements. Due to their unique design, the use of micropiles must require approval at the type selection meeting.

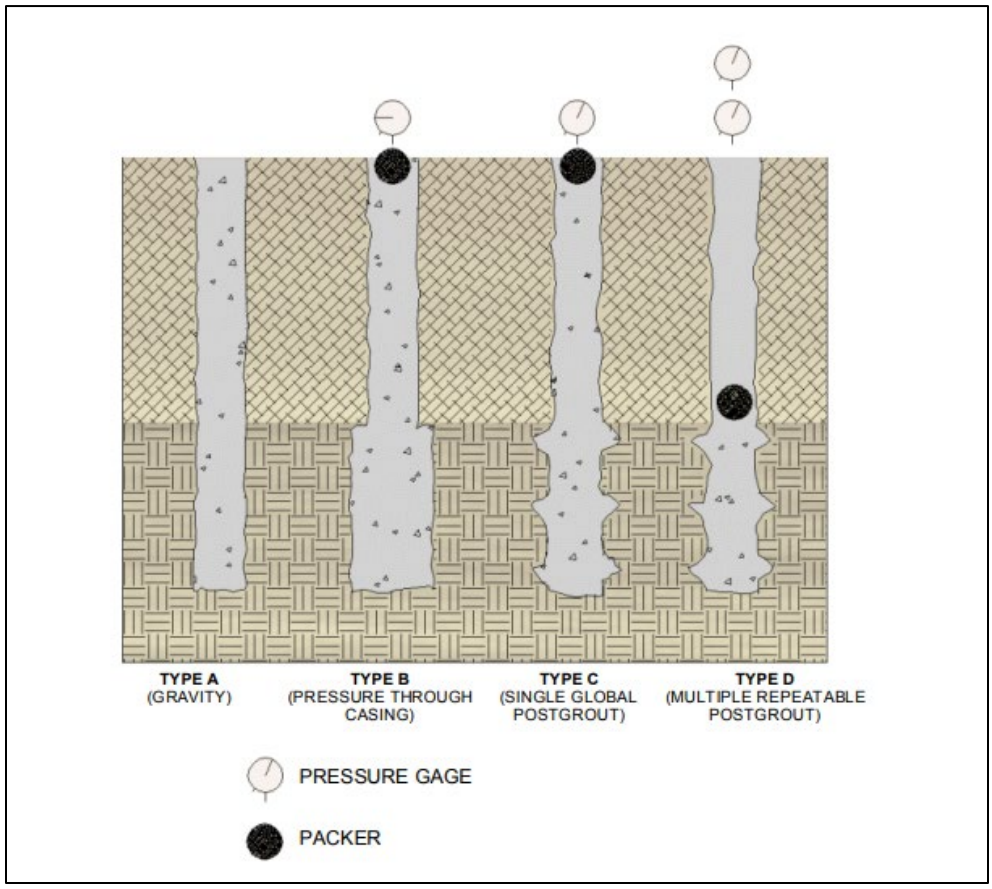
Micropile Classifications

The method of grouting is typically the most important construction process influencing grout/ground bond capacity. AASHTO LRFD BDS classifies micropiles based on the methods of installation/grouting, as follows:

- **Type A:** Micropiles are constructed by placing a sand-cement mortar or neat cement grout in the pile under a gravity head only. Type A grouting is generally used for micropile in rock.
- **Type B:** Micropiles are constructed by injecting a neat cement grout under pressure (typically 6–21 ksf) into the drilled hole while the temporary drill casing or auger is withdrawn.
- **Type C:** Micropiles are grouted as for Type A, followed 15–25 minutes after primary grouting by injection of additional grout under pressure (typically greater than 21 ksf) via a preplaced sleeved grout pipe.
- **Type D:** Micropiles are grouted similar to Type C, but the primary grout is allowed to harden before injecting the secondary grout under pressure (typically 42–170 ksf) with a packer to achieve treatment of specific pile intervals or material horizons.
- **Type E:** Micropiles are constructed by drilling with grout injection through a continuous-thread, hollow-core steel bar. The grout injection serves to flush cuttings, achieve grout penetration into the ground, and stabilize the drill hole. Often the initial grout has a high water to cement ratio and is then replaced with a thicker structural grout near the completion of drilling.

The classification is shown schematically for the various Types in Figure 2 below.

Figure 2: Micropile Types (from FHWA NHI-05-039)



Caltrans divides the design of an individual micropile or a group of micropiles into the following categories:

- **Category A:** Micropiles experiencing primarily axial forces due to axial loading at the top of micropile. This would typically include bridge foundations and may occasionally include walls. For Category A, the contractor determines the length, load testing is performed, and pressure grouting is used. Category A micropiles are sometimes referred to as Case 1 micropiles in the industry.
- **Category L:** Micropiles experiencing primarily shear and bending forces along their length due to lateral loading. This would typically include earth stabilization structures and most walls. For Category L, SD and Geoprofessional (GP) will determine the micropile length, load testing is not performed, and gravity grouting is used. Category L micropiles are sometimes referred to as Case 2 micropiles in the industry.

Design Methods and Requirements

When considering the use of micropiles in foundation design, refer to the following references:

- Caltrans Geotechnical Manual
- Section 10.9 of the AASHTO LRFD BDS with California Amendments (AASHTO LRFD BDS)
- Section 49-5 of the Standard Special Provisions (SSP)
- *Micropile Design and Construction Reference Manual* (FHWA NHI-05-039)

Roles and Responsibilities

The typical responsibilities of the SD and GP for Category A micropile design are listed in Table 1 below.

Table 1: Responsibilities for **Category A** Micropile Design

Structure Designer (SD)	Geoprofessional (GP)
• Determine micropile layout and spacing. • Design of steel reinforcement. • Shown on plans type and length of permanent steel casing/pipe/HSS. • Shown on plans grout compressive strength. • Provide general foundation design information and foundation factored design loads (MTD 3-1, Attachment 1 Tables 3-4 and 3-5). • Determine the elastic deformation based on estimated micropile bonded length. • Show on plans the verification load test locations.	• Provide subsurface information and laboratory testing results. • Specify verification test locations and the control zones. • Specify the number of proof load tests required per footing. • Provide corrosion test results. • Request creep testing, if needed. • Provide estimated micropile bonded length* (only to be used for estimating elastic deformation). • Specify the design tip elevation for tolerable settlement, when applicable. • Provide soil modulus parameters for p-y curve development. (All above information must be included in the Foundation Report)
• SD and GP together will determine the axial Service Load and Factored Test Load to be used in the load testing and they should be shown on the plans. • SD and GP together will determine the allowable total top of pile movement at 1.00SL for load testing.	

Note: **Contractor** will determine the micropile bonded length and installation methods to meet the axial load test requirements.
* Refer to Appendix A for estimation of bonded length.

Investigations

For the design of micropiles, the subsurface investigation, laboratory testing, and evaluation of geotechnical design parameters is similar to that for either driven piles or CIDH piles (FHWA NHI-05-039). The geotechnical investigation should adequately define the subsurface conditions for design purposes and be consistent with the standards of practice identified in the Caltrans Geotechnical Manual. The extent of the exploration, field testing, and laboratory testing must give a reasonable degree of confidence in the property measured. The *Micropile Design and Construction Reference Manual* (FHWA NHI-05-039) provides minimum guidelines for boring spacing and depth, as shown in Table 2 below.

Table 2: Guidelines for Minimum Number of Investigation Points and Depth of Investigation
(from FHWA NHI-05-039)

Application	Minimum Number of Investigation Points and Location of Investigation Points	Minimum Depth of Investigation
Deep Foundations (Micropiles for Structural Support)	For substructure (e.g., bridge piers or abutments) widths less than or equal to 30 m (100 ft), a minimum of one investigation point per substructure. For substructure widths greater than 30 m (100 ft), a minimum of two investigation points per substructure. Additional investigation points should be provided if erratic subsurface conditions are encountered.	In soil, depth of investigation should extend below the anticipated micropile tip elevation a minimum of 6 m (20 ft), or a minimum of two times the maximum micropile group dimension, whichever is deeper. All borings should extend through unsuitable strata such as unconsolidated fill, peat, highly organic materials, soft fine-grained soils, and loose coarse-grained soils to reach hard or dense materials. For micropiles bearing on rock, a minimum of 3 m (10 ft) of rock core must be obtained at each investigation point location to verify that the boring has not terminated on a boulder. For micropiles supported on or extending into rock, a minimum of 3 m (10 ft) of rock core, or a length of rock core equal to at least three times the micropile diameter for isolated micropiles or two times the maximum micropile group dimension, whichever is greater, must be extended below the anticipated micropile tip elevation to determine the physical characteristics of rock within the zone of foundation influence.

Corrosion Testing Evaluation

Soil and/or water samples should be collected during the subsurface investigation to determine the corrosion potential at the project site. The corrosion evaluation will be based on Caltrans Corrosion Guidelines. Micropiles are not recommended for use in a corrosive environment. If it is decided to use micropiles in a corrosive environment, the Corrosion Branch must be consulted for mitigation measures.

Micropile Design

Group Capacity

At small micropile spacings, a block type failure mechanism may occur, whereas at larger pile spacings the individual pile failure may occur. It is necessary to check for both failure mechanisms and design for the case that yields the minimum capacity. For determining the reduction in resistance from group effects in compression and tension (uplift), refer to Section 10.9.3.6 and 10.9.3.8 of the AASHTO LRFD BDS, respectively. If the group efficiency factor (η) is less than 1.0, the nominal resistance must be determined as:

$$R_N = \frac{FDL}{\varphi\eta}$$

Where,

R_N = nominal resistance (k)

FDL = factored design load (k)

Φ = resistance factor, 0.7

η = micropile group efficiency factor

Group Settlement

Micropiles in a group can undergo vertical displacement as a result of underlying soft soil layers. The equivalent footing concept, as used for driven pile groups, is used for estimating settlements of micropile groups. The GP should estimate the micropile group displacement when the underlying soil is cohesive in nature and subject to consolidation. Refer to Section 5.10 of the ***Micropile Design and Construction Reference Manual*** (FHWA NHI-05-039) for step by step procedure for calculating micropile settlement. When a minimum pile penetration is necessary to satisfy the settlement requirement, the GP will specify the design tip elevation for settlement. The settlement design tip will be shown in the Foundation Design Recommendations Table and Pile Data Table in the Foundation Report. However, the contractor will determine the micropile length and installation methods needed to meet the axial load test requirements.

Micropile Load Testing

Micropiles must be load tested in the field to verify the required design loads can be achieved without excessive settlement. There are two types of micropile load testing: verification testing and proof testing.

Verification Testing

The verification testing is performed on sacrificial micropiles prior to construction of production micropiles. The verification test is to verify the design and adequacy of the contractor's drilling, installation, and grouting operations. Each verification micropile must have the identical materials, dimensions, tip elevations, and installation methods as the production micropiles it represents. A sufficient number of verification test micropiles must be provided to represent all the production micropiles. The GP must provide verification micropile locations based on representative subsurface conditions. Since the length of the verification and production micropiles must match, SD and GP should consider performing additional verification tests to allow for optimization of micropile length when there is a wide range in required nominal resistance.

Proof Testing

Proof testing is performed on production micropiles to provide quality assurance. During installation of production micropiles, proof testing is conducted on a specified number of the total production micropiles. The frequency of proof testing must be at least 10%, but not less than 2, of the total micropiles per bridge footing. The proof load test locations will be selected by Caltrans field engineer.

Service Load and Factored Test Load

The load test schedule is based on Service Load (SL) and Factored Test Load (FTL). SL is the maximum axial load for Service Load combination. FTL is typically the highest required nominal resistance of the micropiles. The maximum test load for verification is 1.00 FTL and for proof testing is 0.80FTL. SD and GP together will determine the values of SL and FTL to be used in the load testing. It is recommended that verification and proof testing be performed in tension only, with the FTL equal to the greater of the tension or the compression nominal resistance. Tension load testing is less expensive and allows the ground and/or production piles to be used for reaction to the test load.

Creep Testing

Creep is a primary concern in organic soils, and in cohesive soils with a liquid limit greater than 50 and plasticity index greater than 20 (FHWA NHI-05-039). The creep test consists of measuring the movement of the micropile at a constant load over a specified period of time. If recommended by the GP, creep tests are performed as part of verification and proof tests.

Load Testing Acceptance Criteria

For verification load testing acceptance, the slope of the graph of the applied test load versus the top of micropile movement must not exceed 0.025 in/kip at the maximum test load of 1.00FTL (Caltrans SSP Section 49-5).

For verification and proof load testing, a maximum top of micropile movement at 1.00SL is recommended based on the considerations of the movements of the structure being supported by the micropile foundation. The top of micropile movement is a function of the pile stiffness (elastic deformation) and the grout-to-ground bond movement. GP will estimate the micropile bonded length. The bonded length is assumed to start from the tip of micropile casing/pipe/HSS (assume the casing/pipe/HSS has no geotechnical capacity). Refer to Appendix A for estimation of the micropile bonded length. Once determined, the estimated micropile bonded length is then provided to SD. The estimated bonded length is only to be used for estimating the elastic deformation by SD, and it is **not** to be shown in the Foundation Design Recommendations Table or Pile Data Table. SD and GP together will determine the allowable total top of pile movement at 1.00SL.

Load Test Schedules

The verification and proof load test schedules are listed in the project's Special Provisions. Examples of the load test schedules are shown in Appendix B

Reporting

Foundation Reports for micropiles must comply with the Foundation Reports for Bridges module. Micropile Design tip elevations for compression and tension must not be included in the tables. The GP will specify the micropile design tip elevation for settlement only if a minimum pile penetration is necessary. See Tables 3 and 4 below (micropile design at Abutments 1 and 3 locations).

Table 3: Foundation Design Recommendations Table

Support Location	Pile Type	Pile Cut-off Elev. (feet)	Service-I Limit State Load Per Support (kips)		Total Permissible Support Settlement (in)	Nominal Resistance[1] (kips)				Design Tip Elev. (feet)	Specified Tip Elev.[2] (feet)	Casing Tip Elev.[3] (feet)
						Strength/ Construction		Extreme Event				
			Total	Permanent		Comp. φ = 0.7	Tension φ = 0.7	Comp φ = 1.0	Tension φ = 1.0			
Abut 1	10-inch Micropile with Casing	___	___	___	___	___	___	N/A	N/A	___	___	___
Abut 3	10-inch Micropile with Casing	___	___	___	___	___	___	N/A	N/A	___ (c)	___	___

Note: Design tip elevations are controlled by (a-I) Compression (Strength Limit), (a-II) Compression (Extreme Event), (c) Settlement

1) Column heading modified from *Required Factored Nominal Resistance* to *Nominal Resistance*.
2) Micropile lengths will be determined by contractor. Micropile settlement design tip indicates the minimum pile length.
3) Casing tip elevations are provided by the structure designer.
4) For CIDH piles, the required nominal tip resistance is shown in parenthesis.

Table 4: Pile Data Table

Support Location	Pile Type	Nominal Resistance (kips)		Design Tip Elev. (feet)	Specified Elev. [1] (feet)	Casing Tip Elev.[2] (feet)
		Compression	Tension			
Abut 1	10-inch Micropile with Casing	___	___	___	___	___
Abut 3	10-inch Micropile with Casing	___	___	___(c)	___	___

Note: Design tip elevations are controlled by (a) Compression (c) Settlement

1) Micropile lengths will be determined by contractor. Micropile settlement design tip indicates the minimum pile length.
2) Casing tip elevations are provided by the structure designer.

Appendix A: Micropile Bond Length

The nominal grout-to-ground bond resistance over the bonded length of a micropile must be taken as:

$$R_s = \pi d_b \alpha_b L_b$$

Where,

R_s = nominal bond resistance (k)

d_b = diameter of micropile drilled hole through bonded length (feet)

α_b = nominal micropile grout-to-ground bond strength (ksf)

L_b = micropile bonded length (feet)

The maximum test load for load testing is 1.00 FTL (factored test load), therefore, the equation above can be rewritten and rearranged as:

$$L_b = \frac{FTL}{\pi d_b \alpha_b}$$

As a guide, Table A-1 below, can be used to estimate the grout-to-ground bond strength. Refer to Micropile Classifications section of this module for micropile types.

Table A1: Summary of Typical Grout-to-Ground Bond Strength (α_b)
(from Table C10.9.3.5.2-1 of AASHTO LRFD BDS)

Soil/Rock Description	Typical Range of Grout-to-Ground Bond Nominal Resistance for Micropile Types (ksf)				
	Type A	Type B	Type C	Type D	Type E
Silt & Clay (some sand) (soft medium plastic)	0.7–1.4	0.7–2.0	0.7–2.5	0.7–3.0	0.7–2.0
Silt & Clay (some sand) (stiff, dense to very dense)	0.7–2.5	1.4–4.0	2.0–4.0	2.0–4.0	1.4–4.0
Sand (some silt) (fine, loose-medium dense)	1.4–3.0	1.4–4.0	2.0–4.0	2.0–5.0	1.4–5.0
Sand (some silt, gravel) (fine-coarse, medium-very dense)	2.0–4.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Gravel (some sand) (medium-very dense)	2.0–5.5	2.5–7.5	3.0–7.5	3.0–8.0	2.5–7.5
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	2.0–4.0	2.0–6.5	2.5–6.5	2.5–7.0	2.0–6.5
Soft Shales (fresh-moderate fracturing, little to no weathering)	4.3–11.5	N/A	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	10.8–28.8	N/A	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	21.6–43.2	N/A	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	10.8–36.0	N/A	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	28.8–87.7	N/A	N/A	N/A	N/A

Appendix B: Testing Schedules

Table B1: Verification Load Test Schedule (from 2018 SSP)

Load Increment	Hold Time (minutes)
AL	Until stable
0.25SL	1-2
AL	Until stable
0.25SL	1-2
0.50SL	1-2
AL	Until stable
0.25SL	1-2
0.50SL	1-2
0.75SL	1-2
AL	Until stable
0.25SL	1-2
0.50SL	1-2
0.75SL	1-2
1.00SL	5*
AL	Until stable
0.25SL	1-2
0.50SL	1-2
0.75SL	1-2
1.00SL	1-2
0.80SL + 0.20FTL	1-2
0.60SL + 0.40FTL	1-2
0.40SL + 0.60FTL	1-2
0.20SL + 0.80FTL	1-2
1.00FTL**	5
0.75FTL	1-2
0.50FTL	1-2
0.25FTL	1-2
AL	Until stable

AL=alignment load, 0.10SL
SL=service load
FTL=factored test load
*Hold time up to 60 minutes if creep test is recommended, refer to Section 49-5 the Standard Special Provisions for details.
**Maximum test load

Table B2: Proof Load Test Schedule (from 2018 SSP)

Load Increment	Hold Time (minutes)
AL	Until stable
0.25SL	1-2
0.50SL	1-2
0.75SL	1-2
1.00SL	5*
0.80SL + 0.20(0.80FTL)	1-2
0.60SL + 0.40(0.80FTL)	1-2
0.40SL + 0.60(0.80FTL)	1-2
0.20SL + 0.80(0.80FTL)	1-2
0.80FTL**	5
AL	Until stable

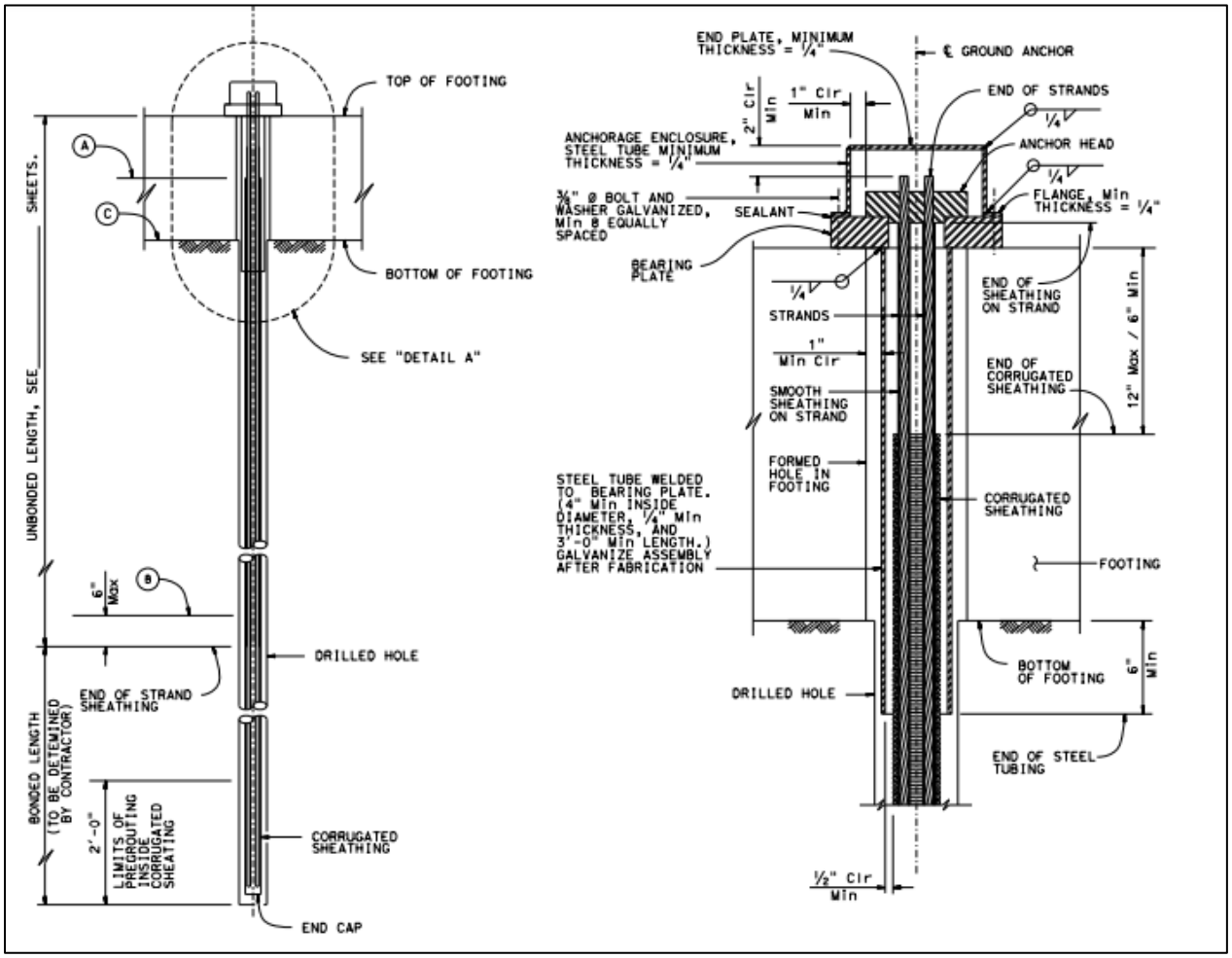
AL=alignment load, 0.10SL
SL=service load
FTL=factored test load
* Hold time up to 60 minutes if creep test is recommended, refer to Section 49-5 the Standard Special Provisions for details
**Maximum test load

Vertical Ground Anchors

A vertical ground anchor, also referred to as “tie-downs,” is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Vertical ground anchors can be used to resist uplift forces at bridge foundations and earth retaining structures such as a Caltrans Type 7 retaining wall. The intent of this module is to provide geotechnical design guidelines when vertical anchors are to be used as part of foundation support elements for bridges and retaining structures.

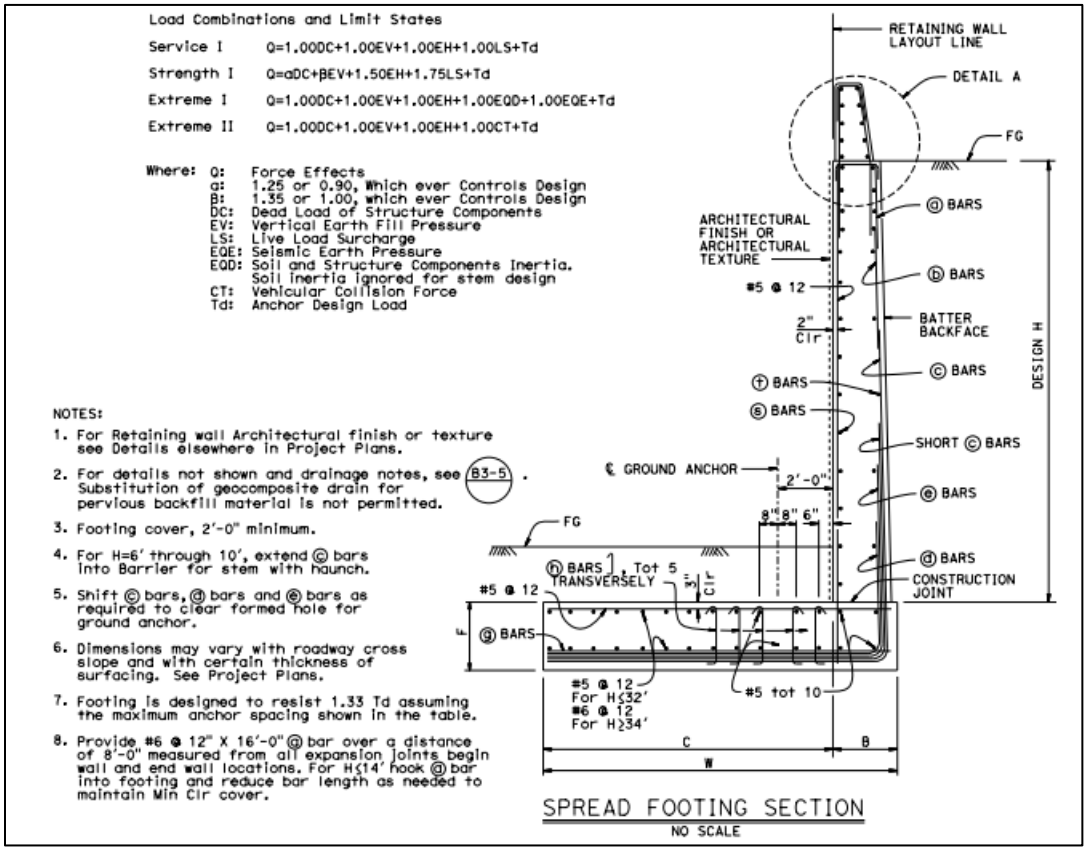
Vertical grouted ground anchors are installed in grout filled drill holes. The basic components of a grouted ground anchor include the anchorage system, tendon, and grout. The anchorage system generally consists of a bearing plate, trumpet (steel tube welded to the bearing plate), and an anchor head with wedges (or threaded nut). The tendon can be either multiple-wire strands or a high strength bar that transmits the tensile force. The grout transfers the tensile force from the tendon to the ground. Figure 1 shows the components of a vertical ground anchor.

Figure 1: Vertical Ground Anchor Details (from Caltrans XS Sheet No. xs12-030-1)



Vertical ground anchors can be used for foundation elements subject to uplifting or overturning. For example, vertical ground anchors are utilized in Caltrans Type 7 retaining wall design to resist overturning of the wall. Figure 2 shows details of a Caltrans Type 7B retaining wall.

Figure 2: Type 7B Retaining Wall (from Caltrans XS Sheet No. xs14-375-1)



Design Methods and Requirements

When considering the use of vertical ground anchors in foundation design, refer to the following references:

- AASHTO LRFD BDS with California Amendments (AASHTO LRFD BDS)
- Section 46 of the Standard Special Provisions (SSP)
- *Geotechnical Engineering Circular N0. 4, Ground Anchors and Anchored Systems* (FHWA IF-99-015)

Roles and Responsibilities

The typical responsibilities of the structure designer (SD) and Geoprofessional (GP) for vertical ground anchor design are listed in Table 1.

Table 1: Responsibilities for Vertical Ground Anchor Design

Structure Designer (SD)	Geoprofessional (GP)
• Determine the unfactored design load (DL). • Determine seismic design load (T_s). • Determine the factored design load: for bridge foundation (FDL); for Caltrans Type 7 wall (T_d). • Determine the factored test load: for bridge foundation (FTL); for Caltrans Type 7 wall (T_p). • Determine the lock-off load: for bridge foundation (LL); for Caltrans Type 7 wall (T_o).	• Specify the unbonded length. • Specify the pullout resistance factor to be used in design. • Request extended creep testing, if needed. • Specify the number of performance tests* required per footing/wall segments. (All above information shall be included in the Foundation Report)
• SD and GP together will determine the minimum spacing between adjacent anchors. • When extended creep tests are required, SD and GP together will determine the number of extended creep tests, test load schedule, and the test locations.	

Contractor will determine the vertical ground anchor bonded length and installation methods to meet the axial load test requirements.
* The performance test locations will be selected by the Caltrans field engineer.

Investigations

The geotechnical investigation should adequately define the subsurface conditions for design purposes and be consistent with the standards of practice identified in the Caltrans Geotechnical Manual. The extent of the exploration, field testing, and laboratory testing must give a reasonable degree of confidence in the property measured. At a minimum, the following items need be identified during the subsurface investigation which may affect the design and construction of the anchored system:

- Groundwater elevation.
- Cohesionless soils which may be susceptible to liquefaction and caving.
- Highly compressible materials such as high plasticity clays and organic soils, which are susceptible to long-term (creep) deformation.
- Obstructions, boulders, and hard rock layers which adversely affect anchor hole drilling and grouting.

Corrosion Testing Evaluation

Soil and/or water samples should be collected during the subsurface investigation to determine the corrosion potential at the project site. The corrosion evaluation will be based on Caltrans Corrosion Guidelines.

Vertical Ground Anchor Minimum Spacing

The minimum spacing of vertical ground anchors should be determined in the consideration of ground mass failure. The recommended minimum spacing between ground anchors should be ten times the diameter of the hole within the bonded length, or 5 feet minimum. Group effects of closely spaced anchors reduce the load carrying capacity of individual ground anchors. The recommended minimum spacing ensures that group effects between adjacent ground anchors are minimized and that anchor intersection due to drilling deviations is avoided. If smaller spacing than the minimum is required to develop the required anchor design force, consideration may be given to staggering anchors or installing anchors with a battered angle. SD and GP together should determine the minimum spacing between adjacent anchors.

Vertical Ground Anchor Minimum Unbonded Length

The minimum unbonded length for vertical ground anchors is 15 feet for strand tendons and 10 feet for bar tendons. Longer unbonded lengths may be required to ensure the bonded zone starts behind the critical potential failure surface. In general, the unbonded length is extended a minimum distance of 5 feet behind the critical potential failure surface. When determining the unbonded length, considerations also should be given to ensure the anchor bonded zone will not have significate impacts on the adjacent foundation elements.

Vertical Ground Anchor Load Testing and Lock-Off Load

All vertical ground anchors shall be load tested in the field to verify the required design loads can be achieved without excessive deformation. There are three types of load testing: performance testing, proof testing, and extended creep testing. They are discussed below.

Performance and Proof Testing

Performance tests involve incremental loading and unloading of a vertical ground anchor. Performance testing is performed on selected production vertical ground anchors. GP shall specify the minimum number of performance testing for each bridge footing and wall segment in the foundation report. The frequency of performance testing shall be at least 10%, but not less than 2, of the total vertical ground anchors per bridge footing; and shall be at least 5%, but not less than 3, of the total vertical ground anchors per retaining wall. The performance test locations will be selected by the Caltrans field engineer. The proof test involves a single load cycle. Proof testing is performed on remaining vertical ground anchors not selected for performance testing.

The performance and proof testing load schedules are listed in the project's Special Provisions. Examples of the load test schedules are shown in Table A1 of Appendix A. For both the performance and proof testing, vertical ground anchors are tested to 100% of the factored test load (FTL). Table 1 below describes how the FTL is determined.

Table 1: Determination of Factored Test Loads

Structure Type	Factored Test Load (FTL)
Bridge Foundations-regular	FTL= greater of $\frac{FDL}{\varphi}$ or $\frac{1.05T_s}{\varphi}$
Bridge Foundations-vertical ground anchor is a failure-critical member of a bridge in Extreme Event Limit State	FTL= greater of $\frac{1.05FDL}{\varphi}$ or $\frac{1.05T_s}{\varphi}$
Caltrans Type 7 Retaining Walls	T_p = greater of $1.33T_d$ or $1.25T_o$

FDL= Factored design load, by SD.
T_s= Seismic design load, by SD.
Φ = Pullout resistance factor, use Φ = 0.7; for seismic design load (e.g., T_s), use Φ =1.0
T_p = Anchor factored test load for Caltrans Type 7 retaining wall, by SD.
T_d = Anchor design load for Caltrans Type 7 retaining wall, by SD.
T_o = Anchor lock-off load for Caltrans Type 7 retaining wall, by SD.

Extended Creep Testing

An extended creep test is a long duration test (approximately 8 hours) that is used to evaluate creep deformations of anchors. The extended creep test load hold times are much longer than the extended load hold times in the performance or proof tests. These tests are required for anchors installed in cohesive soil having a plasticity index greater than 20 or liquid limit greater than 50. For these ground conditions, GP should request extended creep testing in the Construction Considerations Section of the foundation report. When extended creep tests are required, SD and GP together will determine the number of extended creep tests, testing load schedule, and the test locations. Generally, a minimum of two ground anchors should be subjected to extended creep testing per bridge footing; and a minimum of three for retaining walls. Refer to Table A-2 of Appendix A for an example of extended creep testing load schedule (from FHWA IF-99-015).

Lock-Off Load

Depending on the lock-off load, vertical ground anchors can be classified as active or passive ground anchors. For bridge foundations, passive vertical ground anchors are usually prestressed to a lock-off load equal to 10% of the factored design load (FDL); and

active vertical ground anchors are prestressed to a load equal to or over 100% of factored design load. For retaining wall foundations, only active vertical ground anchors are used. SD will specify the lock-off loads (LL for bridge foundation, T_o for Caltrans Type 7 retaining wall) on the project plan sheets.

Reporting

For vertical ground anchors to be used in a bridge foundation, report in accordance with the *Foundation Reports for Bridges* module. For vertical ground anchors to be used in a retaining wall, report in accordance with the *Foundation Reports for Earth Retaining Systems* module.

Appendix A: Example Test Schedules

Table A1: Performance and Proof Load Test Schedules (from 2018 SSP)

Performance Test		Proof Test	
Load Increment	Hold Time (minutes)	Load Increment	Hold Time (minutes)
AL	Until stable	AL	Until stable
0.20T	1-2	0.20T	1-2
AL	Until stable	0.40T	1-2
0.20T	1-2	0.60T	1-2
0.40T	1-2	0.80T	1-2
AL	Until stable	1.00T*	10 or 60
0.20T	1-2	AL	Until stable
0.40T	1-2	-	-
0.60T	1-2	-	-
AL	Until stable	-	-
0.20T	1-2	-	-
0.40T	1-2	-	-
0.60T	1-2	-	-
0.80T	1-2	-	-
AL	Until stable	-	-
0.20T	1-2	-	-
0.40T	1-2	-	-
0.60T	1-2	-	-
0.80T	1-2	-	-
1.00T*	10 or 60	-	-
AL	Until stable	-	-

AL=alignment load, 0.10T
T=factored test load
*Maximum test load

Table A2: Example of Extended Creep Load Test Schedule (from FHWA IF-99-015)

Loading Cycle	Maximum Cycle Load	Total Observation Period (min.)	Movements Measured at Following Times (min.)
AL	Until stable	-	-
1	0.25FDL	10	1, 2, 3, 4, 5, 6, 10
2	0.50FDL	30	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30
3	0.75FDL	30	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30
4	1.00FDL	45	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45
5	1.20FDL	60	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60
6	1.33FDL*	300	1, 2, 3, 4, 5, 6, 10, 15, 20, 25, 30, 45, 60, 300
AL	Until stable	-	-

AL=alignment load, 0.10FDL
FDL=factored design load
*Maximum test load

1 Embankments

This module documents the Department's standard of practice for the investigation, design, and construction of embankments. Highway embankments, bridge approaches, embankment widening, and storm damage issues are addressed. Primary references for this module are:

- Soil Slope and Embankment Design, FHWA-NHI-132033, September 2005.
- Washington State Dept. of Transportation Geotechnical Design Manual, M 46-03.09, December 2013, Chapter 9 Embankments.
- Standard Specifications, State of California, Department of Transportation

The Geoprofessional's role is to provide geotechnical recommendations for the design, construction, and maintenance of embankments that are practical, cost-effective, constructible, and value driven.

Our clients are primarily district design units to which we provide both planning and design recommendations. District design units submit work requests to GS that should include the following information:

- Plan sheets with embankment area(s);
- Cross-sections (typically at 50-foot to 100-foot intervals, depending upon topography, etc.);
- Profiles of the planned alignment.

For planning phase requests these items may not be very well defined, but the geoprofessional should receive some information that indicates planned embankment locations and heights. For design phase requests, these items should be more thoroughly developed and must be provided. Communicate with the client to assure a common understanding of work to be addressed and project constraints.

For the purposes of this module, embankments include the following:

- Rock embankments, defined as fills in which the material in all or any part of an embankment contains 25 percent or more, by volume, cobbles and/or boulders;
- Bridge approach embankments, defined as fill extending from a bridge abutment for 150 feet;
- Embankments are fills that are not classified as rock or bridge approach embankments, but that are constructed with soil;
- Lightweight fills contain lightweight fill or recycled materials as a significant portion of the embankment volume, and the embankment construction is usually controlled by special provision. Lightweight fills are most often used as a portion of the bridge approach embankment to mitigate settlement and/or stability issues, or in landslide repairs to reestablish roadways.

2 Embankment Investigations

The key geotechnical considerations for design and construction of embankments are stability and settlement of the foundation soils, the impact of the stability and settlement on the construction staging and time requirements, and impacts to nearby structures, such as buildings, bridge foundations, and utilities. The investigation should include a detailed site review outside the proposed embankment footprint in addition to within the embankment footprint. The investigation should extend at least two to three times the width of the embankment on either side and to the top or bottom of slopes adjacent to the embankment. Furthermore, areas below proposed embankments should be fully explored if landslide activity is suspected.

2.1 Planning the Field Exploration and Laboratory Testing

The Geoprofessional should assess project requirements and anticipated subsurface conditions to determine the type and quantity of information to be obtained during the geotechnical investigation. To complete the assessment:

- Identify performance criteria (e.g. allowable settlement, time available for construction, seismic design requirements, etc.);
- Identify potential geologic hazards, areas of concern (e.g. soft soils), and potential variability of local geology;
- Identify engineering analyses to be performed (e.g. limit equilibrium slope stability analyses, settlement evaluations, liquefaction susceptibility, lateral spreading/slope stability deformations,);
- Identify engineering properties required for these analyses;
- Determine methods to obtain parameters and assess the validity of such methods for the material type;
- Estimate the number of tests/samples needed and appropriate locations for them.

The goal of the site characterization for embankment design and construction is to develop the subsurface profile and soil property information needed for stability and settlement analyses. Soil parameters generally required for embankment design include:

- Total stress and effective stress strength parameters (friction angle, cohesion);
- Unit weight;
- Compression indexes (primary, secondary and recompression);
- Coefficient of consolidation.

Table 1 provides a summary of site characterization needs, and field and laboratory testing considerations for embankment design.

Table 1: Information Needs and Testing Considerations for Embankments
(Adapted from Sabatini, Et. Al., 2002)

Engineering Evaluations	Required Information for Analyses	Field Testing and Sampling	Laboratory Testing
• settlement (magnitude & rate) • bearing capacity • slope stability • lateral pressure • internal stability • borrow source evaluation (available quantity and quality of borrow soil) • geosynthetic reinforcement • liquefaction • delineation of soft soil deposits • potential for subsidence (karst, mining, etc.) • constructability	• subsurface profile (soil, ground water, rock) • compressibility parameters • shear strength parameters • unit weights • time-rate consolidation parameters • horizontal earth pressure coefficients • interface friction parameters • pullout resistance • geologic mapping including orientation and characteristics of rock discontinuities • shrink/swell/ degradation of soil and rock fill	• CPT (w/ pore pressure measurement) • SPT • piezometers (GWT and pore pressures during construction) • vane shear • geophysical testing • rock coring (RQD) • plate load test • test fill • settlement plates • slope inclinometers • Undisturbed Sampling	• Consolidation Testing (1-D Oedometer) • triaxial tests • unconfined compression • direct shear tests • grain size distribution • Atterberg Limits • specific gravity • organic content • moisture-density relationship • hydraulic conductivity • geosynthetic/soil testing • shrink/swell • slake durability • unit weight • relative density

The size, complexity and extent of the sampling program will depend primarily on the type, height, and size of the embankment(s) as well as the anticipated soil conditions.

Generally, embankments 10 feet or less in height, constructed over average to good soil conditions (e.g., medium dense to very dense sand, silt or gravel, stiff or overconsolidated clays with low expansion potential, with no signs of previous instability, non-liquefiable) will require only a basic level of site investigation. A geologic site reconnaissance, combined with a few shallow borings, hand holes or possibly a few test pits to verify field observations and the anticipated site geology should be sufficient, especially if the geology of the area is well known, or if there is some prior experience in the area.

For larger embankments, or for any embankment to be placed over soft or potentially unstable ground, explorations should be spaced no more than 500 feet apart for uniform conditions. In non-uniform soil conditions, spacing should be decreased to intervals to achieve at least one boring in each major landform or geologic unit or enough borings to

adequately define subsurface conditions. A key to the establishment of exploration frequency for embankments is the potential for the subsurface conditions to impact the construction of the embankment and the long-term performance of the finished project.

Embankments over 10 feet in height, embankments over soft soils, or those embankments that could impact adjacent structures (i.e., bridge abutments, buildings etc.); must have geotechnical borings (existing borings that provide the needed information will suffice) for their design. The more critical areas for stability of a large embankment are between the hinge point and toe of the slope. This is where base stability is of most concern and where most of the borings should be located, particularly if the near-surface soils are expected to consist of soft fine-grained deposits. At critical locations, (e.g., maximum embankment heights, maximum depths and/or thicknesses of soft strata), a minimum of two borings in the transverse direction to define the subsurface conditions for stability analyses should be obtained. Additional borings to define the stratigraphy, including the conditions within and below existing fill, may be necessary for very large fills or erratic site conditions.

Embankment widening projects may need borings near the toe of the existing fill to evaluate the present condition of the underlying soils, particularly if the soils are fine-grained.

In addition, borings through the existing fill into the underlying consolidated soft soil, or, if over-excavation of the soft soil had been done during the initial fill construction, borings to define the extent of removal, should be obtained to define conditions below the existing fill.

In some cases, the stability and/or durability of the existing embankment fill may be questionable because the fill materials are suspect or because slope instability in the form of raveling, down-slope lobes, or slope failures are present. In these cases, consider additional borings through the core of the embankment to sample and test the condition of the fill. If the distress is surficial, consider obtaining hand borings or grab samples for determining compaction and strength characteristics. The depth of borings, test pits, and hand holes will generally be determined by the expected soil conditions and the depth of influence of the new embankment. Explorations must penetrate through problem soils such as loose sand, soft silt and clay and organic materials, and at least 10 feet into competent soil. As a rule of thumb, geotechnical borings should be drilled to a minimum depth of twice the planned embankment height. However, in some cases the width of the embankment may have a greater influence on the stress distribution and/or magnitude of settlement than the height and therefore will require that borings be drilled to depths greater than twice the embankment height. In mountainous areas this minimum depth recommendation may not necessarily apply because bedrock and or competent soils may exist at depths shallower than twice the planned embankment height or less than what the width of what the proposed embankment would be. To ensure that the soils located under the proposed embankment are adequately studied, the Geoprofessional should consider the following:

- local and/or site specific geologic conditions,
- the height and width of the new embankment to be built,
- the pressure influence or stress distribution that the new embankment will impart on the supporting soils.

Cone penetration test (CPT) probes should, if practicable, be used to supplement conventional borings. Besides being significantly less expensive, CPT probes allow the nearly continuous evaluation of soil properties with depth. They can detect thin layers of soil, such as a sand lens in clay that would greatly reduce consolidation time that would otherwise be missed in a conventional boring. To utilize the sand lenses, they must be continuous and extend beyond the loaded area to provide drainage. In addition, CPT probes can measure pore pressure dissipation responses to evaluate relative soil permeability and consolidation rates. Because there are no samples obtained, CPT probes should be used in conjunction with a standard boring program.

Consider geophysical testing for void detection, water infiltration and seepage, and evaluation of subsurface variability. Geophysical testing can also provide nearly continuous coverage along the length of an embankment and may be used both to plan the drilling investigation and to extrapolate information beyond test boring and piezometer locations. Geophysical investigations should be considered where supplementation of the drilling investigation is needed.

2.2 Groundwater

At least one piezometer should be installed in borings drilled in each major fill zone where stability analysis will be required and groundwater is anticipated. Water levels measured during drilling are often not adequate for performing stability analysis. This is particularly true where drilling is in fine-grained soils that can take many days or more for the water level to equalize after drilling. Even in more permeable coarse-grained soils, the drilling fluid can obscure detection of the groundwater level. When encountered, groundwater must be measured after each boring is drilled. Information regarding the time and date of the reading and any fluctuations (such as loss of drilling fluid) that might be seen during drilling should be included on the field logs.

For embankment widening projects, piezometers are generally more useful in borings located at or near the toe of an existing embankment, rather than in the fill itself. Exceptions are when the existing fill is along a hillside or if seepage is present on the face of the embankment slope.

The groundwater levels should be monitored periodically to provide useful information regarding variation in levels over time. This can be important when evaluating base stability, consolidation settlement or liquefaction. As a minimum, the monitoring should be accomplished several times during the wet season (October through April) to assess the likely highest groundwater levels that could affect engineering analyses. If practical, a series of year-round readings taken at 1 to 2-month intervals should be accomplished in all piezometers.

The location of the groundwater table is particularly important during stability and settlement analyses. High groundwater tables result in lower effective stress in the soil affecting both the shear strength characteristics or the soil and its consolidation behavior under loading. The geoprofessional should identify the location of the groundwater table and determine the range in seasonal fluctuation.

If there is a potential for a significant groundwater gradient beneath an embankment or surface water levels are significantly higher on one side of the embankment than the other, the effect of reduced soil strength caused by water seepage should be evaluated. In this case, more than one piezometer should be installed to estimate the gradient. Geophysical methods may be employed to evaluate locations and the extent of seepage within an embankment. Also, seepage effects must be considered when an embankment is placed on or near the top of a slope that has known or potential for seepage through it. A flow net or a computer model may be used to estimate seepage velocity and forces in the soil. This information may then be used in the stability analysis to model pore pressures.

3 Embankment Design

3.1 Typical Embankment Materials and Compaction

General instructions for embankment construction are discussed in the specific construction specifications provided in Section 6, Control of Materials, and Section 19, Earthwork, of the Standard Specifications. Compaction requirements for bridge approach embankments are defined by Highway Design Manual (HDM), topic 208.11 and Figure 208.11A. Compaction requirements for embankments outside of bridge approach embankments are defined by Standard Specifications Sections 19-5, Compaction, 19-6 Embankment Construction, and 19-7 Borrow Material. Determine if any of the material from planned earthwork will be suitable for embankment. Consideration should be given to whether the material is moisture sensitive and difficult to compact during wet weather.

Landscape Architecture may request reduced compaction requirements from 90-95% relative compaction to 85-88% relative compaction to promote the growth of vegetation on embankment slopes. Careful consideration of this type of request should be made as well as discussion with all project stakeholders should be held to determine what will be acceptable slope performance.

3.1.1 Rock Embankments

Standard Specifications Section 19-6.03C, Placing and Compacting, discusses compaction and construction requirements of rock embankments.

Special consideration should be given to the type of material that will be used in rock embankments. In some areas of the state, moderately weathered to decomposed, and moderately soft to very soft rock (i.e. poorly indurated) may be encountered in cuts and used as embankment fill. Degradable fine-grained sandstone and siltstone are often

encountered in the cuts. The use of this material in embankments can result in significant long-term settlement and stability problems as the rock degrades, unless properly compacted with heavy tamping foot rollers (Machan, et al., 1989). The type, size, durability, and layer thickness of the rocky material will need to be considered. As compaction of a rocky fill is generally not measured with standard compaction equipment; a method specification will be required.

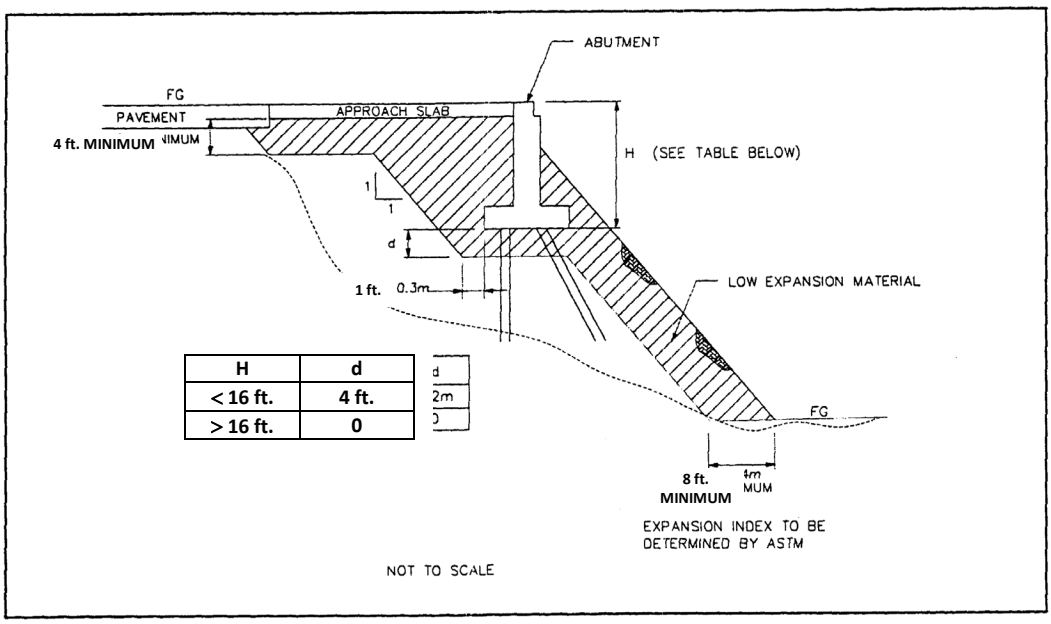
3.1.2 Earth Embankments and Bridge Approach Embankments

Two types of materials are commonly used in earth embankments: embankment fill (such as import borrow, local borrow, excavated material, lightweight imported borrow) and structure backfill.

Specifications for embankment fill and structure backfill are in Section 19 of the Standard Specifications. Specifications for compaction of embankment fill and structure backfill are in Section 6, Control of Materials, and Section 19, Earthwork, and also California Test Methods 216, 226, and 231.

Do not place expansive soil as part of the embankment within the limits of a bridge abutment as shown in Figure 1 for the full width of the embankment. Expansive soil materials for this requirement are defined as having either an Expansion Index (EI) (ASTM D 4829) greater than 50, or a Sand Equivalent (SE) (California Test Method 217) less than 20. This requirement is exclusive of the structure backfill and pervious backfill material requirements as shown on the plans and set forth in the Standard Specifications under Sections 19-3.02B and 19-3.03E, Structure Backfill, and 19-3.02C and 19-3.03G, Pervious Backfill Material, respectively. If you suspect that expansive soil might be available for use either locally or by import, include Figure 1 and appropriate discussion in the Foundation Report.

Figure 1: Expansive Soil Exclusion Zone



H	d
< 16 ft.	4 ft.
> 16 ft.	0

3.2 Fill Placement Below Water

If material will be placed below the water table, material that does not require compaction such as gravel or cobbles or boulders must be specified. Above the water surface, transition to standard embankment materials, using geosynthetics to prevent migration of the finer materials into the void spaces of the coarser underlying material.

3.3 Stability Assessment

In general, embankments 10 feet or less in height with 2H:1V or flatter side slopes, may be designed based on past precedence and engineering judgment provided there are no known problem soil conditions such as organic soils, soft/ loose soils, potentially unstable soils, such as Bay Mud or peat, or liquefiable sands. Embankments over 10 feet in height or any embankment on soft soils, in unstable areas/soils, or those comprised of lightweight fill require more in-depth stability analyses, as do any embankments with side slope inclinations steeper than 2H:1V. Moreover, any fill placed near or against a bridge abutment or foundation, or that can impact a nearby buried or above-ground structure, will likewise require stability analyses.

Prior to the start of the stability analysis, determine key issues that need to be addressed, such as:

- Is the site underlain by soft silt, clay or peat? If so, a staged stability analysis may be required.

- Are site constraints such that slopes steeper than 2H:1V are required (1.5:1 embankments are common in mountainous areas)? If so, a detailed slope stability assessment is needed to evaluate the various alternatives.
- Is the embankment temporary or permanent? Factors of safety for temporary embankments may be lower than for permanent ones, depending on the site conditions and the potential for variability.
- Will the new embankment impact nearby structures or bridge abutments? If so, more elaborate sampling, testing and analysis are required.
- Are there potentially liquefiable soils at the site? If so, seismic analysis to evaluate this may be warranted and ground improvement may be needed. For a bridge approach embankment or if seismic distress to the embankment would impact a bridge or building, then liquefaction (settlement, lateral spreading, and deformation) must be evaluated and the embankment should be designed to remain stable during seismic events. It is not common perform liquefaction mitigation for highway embankments due to the high cost of applying such a policy uniformly to all highway embankments statewide. In the latter case, if liquefaction is identified, a risk discussion must be held with the Project Development Team (PDT).

3.3.1 Safety Factors

The minimum Factor of Safety (FS) to be used in stability analyses for an embankment depends on many factors such as:

1. The degree of uncertainty in the stability analysis inputs
2. The level of investigation and data collection
3. Costs of constructing the slope to be more stable
4. Costs, risks to the travelling public, risks to the roadway, and other consequences should the slope fail
5. Whether the slope is temporary or permanent

Use the FS values below; however, higher or lower values may be appropriate, depending on specifics of the project and considerations listed above.

- Highway embankments (embankments that neither support nor potentially impact structures) should have a minimum factor of safety of 1.25. When repairing an embankment slide or slipout, and a factor of safety for the embankment can be reliably calculated, a minimum factor of safety of 1.15 may be used.
- Highway embankments supporting or potentially impacting structures should have a minimum factor of safety of 1.3.
- Bridge Approach Embankments and embankments supporting important (see MTD 20-1) structures should have a minimum factor of safety of 1.5.
- Under seismic conditions, only those portions of the new embankment that could impact an adjacent structure such as bridge abutments and foundations or nearby buildings require an overall minimum factor of safety of 1.1 using a

pseudo-static analysis. Utilize 1/3 PGA or 0.2 g (maximum) for the horizontal pseudo-static coefficient.

- Temporary embankments, i.e. short-term conditions during construction can be lower than long term factor of safety, typically about 1.1 to 1.2, but no lower than 1.1. Refer to considerations above.

3.3.2 Strength Parameters

Strength parameters are required for both stability and settlement analyses. Use FHWA Geotechnical Engineering Circular No. 5 (Sabatini, et al., 2002), the Soils Correlations Module, or other appropriate references for guidance on the selection of needed strength parameters. Obtain the parameters by a combination of laboratory testing and in-situ testing; in cases for low risk situations, soil correlations may be appropriate.

Tables 2 and 3 present the typical field and laboratory tests used to determine soil strength parameters. These tables are not exhaustive but do contain the most common tests.

Consider the applicable operating ranges of these tests, and that the values that these tests yield are dependent upon various rates and types of loading, boundary conditions, and stress history, etc.

Table 2 - Strength Parameters by Field Testing

Field Test	Parameter	Remarks
Standard Penetration Test (SPT)	Internal Angle of Friction	Values indirectly obtained by correlation (see Correlation Module). Only good for cohesionless soils. Discrete testing.
Cone Penetration Test (CPT)	Internal Angle of Friction, Undrained Shear Strength	Values indirectly obtained by correlation (see CPT module). Good for both cohesive and cohesionless soils. Continuous testing. No samples retrieved for lab testing.
Pocket Penetrometer	Undrained Shear Strength	Only good for cohesive soils. Good for estimating shear strength ranges. PP values may be confirmed with lab tests.
Torvane	Undrained Shear Strength, Residual undrained shear strength	Only good for cohesive soils. Good for estimating shear strength ranges. TV values may be confirmed with lab tests.
Vane Shear	Undrained Shear Strength, Residual undrained shear strength	Limited to soft and stiff clays.

Table 3 - Strength Parameters and Stress History by Laboratory Testing

Laboratory Test	Parameter	Remarks
Unconfined Compressive Strength	Undrained Shear Strength, (S_u)	Good for Clays and Silty soils; $S_u = C_u = q_u/2$
Triaxial	Undrained Shear Strength, (S_u), Effective Stress Parameters (c' and ϕ')	Good for both cohesive and cohesionless soils. Requires careful field sampling techniques. Most commonly, UU and/or CUe.
Direct Shear	Effective Stress Parameters (c' and ϕ')	Good for both cohesive and cohesionless soils. Requires careful field sampling/handling techniques. Shearing occurs over a pre-defined plane.
One-dimensional Consolidation	Overconsolidation Ratio (OCR), Preconsolidation Stress(σ_p'), Compression and Recompression Indices (C_c & C_r), Swelling Index (C_s), secondary compression (C_α), and Time-rate settlement values	Requires engineering judgment/experience to determine parameters.

If the critical stability is under drained conditions, such as in sand or gravel, then effective stress analysis using a peak friction angle is appropriate and should be used for stability assessment. In the case of over-consolidated fine-grained soils, the residual strength based on remolded down-hole vane shear or remolded lab tests, may be appropriate. This is especially true for soils that exhibit strain softening or are particularly sensitive to shear strain.

If the critical stability is under undrained conditions, such as in most clays and silts, a total stress analysis using the undrained cohesion value, based on a combination of field and laboratory tests, with no friction, should be used for stability assessment.

For staged construction, both short (undrained) and long term (drained) stability need to be assessed. At the start of a stage the input strength parameter is the undrained cohesion. The total shear strength of the fine-grained soil increases with time as the excessive pore water dissipates.

3.4 Embankment Settlement Assessment

New embankments, as is true of almost any new construction, will add load to the underlying soils and cause those soils to settle. The total settlement has up to three potential components: 1) immediate settlement, 2) consolidation settlement, and 3) secondary compression.

Settlement must be assessed for all embankments. Even if the embankment has an adequate overall stability factor of safety, the performance of a highway embankment can be adversely affected by excessive differential settlement at the road surface.

Settlement analyses for embankments over soft soils require the compression index parameters for input. These parameters are typically obtained from standard one-dimensional oedometer tests of the fine-grained soils. For granular soils, these parameters can be estimated empirically. Oedometer tests (consolidation tests) and the load increment ratios required for the tests must be determined/performed according to the American Society for Testing and Materials standard; ASTM D2435/D2435M – 11.

3.4.1 Settlement Impacts

Because primary consolidation and secondary compression can continue to occur long after the embankment is constructed, they represent the major settlement concerns for embankment design and construction. Post construction settlement can damage structures and utilities located within the embankment, especially if those facilities are also supported by adjacent soils or foundations that do not settle appreciably, leading to differential settlements. Embankment settlement near an abutment could create an unwanted dip in the roadway surface, or downdrag and lateral squeeze forces on the foundations.

If the primary consolidation occurs prior to placing utilities or building structures that would otherwise be impacted by the settlement, the impact is essentially mitigated. However, it can take weeks to years for primary settlement to be essentially complete,

and significant secondary compression of organic soils can continue for decades. Many construction projects cannot absorb the scheduling impacts associated with waiting for primary consolidation and/or secondary compression to occur. Therefore, estimating the time rate of settlement is often as important as estimating the magnitude of settlement.

To establish the target settlement criteria, the tolerance of potentially affected structures or utilities to differential settlement that will be impacted by the embankment settlement must be determined. Lateral movement (i.e., lateral squeeze) caused by the embankment settlement and its effect on adjacent structures, including light, overhead sign, and signal foundations, must also be considered. If structures or utilities are not impacted by the embankment settlement, settlement criteria are likely governed by the long-term maintenance needs of the roadway surfacing. In that case, the target settlement criteria must be established with consideration of the effect differential settlement will have on the pavement life and surface smoothness.

The amount of total and differential settlement that can be tolerated during and following embankment construction should be evaluated. The PDT (district designer, structure designer and geoprofessional) will determine the applicable design criteria on a project by project basis.

3.4.2 Settlement Analysis

Perform settlement analysis according to the NHI Soil, Slopes, and Embankment Design Manual, Chapter 4.

3.5 Stability Mitigation

A variety of techniques are available to mitigate inadequate slope stability for new embankments or embankment widenings. These techniques include staged construction to allow for the underlying soils to gain strength, base reinforcement, ground improvement, use of lightweight fill, and construction of toe berms and shear keys.

3.5.1 Staged Construction

Where soft compressible soils are present below a new embankment location and it is not economical to remove and replace these soils with compacted fill, the embankment can be constructed in stages to allow the strength of the compressible soils to increase under the weight of the new fill. For stability analysis for staged construction, refer to the NHI manual, Chapter 8.6, Embankments on Soft Ground and WSDOT Geotechnical Design Manual, 9.3.1 Staged Construction and its example in Appendix 9-A.

3.5.2 Base Reinforcement

Base reinforcement may be used to increase the factor of safety against slope failure. Base reinforcement typically consists of placing a geotextile or geogrid at the base of an embankment prior to constructing the embankment. Base reinforcement is particularly

effective where soft/weak soils are present below a planned embankment. The base reinforcement can be designed for either temporary or permanent applications. Most base reinforcement applications are temporary, in that the reinforcement is needed only until the underlying soil's shear strength has increased sufficiently because of consolidation under the weight of the embankment. Therefore, the base reinforcement does not need to meet the same design requirements as permanent base reinforcement regarding creep and durability. Use the creep reduction factors outlined in the WSDOT Geotechnical Design Manual, 9.3.2, Base Reinforcement and WSDOT Standard Practice T925.

The design of base reinforcement is similar to the design of a reinforced slope in that limit equilibrium slope stability methods are used to determine the strength required to obtain the desired safety factor. The design procedures by Holtz, et al. (1995) should be used for embankments utilizing base reinforcement.

Base reinforcement materials should be placed in continuous longitudinal strips in the direction of main reinforcement. Joints between pieces of geotextile or geogrid in the strength direction (perpendicular to the slope) should be avoided. All seams in the geotextiles should be sewn and not lapped. Likewise, geogrids should be linked with mechanical fasteners or pins and not simply overlapped. Where base reinforcement is used, the use of gravel, from imported or local borrow sources, may also be appropriate to increase the embankment shear strength.

3.5.3 Ground Modification

Ground modification can be used to mitigate inadequate slope stability for both new and existing embankments, as well as reduce settlement. The primary ground modification techniques to mitigate slope stability fall into two general categories, namely densification and altering the soil composition. Refer to the *Ground Modification* module for a more detailed discussion and key references regarding the advantages and disadvantages of these techniques, applicability for the prevailing subsurface conditions, considerations for construction, and costs. In addition to the two general categories of ground modification identified above, Prefabricated Vertical Drains (PVD) (also known as "wick" drains) may be used in combination with staged embankment construction to accelerate strength gain and improve stability, in addition to accelerating primary consolidation. PVD reduce the drainage path length, thereby accelerating the rate of strength gain. Other ground modification techniques such as stone columns can function to accelerate strength gain in the same way as PVD, though the stone columns also reduce the stress applied to the soil, thereby reducing the total strength gain obtained. Refer to the *Ground Modification* module for additional guidance and references to use if these techniques are to be implemented.

3.5.4 Lightweight Fills

Lightweight fill is another means of improving embankment stability. Lightweight fills are generally used for two conditions: the reduction of the driving forces contributing to instability, and the reduction of potential settlement resulting from consolidation of

compressible foundation soils. Situations where lightweight fill may be appropriate include conditions where the construction schedule does not allow the use of staged construction, where existing utilities or adjacent structures are present that cannot tolerate the magnitude of settlement induced by placement of typical fill, and at locations where post-construction settlements may be excessive under conventional fills.

Lightweight fill consists of a variety of materials including polystyrene blocks (geofoam), lightweight aggregates (rhyolite, expanded shale, blast furnace slag, fly ash), wood fiber, tire derived aggregate (TDA), and other materials. Lightweight fills are infrequently used due to either high costs or other disadvantages with using these materials. Refer to the Ground Improvement Module for more information.

3.5.5 Toe Berms, Shear keys, and Stabilization Trenches

Toe berms and shear keys improve the stability of an embankment by increasing the resistance along potential failure surfaces. As implied by the name, toe berms are constructed near the toe of the embankment slopes where stability is a concern. The toe berms are often inclined flatter than the fill embankment side slopes, but the berm itself should be checked for stability. The use of berms may increase the magnitude of settlements because of the increased size of the loaded area. Toe berms may be constructed using compacted earth embankment or from granular materials that can be placed quickly, do not require much compaction, but have relatively high shear strength. Use of compacted earth embankment may be warranted due to cost savings from using local borrow as compared to importing granular materials.

Toe berms increase the shearing resistance by:

- Adding weight, and thus increasing the shear resistance of granular soils below the toe area of the embankment;
- Adding high strength materials for additional resistance along potential failure surfaces that pass through the toe berm; and creating a longer failure surface, thus more shear resistance, as the failure surface now must pass below the toe berm if it does not pass through the berm.

Shear keys function in a manner similar to toe berms, except instead of being adjacent to and incorporating the toe of the fill embankment, the shear key is placed under the fill embankment—frequently below the toe of the embankment. Shear keys are best suited to conditions where they key can be embedded into a stronger underlying formation. Shear keys typically range from 5 to 15 feet in width and extend 4 to 10 feet below the ground surface. They are typically backfilled with quarry spalls or similar materials that are relatively easy to place below the groundwater level, require minimal compaction, and have high internal shear strength. Like toe berms, shear keys improve the stability of the embankment by forcing the potential failure surface through the strong shear key material or along a much longer path below the shear key.

Stabilization trenches function to key an embankment into competent foundation material and facilitate drainage within an embankment foundation. The specified

trenches significantly reduce the amount of excavation of poor foundation materials that might otherwise be required. See Section 8.B.8, Slope Stability and Foundation Investigation, 1973 for more stabilization trench details.

3.6 Settlement Mitigation

3.6.1 Over-excavation

Over-excavation simply refers to excavating the soft compressible soils from below the embankment footprint and replacing these materials with higher quality, less compressible soil. Over-excavation (remove and replace) should be assessed prior to consideration of other mitigation strategies, such as PVD, surcharges and lightweight fills. Because of the high costs associated with excavating and disposing of unsuitable soils as well as the difficulties associated with excavating below the water table, over-excavation and replacement typically only makes economic sense under certain conditions. Some of these conditions include:

- The area requiring overexcavation is limited;
- The unsuitable soils are near the ground surface and do not extend very deep (typically, even in the most favorable of construction conditions, over-excavation depths greater than about 10 feet are in general not economical);
- Temporary shoring and dewatering are not required to support or facilitate the excavation;
- The unsuitable soils can be wasted on site;
- Suitable excess fill materials are readily available to replace the over-excavated unsuitable soils.

3.6.2 Acceleration Using PVD

Refer to the *Ground Modification* module.

3.6.3 Acceleration Using Surcharges

Refer to the *Ground Modification* module.

3.6.4 Lightweight Fills

Refer to the *Ground Modification* module.

4 Instrumentation

Some of the more common instrumentation methods used to design and construct embankments includes the following:

- Piezometers – standpipe, vibrating wire
- Time Domain Reflectometry
- Slope Inclinometers

- Survey Hubs
- Settlement Platforms

Place as many instruments as needed to ensure that foundation soils provide adequate bearing for the embankment prism. Likewise, the embankment prism should be sufficiently instrumented to ensure settlement is complete during construction.

A more comprehensive discussion of these and other monitoring techniques is available in the Geotechnical Instrumentation for Monitoring Field Performance (Dunnicliff, 1993) and Geotechnical Instrumentation Reference Manual, NHI Course No. 13241 FHWA-HI-98-034 (Dunnicliff, 1998).

5 Reporting

Present planning phase embankment recommendations for highways and bridge approach embankments in the District Preliminary Geotechnical Report (DPGR).

Present embankment recommendations for highways and bridge approach embankments in the Preliminary Geotechnical Design Report or Geotechnical Design Report, as appropriate. The recommendations for the bridge approach embankments should be summarized in the Foundation Report so the Structure Designer is provided information as to any settlement, stability, waiting periods, PVD, etc. This will assist with any design impacts to the bridge, structure approach slab design and structure specification development. The details for the bridge approach embankment mitigation are typically put together by district design.

Present methodologies used and results for slope and foundation stability and settlement, and seepage if applicable. Present findings, conclusions and recommendations. For clarity, provide tables and/or figures such as plan views, elevations, and cross-sections, to detail the limits of specific recommendations so the client can prepare accurate plans, specifications, and estimates. Communicate regularly with district design units during PS&E process. Review the plans and specifications to ensure that they are prepared correctly and reflect the intent of the recommendations. Section 5.3 lists issues that need to be checked during the PS&E review.

5.1 Information to Provide for each Type of Recommendation

5.1.1 Loading rates

- Allowable height of fill for each stage and maximum rate of construction.
- Geotechnical instrumentation to monitor field performance and provide information relevant to decisions regarding the rate of construction.
- If instrumentation is required to control the rate of fill placement, explain how this will be done and how the readings will be used to control the contractor's

operation. This information is typically presented in a non-standard special provision (NSSP) produced by the geoprofessional.

5.1.2 Preloading, Surcharges (heights and periods)

- Geometry (size and limits of surcharge).
- Rate(s) of loading (1-ft per week, etc.).
- When the surcharge will be removed.

5.1.3 Settlement

- Settlement waiting period (duration).
- Whether construction of adjacent structures be delayed during embankment settlement period.
- Need for monitoring of adjacent structures.
- Provide magnitude and time of settlement with and without recommended measures.
- State whether settlement mostly completed during construction and estimate amount of settlement that is expected to continue after embankment construction, post construction total and differential settlements.
- Provide locations that have significant differential settlement that can affect the performance of highway embankment.
- Discuss post construction settlement and potential effect to structures and utilities located within the embankment, especially if those facilities are also supported by adjacent soils or foundations that do not settle appreciably.
- Discuss potential settlement of existing structure foundations as the new embankment is placed.
- Discuss time periods that the settlement should be monitored and the frequency of observations.
- List types of monitoring such as survey hubs, monuments, and settlement plates for vertical and lateral movement.
- List type of piezometers for pore pressure monitoring.

5.1.4 Over-excavation

- Location and geometry (depths, limits of removal).
- Backfill materials.
- Geosynthetic materials used for separation and/or reinforcement.

5.1.5 Lightweight Fill

- Type (s) of lightweight fill.
- Construction requirements.

- Geometric limits.
- Other needed materials and details (geosynthetic, drainage, etc).

5.1.6 Stabilization trenches

- Trench configuration and geometry, depth to and description of competent material.
- Backfill material and whether excavated material can be used as backfill.
- Location and dimension of drainage blanket, drainage material, pipe.
- Geosynthetic materials.

5.1.7 Buttresses/Berms

- Geometry (limits, height, width, slopes, cross-sections).
- Backfill materials.
- Drainage materials and details.
- Geosynthetic materials.

5.1.8 Surface Drainage

- Geometry (slopes, plans and cross-sections).
- Surface drainage design may include sheet flow, swales, ditches, catchment areas, and erosion control/slope protection.

5.1.9 Subdrainage

Trenches

- Geometry, size, type of geotextile, drainage pipe, type of trench back fill materials.

Horizontal Drains

- Location, length, vertical and horizontal angle, spacing, type of pipe and geotextile.

Drainage Blanket

- Slope, thickness, type of geotextile, size and type of drainage pipes.

Relief Wells and Drainage Galleries

- Location, spacing, diameter, depth, bottom bell out.
- Backfill material.
- Methods of water disposal i.e., pumping and discharge channels, or horizontal drains.

5.2 PS&E Review

Review PS&E to verify that the design and construction recommendations in the report have been properly incorporated into the plans and specifications. The following issues, where applicable, should be addressed in the PS&E:

- Slope inclination required for stability.
- Embankment foundation preparation requirements, over-excavation limits shown on plans.
- Plan details for special drainage requirements such as lined ditches, interceptor trenches, drainage blankets, etc.
- Special embankment material requirements.
- Special treatment required for fill placement such as non-durable rock, plastic soil, or lightweight fill.
- Magnitude and time for settlement.
- Settlement waiting period estimated in the Special Provisions.
- Size and limits of surcharge.
- Special monitoring needs.
- If instrumentation is required to control the rate of fill placement, do the SP's clearly spell out how this will be done and how the readings will be used to control the contractor's operation?
- Special Provisions state that instrumentation damaged by contractor will be repaired or replaced at no cost to the State.
- Settlement issues with adjacent structures, should construction of structures be delayed during embankment settlement period.
- Monitoring of adjacent structures.

6 Construction

When it comes to designing and building embankments, the Geoprofessional's role doesn't end once the GDR has been submitted. In fact, construction is typically where the Geoprofessional's role becomes even more critical.

The geoprofessional needs to make every effort to ensure that recommendations for embankment construction are constructible and that if any unforeseen construction problems arise, that they are resolved to not affect the schedule. A good working knowledge of the typical equipment used for embankment construction is important to constructability.

Geoprofessionals must make themselves available to Resident Engineers (RE's) whenever a problem with embankment construction arises. Geoprofessionals should attend relevant construction meetings called by the RE.

Typical problems that may be encountered during embankment construction include, but are not limited to, the following:

- Unexpected weak or soft soils encountered during embankment foundation preparation to receive embankment materials.
- Conflicts with placement of instrumentation (Settlement Platforms, vibrating wire piezometers, slope inclinometers, etc.).
- Conflicts with utilities or other facilities, and the equipment that a Contractor may use for a particular construction operation.
- Instrumentation data that appears erratic or unreasonable; instrumentation malfunction.
- Anticipated settlement not achieved during estimated timeframe. Items to discuss with the RE: Does increasing the settlement period become a critical path item? What are the risks? Are they acceptable?
- Problems with compaction – moisture, right equipment for the job, removal and replacement of problem soils with fabric and compacted soil.
- Springs, artesian conditions.

Construction issues, such as those above, illustrate the need to be flexible and willing to consider alternatives that will facilitate the progress and construction of an embankment.

Caltrans' Construction Manual, Construction Details, Earthwork (Section 4-19) is a good reference for construction activities performed during embankment construction.

7 References

1. Soil Slope and Embankment Design, FHWA-NHI-132033, September 2005.
2. Washington State Dept. of Transportation Geotechnical Design Manual, M 46-03.09, December 2013, Chapter 9 Embankments.
3. Geotechnical Instrumentation Reference Manual, NHI Course No. 13241 FHWA-NHI-98-034 (Dunnicliff, 1998).
4. Geotechnical Instrumentation for Monitoring Field Performance (Dunnicliff, 1993)
5. Subsurface Investigations – Geotechnical Site Characterization Reference Manual, NHI Course No. 132031 FHWA-NHI-01-031 (Mayne, Christopher, and De Long).
6. Caltrans' Construction Manual, Construction Details, Earthwork (Section 4-19), June 2012.
7. Slope Stability and Foundation Investigation, California Division of Highways, September 1973 (Forsyth and McCauley).
8. WSDOT Standard Practice T925, January 2009
9. ASTM D2435/D2435M-11; Standard Test Methods for One-Dimensional Consolidation Properties of Incremental Loading; American Society for Testing and Materials International (2011.05.01)

Landslides

This module presents the Caltrans practice for investigating, characterizing, analyzing, and mitigating landslides as well as the requirements for the preparation of geotechnical reports.

Landslides affect the safety, stability, and operation of California highways. Landslides occur in natural and man-made slopes affecting rural highways and major freeways. The Geoprofessional (GP) provides landslide-related recommendations to clients during all phases of the project delivery process, in support of Maintenance, and during emergency response. Landslide-related work requests come from numerous sources including District Design, Maintenance Engineering, Field Maintenance, and Construction.

Investigations

The purpose of a landslide investigation is to characterize the landslide, identify the factors that influence behavior, and facilitate development of mitigation strategies and appropriate geotechnical recommendations. The scope of the field investigation depends greatly on the size and complexity of the landslide, and the likely cost of appropriate mitigation. A landslide investigation typically consists of a literature review, interviews with personnel familiar with the site, field investigation (mapping and subsurface investigation), instrumentation, and monitoring. Chapter 7 *Organization of Investigation Process* of TRB 247 provides a description of the investigation process, considerations, and a checklist for planning a landslide investigation.

Understanding landslide movement and groundwater fluctuation are essential to accurately characterize a landslide. Therefore, a landslide investigation may require months or years to complete as time dependent data is collected by the monitoring of instruments. Consider the following when planning an investigation, instrumentation, and monitoring program:

- Plan the investigation to be proportional to the risks to public and worker safety, the impacts to operation of the highway facility, and the cost and complexity of likely landslide mitigation strategies. In other words, the investigation should not cost more than the mitigation.
- Collect data for a specific purpose that provides essential knowledge needed to provide risk assessment and design recommendations.
- Plan the investigation to fit within the timeframe for project development or assist the client in planning a project development schedule that allows for the appropriate investigation.

Preliminary Investigation

Review all information available in published sources and GeoDOG including reports, geologic maps, and relevant landslide maps from the California Geological Survey (CGS). CGS has a series of Highway Corridor Landslide Hazard Maps and corresponding reports for Caltrans.

Contact the local Maintenance office to obtain information about the site including history and current conditions. Maintenance typically will have records of work performed at the site. The conditions reported by Maintenance or other Caltrans personnel may not be evident at the time of the field investigation.

Field Investigation

For a small landslide, a field review may be adequate to characterize a landslide and allow for the determination of an appropriate mitigation strategy. The investigation of medium to large, complex landslides typically includes surface mapping, subsurface exploration, instrumentation and monitoring.

The use of Unmanned Aircraft Systems (UAS) can be used to provide enhanced visual perspectives, create three-dimensional terrain models, and monitor ongoing distress. The GP should consult with a GS UAS expert on the capabilities of this technology for the landslide investigation. Information on the UAS program and a list of Caltrans Remote Pilots is available on the Caltrans website.

Surface Mapping

Create a plan map of the landslide. Maps may range from rough sketches with approximate dimensions to detailed drawings on high-resolution terrain maps. The landslide plan map should show significant features including landslide boundaries, scarps, slope angles, materials and contacts, discontinuities, surface water, and seeps.

As-built or design plan sheets can be used as a base map for landslide mapping but may not extend far enough above or below the highway to show the entire landslide. Additional topographic data can be requested from Caltrans Surveys. Data collected using a Handheld Global Positioning System (GPS) unit can be used to generate topographic maps and augment existing Design survey data. Standard USGS 1:24000 topographic maps are generally too small scale to provide the required resolution for detailed landslide mapping; however, when no other base maps are available these topographic maps can be enlarged to a more useable scale using a photocopier. Digital terrain models generated from Light Detection and Ranging (LIDAR) data are excellent base maps for landslide mapping.

Exploratory Borings

Exploratory borings are commonly drilled for landslide investigations. Borings can reveal the location and geometry of slide planes, the groundwater surface, and the distribution of materials within the slide. Boring samples can be tested to provide soil strength parameters that greatly influence slide behavior and analyses of the slide. Borings also provide a means to gather downhole data and install instrumentation.

Locate borings within the slide mass to develop data on landslide geometry and movement that cannot be inferred from surface mapping or slide behavior. Large, complex landslides that threaten the highway and adjacent properties may be characterized using up to a dozen borings. Other large landslides that primarily affect the highway and adjacent undeveloped open space are commonly characterized by fewer than six borings. Smaller landslides may be characterized by performing as few as two borings.

Borehole sites on landslides are often difficult to access and may require the grading of access roads or placement of drilling equipment by helicopter. Such investigations are involved, expensive, and require the cooperation of numerous stakeholders. The total number and placement of exploratory borings is often dictated by the drill rig access or working platforms available to perform borings. When possible, locate borings near the middle and the point of greatest depth of a landslide. Also locate borings to identify undisturbed formation and stable ground above and below a landslide. Identifying the location and strength of stable ground can facilitate the design of mitigation features such as buttresses and retaining walls.

While uncommon elsewhere, downhole logging of bucket auger borings in landslides is practiced in Southern California. Bucket auger borings facilitate observation, measurement, and sampling of slip surfaces where the presence of groundwater or caving soil does not prohibit this technique.

In some circumstances, exploratory borings may be of little value in the evaluation and mitigation of landslides that have undergone a rapid mass movement event. Such landslides will generally be mitigated through mass grading. Prior to subsurface exploration, the GP should consult with the Branch Chief and Specialist to develop an economical exploration program that provides value to the evaluation and mitigation effort.

Gather disturbed or undisturbed soil samples for strength testing of materials likely to influence slide behavior or affect the performance of mitigation features.

Laboratory Testing

The laboratory testing program conducted for landslides is similar to testing programs conducted for the investigation of soil or rock cut slopes. The testing program should identify the material strength parameters that control the slide behavior. The strength of

material at the head scarp has less influence on slide behavior than the material along the middle and toe of the slide where effective stress has greater influence.

The choice of strength tests to perform should be determined based on the stress conditions in the soil in relation to the failure mode and failure surface. Shear strength parameters of cohesive soils should be obtained from undisturbed soil samples using consolidated undrained (CU) tests with pore pressure measurement if portions of the slope are saturated during failure. Effective strength parameters from these tests may be used to analyze the potential benefit achieved by dewatering. Unconsolidated undrained (UU) tests can be used to obtain undrained shear strength parameters for short term stability analysis including seismic stability, or when it is determined that total stress/strength parameters are sufficient.

Landslide behavior is often controlled by residual shear strength parameters of soil along the slide plane. When possible, determine residual shear strength by performing repeated direct shear tests or ring shear tests.

On many landslide projects, the laboratory testing program provides only limited information useful for the evaluation of slide behavior or the mitigation strategies. Slide behavior may be governed by thin discreet discontinuities. Even with significant laboratory data, the governing strength parameters are often estimated by means such as Hoek and Brown Failure Criterion or “back calculation” during model calibration. Strength parameters estimated through analytical methods should be compared to laboratory test results and reconciled when necessary. The results of previous landslide investigations in similar materials may serve as a basis for determining reasonable strength parameters.

Geophysical Techniques

Borehole and surface geophysical techniques can aid in characterizing the subsurface conditions of a landslide. Chapter 10 of TRB 247, *Subsurface Exploration*, discusses the various surfaced-based and borehole logging geophysical methods. Table 10-5 provides a list of borehole logging methods, their applications, and limitations.

Seismic refraction is a useful geophysical technique for investigating landslides. The velocity structure of a landslide mass; the depth to the failure surface; the lateral extent of a landslide; the location of contacts, faults, and discontinuities; and material rippability are variables that may be estimated using seismic refraction. Seismic refraction can help to interpolate conditions between boreholes. *Seismic Refraction Analysis of Landslides* (Narwold and Owen, 2002) discusses the use of seismic refraction in characterizing landslides and provides three case studies.

Instrumentation and Monitoring

Common instrumentation used by Caltrans in landslide investigations include slope inclinometers (SI), piezometers, crack meters, extensometers, and survey monitoring networks. The purpose of monitoring is to determine the depth to failure surface(s), direction of movement, rate of movement, aerial extent of movement, ground water elevations and fluctuations, correlation of rates of movement to rainfall and groundwater. Monitoring can also serve as part of a warning system during construction activity or for landslide management. The frequency of instrument measurement should be a function of the anticipated rate of movement or fluctuation.

SI are commonly placed in boreholes to identify the location of slide planes and the direction and rate of movement. SI equipped with time domain reflectometry (TDR) are often favored in landslides with more pronounced movement because this type of SI remains functional over greater displacements. SI equipped with in-place inclinometers featuring accelerometer sensors and remote data collection systems are practical for remote and difficult access sites.

Piezometers may be used to measure the occurrence and fluctuation of groundwater in or around the landslide. SI casing is sometimes modified to serve as both an SI and a piezometer. This dual instrument method requires some portion of the casing to be sand backfilled (as opposed to grouted) which may lead to bridging and voids around the casing thereby resulting in aberrant data.

Crack meters are typically low-tech monitoring systems that can be easily installed and monitored by the GP. Crack meters are typically lath, PK nails, survey stakes (hubs), or painted points/lines placed within the active slide mass paired with a similarly fixed monument placed on relatively stable ground beyond the active slide mass. Crack meters monitor the magnitude and rate of tension crack expansion by periodic measurement of distance (and sometimes inclination) by the GP. This technique can be used for short-term or long-term monitoring and is often employed during the initial measurements of landslide activity prior to the establishment of more elaborate monitoring systems.

Extensometers are essentially more formal versions of the crack meter system that use anchored wires or rods to measure relative displacement of the landslide mass.

Survey monitoring networks yield a more 3-dimensional representation of slide movement. The two basic types of networks used by Caltrans are:

1. A series of points located along straight lines that cross the slide boundaries. Progressive vertical and horizontal displacement of the points along the line provides data on the limits and the rate of sliding.
2. Reflectors or survey hubs placed at discrete points about the landslide. The initial survey of each point provides the basis to which all future survey readings are compared. The collected data provides information on changes to each hub location in the X, Y, Z directions. The data are used to determine rate and direction of movement of the slide mass, or differential movement within the slide mass.

To assure performance of landslide mitigation measures, some monitoring programs extend years beyond the close of construction. In this circumstance it is necessary to budget long term funding to maintain and read instruments, analyze data, produce reports, and act on the findings. Long term monitoring requires coordination between GS Management, Program Management and District partners.

Landslide Classification

Classify landslides based upon the landslide material and the type of movement. The type of material is reported as rock, debris, or soil. Type of movement is reported as either a fall, topple, slide, spread, or flow. The landslide classification should be reported in geotechnical reports as part of the description of a landslide.

A three-digit number in the following sequence identifying the *state of activity*, the *dominant type of movement* and the *thickness of the landslide* should be reported on the landslide map based on the criteria below:

State of Activity

1 = Active: Landslide is moving (currently active) or movements have been recorded in the past (historically active). Movement may be episodic. Ground surface exhibits fresh cracks, abrupt scarps, displaced or damaged man-made features and disrupted and/or young vegetation.

2 = Dormant: Geomorphic features related to the landslide are relatively fresh but there is no record of historic movement. Cracks are generally absent or eroded; scarps may be prominent but are slightly rounded. Landslide surface is vegetated, but vegetation may be younger than the vegetation on adjacent slopes.

3 = Relict: Geomorphic features related to the landslide have been greatly eroded and are subtle. Scarps are rounded. The landslide surface is covered with mature vegetation which is indistinguishable from the vegetation on adjacent slopes.

Figure 9-7 in TRB 247 provides diagrams to help determine the state of activity, based upon the geomorphic features of active and dormant landslides.

Dominant Type of Movement

Section 8 *Type of Movement* in Chapter 3 of TRB 247 describes the 5 five different types of landslide movement:

1 = Fall
2 = Topple
3 = Slide
4 = Spread
5 = Flow

Maximum Thickness of Landslide

The maximum thickness of a landslide can be estimated from field observations, subsurface investigation, cross sections, and geophysical data.

1 = Less than 5 feet
2 = 5 to 15 feet
3 = 15 to 50 feet
4 = Greater than 50 feet

Landslide Mitigation

Landslide mitigation strategies are grouped into four categories: Avoidance, Stabilization, Protection, and Management. The categories trend from higher to lower cost, effectiveness, planning, and design effort; and from projects that are generally performed as Capital Improvement to SHOPP to Maintenance. The strategies are often used in combination, and the measures can overlap from one category to another.

Avoidance Sometimes referred to as ***Relocation,*** involves measures such as roadway realignments, bridges, viaducts, retaining walls, and tunnels to separate the highway from the adverse impacts of the landslide. Avoidance allows the landslide movements to continue following mitigation, but those movements no longer affect the highway.

Stabilization Involves measures such as earthwork, buttresses, dewatering, retaining walls, shoreline armor, anchor bolts, slope contouring, and drainage systems to preclude or minimize further landslide movement.

Protection Involves measures such as rock sheds, rockfall barriers, draperies, rockfall fences, and catchment ditches to protect travelers and highway features from the collateral damage resulting from landslide processes. Protection measures typically control rock and soil emanating from a landslide.

Management Involves measures such as monitoring systems, patrols, planned road closures, signing, periodic maintenance, and minor rebuilding to allow operation of the highway within a tolerable amount of movement and disruption. Management measures are often practical for large, slow moving slides when the obstacles to other mitigation strategies prove insurmountable.

In practice, landslide mitigation often begins with Management, progresses to include the simpler, lower cost measures of Protection and Stabilization, and when the highway can no longer be operated safely and economically, the mitigation progresses to more robust measures of Protection and Stabilization. Avoidance is often the final solution when other

strategies prove non-viable, but in rare instances Avoidance can be the simplest strategy. The GP will perform Risk Assessment (Section 5) by considering all strategies. The selected mitigation strategy to assure a safe traversable roadway is chosen in partnership with District stakeholders.

Many of the landslide mitigation measures mentioned above are discussed in detail in Cornforth (2005), TRB Special Report 247 (1996), portions of the Geotechnical Manual specific to rockfall, retaining walls, etc., or are relatively self-explanatory. Three measures commonly practiced by Caltrans are briefly presented below:

1. Dewatering

Dewatering can influence the stability of a slide mass by both decreasing the driving force and increasing the resisting force. The decrease in driving force results from lowering the weight of the slide mass as saturation diminishes. The increase in resisting force results from the decrease of pore water pressure acting on the slide plane thereby increasing the effective shear strength.

Dewatering a landslide is generally accomplished in two ways:

i. Reduce the quantity of water entering the slide. Ravines and graben ponds often feed surface water into the head of a landslide. Grade and/or place drainage systems near the head scarp to divert surface water elsewhere.
ii. Remove groundwater from within the slide mass or surrounding terrain using vertical wells or horizontal drains.

2. Earthwork

Earthwork can be used for landslide mitigation in numerous ways:

- Decrease driving force by removing material at the head scarp.
- Increase resisting force by placing material at the slide toe.
- Increase resisting force by constructing buttress or shear key.
- Removing slide debris from roadway.
- Removing unstable material from slope.
- Grading to improve surface drainage.
- Grading to create catchment basins and berms.

Due to the relatively high impact and cost of material excavation, transport, and disposal, projects involving earthwork should strive to adhere to the principles of reduce, reuse, recycle, replenish, and dispose:

Reduce overall quantities by selecting maintenance and repair techniques and practices that reduce the overall footprint of disturbance and are the least disruptive.

Reuse suitable material on the same project, nearby projects, or for maintenance purposes. Organic material such as duff or topsoil may have appropriate surface

applications. Avoid the export of exotic plant material elsewhere. Minimize multiple handlings of the material.

Recycle material for non-highway uses on other approved public or private development projects or activities. Minimize multiple handlings of the material.

Replenish sediment supplies to natural systems by removing or bypassing man-made barriers. Replenishment may occur where the highway or highway management practices inhibit natural flow of sediment.

Dispose of excess material that cannot be put to any other beneficial use. Minimize multiple handlings of the material.

Trucking is the conventional method for excess material transport. Trips over 10-miles one-way from the source are considered long-haul trips increasing transport costs exponentially. These heavy truck trips on narrow winding highways result in pavement deterioration, reduced roadway safety, and the release of greenhouse gases.

Along sections of the coast where the ocean serves as a natural recipient of sediment generated by rapid erosion and mass wasting, several alternative material transport mechanisms may be considered. These mechanisms include mechanically depositing material over the side of the highway; loading the toe area of natural landslides for gradual redistribution of material by wave action thereby simulating natural processes; barging material to approved offshore ocean disposal sites; and pumping material into the surf zone or beyond to avoid burial of intertidal habitats. These alternatives require working with coastal stakeholders and are best implemented if planned for in advance of slide events.

3. Structures

Structures are often used as landslide avoidance measures. Soil nail or ground anchor walls can be used to isolate the roadway prism from the effects of a landslide on the slope below. Bridges, tunnels, and viaducts can be used to skirt over, under, and around landslides. Rock sheds can provide protection to travelers from landslides activity above the roadway.

Like dewatering and earthwork, retaining structures are used to stabilize a landslide by increasing the resisting force or decreasing the driving force. Ground anchor walls or shear piles may be placed at the toe of (or within) a landslide to exert resisting forces opposed to the direction of movement. In some cases, fill walls have been placed on the toe of a landslide to increase the effective stress on the slide plane. A strategically placed wall may allow removal of some, or all, of the slide mass.

Risk Assessment and Project Development

To formulate an appropriate mitigation strategy, all landslide mitigation efforts require risk assessment. Informal or preliminary risk assessment may occur during emergency response or early project development. Formal risk assessment is recommended for project design and the GDR. The GP must use the following generalized project development process.

1. Perform preliminary landslide review and characterization.
2. Identify the geotechnical conditions that influence instability.
3. Develop alternative mitigation strategies through a systematic consideration of Avoidance/Stabilization/Protection/Management measures.
4. Review the alternative strategies in consideration of the highway facility, constraints, and risks as listed here:
    a) Safety risks to travelers and Maintenance workers.
    b) Facility characteristic (traffic volumes, available detours, economic link, safety link).
    c) Consequence of immediate and long-term landslide activity.
    d) Complexity, schedule, and cost of alternative mitigation strategies.
    e) Complexity, schedule, and cost of project development.
    f) Relative effectiveness of alternative strategies.
    g) Necessary geotechnical investigation and data collection.
    h) Risks associated with insufficient geotechnical investigation (overly conservative and costly project, unstable project). Define the necessary investigation.
    i) Risks associated with insufficient project resources or schedule (repair may not address the problem). Define the necessary resources and schedule.
    j) Constructability issues: contractor, equipment, and material availability; complexity of construction; safety or workers and public during construction.
    k) Environmental Impacts.
    l) Future projects on the highway (i.e. widening).
    m) Aesthetics.
    n) Ongoing maintenance requirements and cost.
    o) Temporary vs. permanent mitigation.
    p) Does available funding allow for a project that mitigates the landslide or measurably improves corridor reliability?
5. Create a table of risks vs. benefits for the alternative mitigation strategies being considered. A decision matrix may be useful.
6. Present the alternatives, benefits, and risks to the project design team.
7. Select preferred alternative(s) for further development.
8. Perform the geotechnical investigation and analyses.

9. Based on unforeseen outcomes of the investigation and analyses it may be necessary to repeat portions of steps 2 through 8.
10. Finalize repair strategies, design, and construction recommendations.
11. Prepare Maintenance Support Memorandum or GDR as appropriate for the project.
12. Assist in the preparation of plans (formal or informal).
13. Identify construction safety risks and prepare construction safety plan or provide information adequate for the Contractor to prepare a construction safety plan.
14. Perform work (Maintenance and/or Construction activities). The GP will oversee and provide technical expertise for all landslide related construction activities.

In many cases it is impractical to completely stabilize a landslide; therefore, an assessment of the effectiveness of each strategy in conjunction with the associated risks and impacts will facilitate selection of appropriate mitigation. The selection may be documented in a decision matrix like Table 1.

Table 1: Landslide Mitigation Strategy Selection Matrix

Landslide Mitigation Strategy	Score = 0 - 10		Score = 0-5					Total: (Max. 45)
	Effectiveness	Cost	Traffic Impact	Environmental Impact	Right-of-Way Impact	Constructability	Construction Duration	
Total Removal of Landslide	10	2	0	3	4	2	1	22
Partial Removal of Landslide	7	4	1	5	5	4	2	28
Stabilize Landslide with Earthen Buttress	10	2	0	3	4	2	1	22
Stabilize Landslide with Horizontal Drains	2	4	2	5	5	2	2	22
Buttress with Anchored Soldier Pile Wall	10	4	5	2	3	2	2	28
Manage Landslide through Periodic Closures and Repaving	1	10	5	5	5	5	5	36

Landslide repair strategies are commonly estimated and programmed before a thorough field investigation starts, which results in major risks involving project funding and schedule. This can result in: 1) a geotechnical investigation that is overly abbreviated thereby leading to inaccurate assumptions and questionable analyses and design, or 2) a project that is insufficiently funded to perform the necessary mitigation. Early and thorough coordination with the District Project Development Team can minimize risks associated with estimating and programming.

The Coast Highway Management Plan (CHMP) contains a good qualitative description of considerations for several mitigation strategies for the Avoidance, Stabilize, Protection and Management approaches and provides input as to the likely project funding source.

Landslide Modeling

Landslide modeling employs charts, numerical equations, and/or computer programs to represent and simulate landslide geometry, soil conditions, and behavior. Given the complex nature of many landslides, the landslide model is an approximate representation of the actual landslide. The extent to which a model represents a landslide depends on the purpose of the model (preliminary models vs. final calibrated models), extent and validity of data gathered and used to construct the model, the modeling method employed, and the accuracy of the simulated slope failure mechanism. TRB 247 (Chapters 13-15) and Cornforth (Chapter 9) discuss slope stability modeling for landslids.

The purpose of landslide modeling is often twofold: 1) to better understand the mechanics of a landslide, and 2) to evaluate the effectiveness of proposed mitigation strategies.

Modeling Practice

Modeling may be performed for slides in soil, rock, and intermediate geomaterials. Landslide modeling differs from typical slope stability modeling in that failure limits and other critical conditions already exist and can be determined through site exploration, instrumentation, and testing. After the exploration program, models are developed to simulate the site conditions. To understand the slide behavior, the GP may attempt to simulate numerous slope configurations and ground conditions: the stability prior to the slide event; the instability during the slide event; the quasi-equilibrium following a slide event; and the effects of various mitigation alternatives.

The GP must identify appropriate stability analyses to represent the landslide. Landslide stability analyses are typically based on a two-dimensional cross-section of the slide, generally through or near the mid-width (Cornforth), but also for other cross sections likely to govern unstable behavior. Landslide models yield a factor of safety for a user defined cross section of materials, material properties, applied loads, and groundwater conditions.

Landslides may be relatively simple and the underlying mechanics clear, such as a rotational slide in a homogenous embankment fill that has been subject to a saturation event, or a translational slide of a large rock slab. Because the mechanism is understood, a rough model may be developed using slide geometry measurements and estimated

strength parameters. The model can then be used to evaluate the merits of various mitigation measures. This practice is common during many emergency response efforts that are progressing rapidly.

Complex landslides require iterative modeling to fully reveal the underlying mechanics. Once the mechanism of a complex landslide has been simulated and confirmed by model performance, mitigation strategies are applied to the model and evaluated.

Landslide models should be simplified to the greatest extent possible. Simple surface and subsurface contact geometry will minimize the potential for error. In general, a representative landslide model should be developed in a progression from simple to complex. More complex geometry, external loading, groundwater conditions, and structure should be added incrementally while observing output results for both anticipated and unexpected behavior.

If a landslide model represents the slope geometry, slide surfaces, distribution of materials, and discontinuities correctly, the only remaining variables are the shear strengths of materials and pore water pressure.

Preliminary Modeling

Preliminary and ongoing landslide models are of great value after the initial field reconnaissance and before the subsurface investigation. Preliminary modeling can indicate such parameters as the likely depth of slide surfaces and the zones of soil units that most influence slide behavior. Preliminary models can be developed based on initial field maps and cross sections, and estimated soil strength parameters derived from experience with similar materials and formations. Use preliminary model results to estimate the appropriate depth of borings and sampling locations.

Model Calibration

Landslides mechanics are typically modeled and calibrated by constructing a model that represents site conditions immediately prior to a slide event that has disrupted landslide equilibrium. Model parameters such as soil weight, strength, and pore water are subsequently adjusted within limits revealed by the investigation program until the model output indicates a factor of safety of 1.0. This process is often termed *back analysis*. B*ack analysis* often results in a greater certainty in strength parameters that are difficult to measure through testing. The shear strength of material along the slide plane is often determined through *back analysis.*

Models of active landslides should be calibrated at a factor of safety from 0.95 to 1.0. The GP should utilize supporting information in assigning a rational factor of safety value for a given slide simulation. A factor of safety less than 1.0 is sometimes assigned to models of slides that are moving.

In the *back analysis*, the GP should attempt to simulate the conditions that contributed or caused failure (such as high groundwater levels or a seismic event), as close as possible

to the actual conditions that existed at the time of failure. In the case of uncertainty, it is recommended that assumptions of more extreme conditions such as higher groundwater levels or ground accelerations not be made. Assuming a higher than actual piezometric surface or ground acceleration may lead to overestimation of the shear strength of materials.

Sensitivity Analysis

Model calibration includes a sensitivity analysis where model parameters such as piezometric surface, cohesion, or friction angle are held constant except for the single parameter being evaluated. This parameter is then varied within a reasonable range of values and the effect on model results is observed. This analysis reveals how sensitive the model is to changes of each parameter. Changes in some parameters may have little influence on model behavior while changes in other parameters may have great influence. The more sensitive parameters may indicate where investigation and testing should be focused. In general, changes in cohesion and piezometric surface will have the greatest effect on model stability. Additionally, models that appear to be overly sensitive or behave in a counter intuitive manner may contain errors.

Mitigation Modeling

Once a model has been calibrated, mitigation strategies are typically modeled by reconfiguring versions of the calibrated model to simulate the final slope geometry and the presence or effect of the various mitigation strategies. The relative increase in the factor of safety may be used to judge the effectiveness of a proposed mitigation.

It is important to recognize that, if a landslide has been sufficiently modeled, it is often not necessary for the governing strength parameters to be precisely determined to evaluate the relative effectiveness of various mitigation strategies. The GP should determine an appropriate amount of model development to suit the project.

For example, since it is difficult to test the in-situ soil strength of granular soils that may comprise a landslide, a landslide in granular material may be simulated using assumed soil strength parameters. Subsequently, various stabilization strategies such as slope buttressing, grading, and dewatering can be simulated thereby allowing comparison of the relative effectiveness of each strategy.

Common Modeling Methods

Refer to Cornforth (2005) Chapter 9, TRB 247 Chapters 13 through 15 to determine appropriate stability analyses to simulate a given landslide. Appropriate stability analyses may be relatively simple. Infinite slope analyses should be conducted for shallow slope failures in soil that roughly parallel the ground surface. Many translational landslides in rock may be modeled using relatively simple mathematics as block or wedge failures along distinct planer discontinuities. Landslides in homogenous materials with semi-circular failure surfaces may be quickly characterized using charts available in NAVFAC

DM-7 (2005). Landslides in transitional material may be modeled as rock and again as soil to determine the model that demonstrates the most representative behavior.

Develop landslide models using widely accepted, two-dimensional, limit equilibrium methods of slope stability analysis such as Simplified Janbu, Modified Bishop, Spencer, and Morgenstern-Price. While the latter two methods are preferred for slope stability modeling by many GPs, it should be recognized that the primary reason for modeling a landslide is to estimate the increase in stability that may be achieved by the proposed mitigation alternatives. Therefore, the method used to develop the calibrated landslide model should also be used to develop the models of various mitigation alternatives. The general modeling methodology is as follows:

- Use the information gathered from the landslide investigation to prepare critical cross sections of the landslide geology. The cross sections should include the ground surface profile, slide surface geometry, subsurface contacts, subsurface materials and their assumed or tested strength parameters, and groundwater surfaces.
- Consult geotechnical archives, colleagues, and specialists to discover previously developed models representing similar landslides. Use an appropriate model as an informal template.
- Using the data tables in the modeling software, enter the cross-section geometry, soil strengths, groundwater surfaces, and loading conditions.
- Define the critical failure surface, or the range of critical failure surfaces to be searched, as revealed by the investigation and critical cross sections. The failure surface may be translational, rotational, or irregular. If the developed information is insufficient to determine the landslide mechanism, develop models representing all suspected failure mechanisms. The mechanism resulting in the lowest factor of safety is the likely failure mechanism.
- Run the model to calculate the critical failure surfaces and/or determine factors of safety. The factor of safety of an active landslide should be near 1.0; however, the GP must apply judgement in evaluating the model output. Slightly higher or lower factors of safety may be rational model outputs depending on the conditions being modeled. For example, just prior to high rainfall that triggered a landslide, the representative model may have a low piezometric surface and a factor of safety slightly greater than 1.0. Raising the piezometric surface to simulate the influence of rainfall may result in a factor of safety slightly less than 1.0.
- Debug the model. One or more errors may have been incorporated into the initial model. For example: failure surface searches running through air may produce erroneous model results; vertical contacts may produce erroneous model results; data table input values may have been transposed; different strength values may have been applied to the same geologic unit. An error may cause the model run to abruptly abort. Irrational search limits may cause the model run to finish but with illogical results. For example, a significant length of failure surface passing through strong material as opposed to an adjacent weak material may indicate the

search limits did not allow a search of the weakest surfaces. It may require several runs and revisions to debug the model.

- Calibrate the model by adjusting model parameters within the range of uncertainty until the factor of safety of the active landslide is just below 1.0. Compare the calibrated model to site geology and test data to verify that the model construction and results are supported by the data gathered. For example, if you needed to drop the friction angle of a material below a reasonable value to simulate a failure, then there is likely some other part of the model that needs adjustment. Reevaluate the compiled data and gather more data if necessary.
- Use the calibrated model as the basis for development of modified models that simulate the effect of proposed mitigation strategies. It is helpful to remember that you are often not modeling the actual strategy but rather the effect of a strategy. For example, the effect of horizontal drains is not typically modeled by placing a drain in the model but rather by defining a lowered piezometric surface. Similarly, even though a ground anchor wall or shear piles may be simulated and depicted in the graphical output of the model, the wall may only affect the model as a defined load applied at a defined location in a defined direction. When applying forces to the limit equilibrium model, care should be taken so that all base forces remain compressive and no tensile forces are introduced.

Modeling Software

Programs commonly utilized for landslide modeling are GSTABL7, SLOPE/W, SLIDE, and RockPack. These programs can provide safety factor calculations by limit equilibrium methods. Most software packages include example files that model slope stability and mitigation that may aid the GP in model development.

Factor of Safety Criteria for Landslides

Cornforth (2005) Chapter 10 discusses the factor of safety and guidelines for setting minimums as a function of investigation rigor and landslide size. The suggested minimum static factors of safety range from 1.5 to 1.15. Slope stability factors of safety falling in the same general range can be found in numerous other sources. However, for the Caltrans GP, these target criteria are often no more than tenuous goals for highway related landslide stabilization. Landslides may be complex features with large dimensions that often extend well beyond the highway right-of-way. Geographic features such as mountains, rivers, and oceans may limit or preclude investigation and available mitigation strategies. Landslides may occur along remote highways that act as lesser or greater transportation links. Numerous stakeholders with competing interests and viewpoints may be involved in all aspects of remedial activities. Funding for landslide mitigation may be limited.

Modeling may reveal that available mitigation strategies provide only marginal improvement to stability, or possibly no improvement in stability. Some landslides may simply defy stabilization and creep relatively slowly or move in small, punctuated events.

The GP may ultimately provide recommendations that yield only a modest increase in stability, and some landslides may remain at a factor of safety of less than 1.0 following mitigation efforts.

For landslide stabilization efforts, static factors of safety of 1.3 should be achieved where reasonable. Elsewhere, the goal of landslide stabilization should be to attain the highest achievable factor of safety while working to satisfy stakeholders and working within geographic and budgetary constraints imposed on the project. At a minimum, any landslide stabilization strategy should strive for a minimum ten-percent (10%) increase in static stability.

Seismic factors of safety may be evaluated by applying a horizontal acceleration of 1/3 the peak ground acceleration (PGA) to the landslide model. Any landslide stabilization project should strive to achieve a minimum seismic factor of safety of 1.0. However, due to project constraints, the available mitigation alternatives may not provide a seismic factor of safety meeting this minimum target.

A seismic factor of safety evaluation should be conducted for all proposed landslide mitigation strategies regardless of the outcome. Such an evaluation will serve to fully document the designed slope stability condition. Proposed landslide stabilization designs should either meet minimum static and seismic stability targets or the proposed design should be supported by an appropriate risk assessment.

Reporting

Landslide recommendations are reported in either a Maintenance Support Memorandum, District Preliminary Geotechnical Report, Preliminary Geotechnical Design Report, or Geotechnical Design Report. The level of information provided should be commensurate with the project scope. Design parameters and recommendations for specially designed walls or structures are provided to Structure Design in a Foundation Report. Reports should include the following information:

- Description of slide geometry including the slide boundary and depth to failure surface
- Plan map and typical cross section of the unmitigated landslide
- Impact to existing or proposed facility
- Geological interpretations and factors causing the landslide
- Scope, duration, resource estimate for landslide investigation (preliminary reports)
- Recommendations for mitigation including the proposed mitigation footprint, plan view, and cross-section
- Analyses and procedures including the names and descriptions of software applications used for data reduction and interpretation
- Model description and output.
- List of viable repair strategies in conjunction with cost and risk assessment.

Various mitigation measures and the corresponding information that should be reported for each measure are listed below:

Drainage

- Surface drainage
  - Brow ditches
    - Location
  - Lined ditches
    - Location
    - Liner type
    - Cushion fabric
- Subsurface drainage
  - Horizontal Drains
    - Location (provide plan map)
    - Inclination
    - Length
    - Number
    - Spacing
    - Slot size
    - Orientation
    - Collector system
  - Drainage wells
    - Depth
    - Diameter
    - Permeable Backfill
    - Spacing
    - Location, number and inclination of outlet pipes
    - Diameters of belled out portions
  - Deep under drains
    - Location
    - Depth
    - Limits of UD
    - Location, number and inclination of outlet pipes
    - Permeable material

Earthwork

- Buttresses
    - Plan view and cross section showing geometry
    - Material type
    - Drainage details
    - Cut slope and finished grade slope ratios
- Stability Trenches/Shear Keys
    - Plan view and cross section showing geometry
    - Material type
    - Drainage details
    - Cut slope and finished grade slope ratios
- Slide removal
    - Plan map and cross sections
- Regrading
    - Plan map and cross sections
    - Drainage considerations
- Roadway Realignment (Avoidance)
    - Geologic map along proposed alignment

Structures

- Bridges/Viaducts
    - Location
    - Limits
- Retaining Walls
    - Location
    - Limits
    - Height/depth
    - Wall type
- Slope stressing
    - Location

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4. Coast Highway Management Plan (CHMP) – Guidelines for Landslide Management and Storm Damage Response – March 2004
5. C. F. Narwold, W. P. Owen, Seismic refraction analysis of landslides: Proceedings of the 2nd annual conference on the application of geophysical and NDT methodologies to transportation facilities and infrastructure, 2002, FHWAWRC-02-001.

Soil Cut Slopes

Soil cuts are excavated along natural hillsides, through ridges and mesas, and into existing embankment. Any slope excavated into existing fill, alluvium, colluvium, residual soils, or weak sedimentary formation is considered a soil cut slope. Slopes excavated into highly fractured and weathered rock may also be considered soil cut slopes.

Soil cut slopes may be preferred over other roadway prism widening alternatives such as retaining walls because they require less cost and time to build, generate material that may be needed elsewhere on the project, allow for future widening, or offer a pleasing aesthetic. Temporary soil cut slopes are often necessary to facilitate construction of embankments, bridges, retaining walls, and detours.

The stability of soil cut slopes can be assured through appropriate geotechnical investigation, analysis, and design thereby preventing landslides, slip outs, slumps, severe erosion, safety issues, operational interruptions, and long-term maintenance costs. These guidelines present the Caltrans practice for investigating, analyzing, and designing soil cut slopes.

Investigations

The evaluation of soil slopes, and the design of stable slopes, depends on accurate characterization of the geologic conditions that will influence slope behavior. The investigation should identify the parameters necessary to perform stability analyses and slope design. The exploration program should also be developed with consideration of the potential use of excavated material elsewhere on the project.

Desktop Review

Prior to gathering geotechnical data by other means, site information should be gathered by performing a desktop review. Proposed soil cut slopes are often located along existing highways, therefore previously developed information may exist. Common sources of valuable site information are in Table 1.

Table 1. Source of Historical Data (after FHWA)

Information	Source
Aerial photos	California Geological Survey (CGS), U.S. Geologic Survey (USGS), local libraries, local and national aerial survey companies
Topographic maps	CGS, U.S. Geologic USGS
As-built plans and Log of Test Borings for existing roadways and structures	Caltrans Document Retrieval System, OGD daily files and project files
Structure maintenance and inspection reports	Caltrans Bridge Inspection Records & Information System (BIRIS)
Prior geotechnical investigations and reports; storm damage or landslide investigations and reports	Digital Archive of Geotechnical Data (GeoDOG), OGD daily files and project files, USGS landslide website.
Climate and precipitation records; drainage	Cal. Dept. of Water Resources (CDWR), Cal. Climate Data Archive
Geologic reports and maps	USGS, CGS
Soil survey	USDA, SoilWeb (UC Davis website)
Flood insurance maps	FEMA, USGS, CGS, CDWR
Seismic Hazard Map and fault database	USGS, Caltrans ARS Online
Previous and present land use and development	Land owners, city/county land development offices

Review of aerial photos and topographic maps can provide large scale information about geological conditions at a site. In the field, large features are often obscured by vegetation and therefore difficult to identify by surface mapping alone. Aerial photos shot from various vantage points may reveal signs of unstable slopes including tension cracks, scarps, hummocky terrain, fresh ground disturbance, or an abrupt change in stream flow direction.

Site Reconnaissance and Mapping

Reconnaissance of the host proposed cut slope terrain provides important information for the development of an exploration program. Complex sites may require detailed geologic mapping. During site reconnaissance, the GP will observe the behavior and performance of existing cut slopes and natural slopes. The GP should measure and record the geometry of the existing slopes and record any sign or cause of slope instability. Landforms identified during the office review should be checked during site reconnaissance to determine the underlying geology.

Changes in ground surface slope angle may reflect differences in the physical characteristics of soil and rock materials or the presence of groundwater or seasonal seeps. The occurrence of plant species that draw water from permanent or seasonal

water tables indicates the presence of groundwater. Patterns in the vegetation may reveal patterns in soil type or seepage.

Subsurface Exploration

The subsurface exploration programs for soil cut slopes most commonly use geotechnical borings. Occasionally, Cone Penetration Tests (CPT) are used when the soils being investigated are not too dense, hard, or rocky, and level access exists. Seismic Refraction is sometimes used to estimate soil density and identify subsurface contacts. Subsurface exploration is conducted to: reveal soil types and geologic structure that cannot be otherwise determined; collect soil samples for laboratory testing; and gather groundwater information.

Consider the following when developing and performing an exploration program:

- Typically, one boring should be performed every 200 to 500 feet along each proposed cut slope greater than 20 feet high. The actual number of borings may vary based on the uniformity or variability of site geology and the quality of data that is acquired through other means
- At locations where more detailed slope stability analysis is necessary, additional borings perpendicular to the cut should be considered.
- Borings should extend a minimum of 15 feet below the bottom of cut elevation to allow for design changes that may deepen the cut and to provide adequate information on all soils that may influence slope stability analysis
- Boring depth should be increased at locations where base stability is a concern due to groundwater and/or soft or weak soil zones
- Borings should extend through any weak zones into competent materials.
- Auger borings will facilitate the collection of bulk samples at depth
- Field logging should include in situ testing such as Standard Penetration Test (SPT) for granular soils, and Pocket Penetrometer Test (PPT) and Vane Shear Test (VST) on cohesive soils. Undisturbed samples for lab testing may be collected in cohesive soils
- Samples should be gathered from the soil zones most likely to control slope behavior
- Hand augers, test pits, trenches, or similar means of exploration may be used in lieu of borings for investigating subsurface conditions for sliver cuts or shallow cuts, and where site access for large drilling equipment is restricted

Because site access is often very difficult at proposed soil cut slopes subsurface explorations are not generally conducted if a proposed slope will be cut into an existing slope where all the following conditions exist:

- The proposed slope is less than 20 feet high
- The proposed slope has the same or flatter angle of inclination as the existing slope
- The slope height does not increase significantly because of the cut
- There is no evidence of instability
- There is no evidence the material type is likely to be different at the new slope face
- The existing slopes provide good exposures of the underlying material
- There is no potential for seepage to be encountered in the cut

Groundwater Measurement

The presence of permanent or transient groundwater can greatly impact slope behavior. Borings will reveal the occurrence of groundwater at the time of the drilling. However, it may be necessary to install piezometers (open standpipe, vibrating wire, or pneumatic for groundwater pressure) to accurately determine the groundwater regime and to record any fluctuations. Ideally, the monitoring of piezometers should continue through at least one wet season. Continuous monitoring can be achieved by using electrical piezometers such as vibrating wire type in conjunction with digital data loggers.

The values of soil permeability and infiltration rate are generally determined based on correlations with grain size and/or knowledge of the site soil based on previous experience. However, borehole permeability tests, such as slug or pump tests may be conducted to facilitate the design of subsurface drainage features such as horizontal drains.

Laboratory Testing

Laboratory testing of representative soil samples is performed to determine the soil parameters that will influence slope behavior and the suitability of excavated material for reuse as compacted fill. The laboratory tests may include natural moisture content, grain size analysis, Atterberg Limits, corrosion, unit weight, shear strength, and R-value tests.

The choice of which type of strength test to perform should be based on expected stress conditions in the soil in relation to the anticipated failure mode and failure surface. Shear strength parameters of cohesive soils should be obtained from undisturbed soil samples using consolidated undrained (CU) tests with pore pressure measurement if portions of the proposed slope are saturated or might become saturated in the future. Effective strength parameters from these tests should be used to analyze long term stability of cut slopes or to evaluate long term effects on soil rebound upon unloading. Unconsolidated

undrained (UU) tests can be used to obtain undrained shear strength parameters for short term stability analysis including seismic stability, or when it is determined that total stress/strength parameters are adequate. Repeated direct shear tests can be performed to determine residual shear strength parameters for soils located in existing landslide areas. Residual strength parameter should also be obtained for cuts in heavily overconsolidated clay: the removal of overlying soil can release locked-in stress and allow the clay to deform, causing its strength to drop to the residual value.

Table 2 summarizes some of the engineering evaluations commonly performed for soil cut slopes, required information for the evaluations, and the field and laboratory tests conducted to obtain information.

Table 2. Information Needs and Testing Considerations for Soil Cut Slopes (After FHWA)

Engineering Evaluations	Required Information	Field Testing	Laboratory Testing
• Slope stability • Bottom heave • Lateral pressure • Dewatering • Pore pressure • Soil softening / progressive failure • Fault rupture • Liquefaction	• Subsurface profile • Groundwater • Shrink/swell properties • Unit weights • Moisture content • Permeability • Consolidation parameters • Shear strengths • Geologic mapping • Seismicity / faults	• SPT • CPT • PPT/VST • Piezometer • Geophysical testing • Test pit • Trench	• Mechanical analysis • Atterberg Limits • Moisture content • Consolidation test • Hydraulic conductivity • Strength test (unconfined compression, triaxial, direct shear, torsional shear)

It is usually very difficult to determine accurate soil strength parameters of all soils and structural weaknesses within a slope. Laboratory tests conducted on samples containing a structural weakness such as a fissured clay zone are sometimes performed; but field tests such as a torvane are likely to yield more useful results. Soil strength parameters are often estimated through slope stability modeling of known failure scenarios for a given site, a process commonly referred to as “back calculation.”

Factors That Influence Slope Behavior

Common adverse behaviors of soil cut slopes include landslides, surficial sloughing and creep, and severe erosion. Table 3 summarizes important aspects of the stability of cut slopes.

Table 3. Important Aspects of Stability of Soil Cut Slopes
(After Duncan et al. 1987)

	Cohesionless soil slope	Cohesive soil slope
Factors that control stability	Internal friction Unit weight of soil Slope angle Pore pressures External water	Strength of soil Unit weight of soil Slope angle Pore pressures External and internal water
Failure mechanism	Surface raveling	Deep sliding, possibly extending below toe of slope Sloughing of very soft clay
Critical stages for stability	Long-term or earthquake	End-of-construction, long-term, or rapid drawdown
Analysis procedure	Effective stress or dynamic	Total stress, effective stress, or combination

Slope behavior is a function of numerous factors:

- Geometry (height, angle of inclination, benching)
- Material properties (friction angle, cohesion, weight, permeability, pore pressure, expansion/contraction)
- Groundwater conditions (phreatic water, perched water, seepage, pore pressure)
- Soil structure (layering, layer orientation, weak surfaces, fissures)
- External forces (seismic ground motion, surface loads)
- Surface conditions (vegetation, concentrated runoff, adjacent water bodies, erosion control, slope armoring, expansion/contraction)

Most of these factors and the related slope behaviors are familiar to the GP. Common adverse behaviors and their influencing factors are discussed below:

Adverse Behaviors Related to Material Properties

Adverse behaviors of cohesionless slopes typically occur at shallow depths. Sands are very susceptible to surficial sloughing, shallow sliding, erosion, and piping. Adverse behavior of cohesive slopes may be shallow or deep. Expansive clay slopes may experience surface degradation and creep due to seasonal wetting and drying cycles. Clay slopes are prone to relatively deep circular sliding, the steeper the slope the deeper

the slide. Both natural slopes and fill embankments often consist of heterogeneous material which are difficult to represent by a simple set of soil parameters. Most soil slopes contain some amount of both granular and cohesive material and therefore can display mixed behaviors.

Adverse Behaviors Related to Surface Water and Groundwater

Most adverse slope behavior is greatly influenced by water. Concentrated storm runoff can result in severe slope erosion leading to a loss of structural support and catastrophic failure. Perched groundwater and infiltration from irrigation, rainfall, or snowmelt frequently causes surface sloughing and landslides. Water within a slope has the following effects on slope stability:

- Reduced shear strength of unsaturated soils by loss of apparent cohesion and reduction of friction between particles
- Increased seepage force acting downslope
- Reduced resisting force as saturation and increased pore water pressure results in reduced effective stress and shear strength
- Increased driving forces due to increased weight of the soil mass
- Degraded slope surface due to expansion/contraction and freeze/thaw

These negative consequences of water within a soil cut slope are generally more pronounced in slopes containing appreciable amounts of clay or silt.

The occurrence of groundwater will likely vary with the seasons and adjacent land use. Rainfall, irrigation, and surface water bodies influence the existence and location of perched and phreatic water tables, as does material type and soil structure.

Submerged or partially submerged soil cut slopes can present unique design challenges; however, such slopes are rare on Caltrans highway projects. The GP should consult appropriate design resources (e.g., Oregon Department of Transportation Geotechnical Design Manual Chapter 7) if the project involves submerged or partially submerged soil cut slopes.

Adverse Behaviors Related to Soil Structure and Residual Soils

Residual soils are derived from the weathering of rock; therefore, the behavior of cuts in residual soils tend to be influenced by soil structure. Slides are more likely to occur in residual soils along structural weaknesses inherited from the parent rocks such as clay layers or clay filled seams dipping out of slope. Fracture systems in the adjacent parent rock often convey groundwater to the slope through down gradient migration. High groundwater pressures commonly exist in intermediate geomaterial (IGM), partially weathered rock, and fresh rock. The groundwater may be confined by an impervious residual soil cover. If the soil cover is removed or reduced in thickness during a cut, a stability problem may develop.

For residual soils derived from claystone and shale, the most typical failure mode is a shallow slide associated with abnormally high groundwater in the underlying fissured shale. For residual soils of metamorphic and igneous rock, most slope failures occur in the upper layer and are usually related to high pore pressure due to rainfall. For residual soils originating from limestone and other carbonate rock, stability problems are usually related to sink holes, intense fracturing, and frequent interbedding of soft clay.

Adverse Behaviors Related to External Forces

Seismic events can induce significant inertia forces in a soil mass thereby triggering slope movement and landslides. In unusual circumstances, such as depressed highway alignments cut down into alluvial planes, soil cut slopes may be subject to liquefaction of the underlying soils. Soil cut slopes may be experience loading through the placement of a structure or embankment thereby increasing driving force and creating an imbalance with the resisting force leading to slope failure. All factors that will influence slope stability must be accounted for in soil cut slope design.

Soil Cut Slope Design

Soil cut slope design should follow Topic 304 of the Caltrans Highway Design Manual (HDM) and these guidelines. Soil cut slopes are often designed as compound slopes that include retaining walls at the top, bottom, or mid height.

Most permanent soil cut slopes inclined 2:1 are considered stable. However, the presence of unfavorable soil types, groundwater conditions, or soil structure may necessitate designs that may include flatter slopes, retaining structures, or dewatering. Steep soil cut slopes are often proposed where there is limited distance between grade separated highway features or limited distance between highway features and the right-of-way. Soil cut slopes may be inclined more steeply than 2:1 if the slope is relatively short (typically less than 15 feet in height) and displays strength derived from favorable remnant rock fabric, cementing, or cohesion. Slopes inclined steeper than 2:1 are common in weak sedimentary rock and residual soils derived from weathered rock. In these materials, the design of steep slopes will reduce the amount of direct rainfall and resulting erosion.

Cuts in overconsolidated cohesive soils consisting of stiff to very hard silt and clay of varying plasticity often contain pre-existing fissures, cracks, and other planes of weakness. These soils may stand at nearly vertical inclinations for a limited period. However, the relaxation of horizontal stresses in such soils can cause creep and lead to rapid failure. Slopes in such soils should be designed based on their residual friction angle and often must be inclined in the range of 1V:4H to 1V:6H.

Cut Slope Drainage Systems

The stability of soil cut slopes is often dependent on the appropriate design of drainage systems. Concrete lined drainage ditches above the top hinge and along slope benches

help prevent the erosive effects of concentrated flow. Surface drainage may also control seepage from a soil cut slope. Surface drainage systems are typically designed by District Hydraulics with input from the GP.

Subsurface drainage systems help control the adverse effects of groundwater. Horizontal drains, relief wells, cut-off trenches, or underdrains may be designed to diminish groundwater within or beneath a slope. Subsurface drainage systems are designed by the GP.

Horizontal drains consist of slightly inclined borings drilled into a slope and fitted with perforated pipe; the pipe may be wrapped in filter fabric. Horizontal drains intercept and drain granular water-bearing soils or fractures. Drain installation is difficult in soils containing boulders, cobbles, or cavities. Horizontal drains require periodic refurbishing or replacement as they tend to become clogged over time.

Relief wells are vertical dewatering wells equipped with pumps and plumbing. Relief wells require permits, electrical power, control systems, and periodic refurbishment, and may require that permanent fees be paid to the water owner; therefore, relief wells are often considered a last resort.

Cut-off trenches are lateral ditches filled with permeable material and collector pipes located near the top of the cut slope to intercept and convey shallow ground water around the slope. Underdrains are similar ditches located along the base of a slope intended to reduce pore pressure and control seepage.

Slope Surface Treatment

Vegetation should be established on slopes to prevent long-term erosion and create a pleasing aesthetic. The use of straw wattles, jute fabric, or other soil surface stabilization methods may be required to establish vegetation on slopes. Recommendations for erosion mats and plant establishment on soil cut slopes are typically provided by the District Office of Landscape Architecture; the commonly recommended systems are available in the Caltrans Erosion Control Toolbox. The GP will provide recommendations when more robust erosion control measures such as Cellular Confinement Systems, anchored mesh, or slope armoring are deemed to be necessary to stabilize the slope surface.

Temporary Soil Cut Slopes

Temporary soil cut slopes should undergo appropriate design and review. During project planning and design, the GP should evaluate potential temporary cuts that may influence construction staging, right-of-way requirements, the need for shoring, or the need for alternative designs. During project construction, it is often necessary for the GP to review temporary cuts proposed by the Contractor.

Temporary cuts inclined 1H:1V are common on Caltrans construction projects. These cuts in both natural slopes and embankment fill are typically stable in the short-term;

however, cuts greater than 25 feet in height are likely to experience problems if they exist for long periods or are exposed to extreme weather events. Steeper temporary soil cuts may be utilized by contractors when site-specific soil testing and analyses reveal the slope meets minimum stability criteria for temporary slopes. The design of all temporary cut slopes must follow Cal/OSHA requirements and the Caltrans Trenching and Shoring Manual.

Stability Improvement Techniques

Table 4 summarizes numerous methods that can increase the stability of a soil cut slope.

Table 4. Categories of stabilization techniques for soil cut slopes

Category of Stabilization	Stabilization Techniques
Load distribution	• Flattening slope • Slope buttressing
Surface/subsurface drainage	• Swale/ditch • Downdrain • Underdrain • Interceptor (cut-off) drain • Horizontal drain • Relief well
Slope protection	• Vegetation • Erosion control mats / blankets • Soil confinement systems • Anchored wire mesh • Rock / concrete riprap • Slope paving
Earth retaining systems	• Buttresses / berms • Gravity walls • Semi-gravity walls • Non-gravity cantilever walls • Soil nail / ground anchor walls • Prefabricated modular walls

Slope Stability Analysis

It is not necessary to perform stability analyses on all proposed soil cut slopes. Past performance of nearly identical slopes is the best predictor of future performance of new slopes. Stability analyses should be performed on all proposed soil cut slopes where stability is questionable. Cut slope stability may be questionable if one or more of the following conditions exist:

- Cuts greater than 20 feet high
- Cuts steeper than 2H:1V
- Cuts with irregular geometry
- Cuts within adverse or weak structure
- Cuts where high groundwater or seepage forces are present
- Cuts in weak soils
- Cuts above weak soil and high groundwater
- Cuts in old landslides or in formations known to be susceptible to sliding.
- Cuts in transition zones (cut/fill, soil/rock)
- Cuts above or below structures

Stability Modeling

Simple stability charts (NAVFAC, Abramson, et al. 1996) can be used for slope stability analyses to determine if a proposed cut slope will be stable. If the stability of a proposed slope is questionable, or if complex conditions of the proposed slope exceed the simplified conditions represented by the charts, more rigorous computer programs such as SLOPE/W, SLIDE, XSTABLE, ReSSA, are used by the GP. These programs use limit-equilibrium methods that satisfy forces and moments and allow the selection of various analyses methodologies including Bishop, Janbu, Spenser, Sarma, Morgenstern-Price, and general limit-equilibrium.

Limit-equilibrium analysis methods do not truly simulate the potential failure mechanism of a slope because all limit-equilibrium methods assume failure at a constant factor of safety along the entire slip surface. This simplified analysis neglects several factors including the stress-deformation behavior within the slope itself. Nevertheless, limit equilibrium analysis methods are widely used by the GP as a tool for slope design. More advanced computational methods such as finite element (e.g., PLAXIS) and finite difference methods (e.g., FLAC) are increasingly used to predict failure mechanisms and slope movements in two-dimensional and three-dimensional models. However, because of the heterogeneity of soil and the difficulty of precisely identifying subsurface contacts and soil strength parameters, more advance computational methods do not necessarily provide more accurate predictions of slope stability.

A stability evaluation is conducted for each unique, critical proposed slope configuration through the following process:

1) Develop representative cross sections for analyses. Critical slope configurations are commonly the tallest cut with the weakest ground conditions. The developed cross sections should extend well beyond the top of the proposed cut. The following information must be established for the cross sections:
    - Existing and proposed slope surfaces with vertical plane geometry coordinates.
    - Distribution of surface and subsurface soil units with vertical plane geometry coordinates
    - Estimated or established unit weight and strength parameters for each soil unit
    - Location of the water table and flow characteristics with vertical plane geometry coordinates
    - Location and strength of any structural weakness
    - Location and magnitude of external loading
    - Location and parameters of proposed stability enhancement features such as buttresses or retaining walls
2) For simple slopes use NAVFAC charts to evaluate stability. For more complex slopes use a more rigorous computer program. Develop computer models by creating electronic files representing each critical cross section created in Step 1.
3) Select the evaluation methodology and refine search criteria to debug the models.
4) Calibrate the models: Use any actual slope failure scenarios observed at the site to “back calculate” unknown soil strength. Refine the models to eliminate unreasonable results such as the weakest failure surface extending through the strongest material.
5) Run the program for each slope model to yield factors of safety for the proposed slopes.

Consider all short and long-term conditions likely to influence stability. Consider soil drainage characteristics and the loading condition. Stability analyses are generally categorized as:

- Drained – for long-term, or very slow loading or rapid draining conditions
- Undrained – for short-term or rapid loading conditions
- Intermediate – for staged construction or rapid drawdown conditions

If pore water pressures can be measured or calculated, effective stress may be used for slope stability analysis, otherwise total stress should be used. Table 5 summarizes the

analysis methods and corresponding shear strength parameters used for different loading conditions.

Table 5: Shear strength, drainage condition, pore pressure, and unit weight for slope stability analysis (after Duncan 1992)

	Undrained	Intermediate	Drained
Analysis procedure and shear strength for free draining soils	Effective stress analysis, using c and Φ'	Effective stress analysis, using c and Φ'	Effective stress analysis, using c and Φ'
Analysis procedure and shear strength for impermeable soils	Total stress analysis, using c and Φ from in situ UU or CU tests	Total stress analysis, using c_u from CU tests and estimate of consolidation pressure	Effective stress analysis, using c and Φ'
Internal pore pressures	No internal pore pressure for total stress analysis, set u=0	No internal pore pressure for total stress analysis, set u=0	Φ from seepage analysis
	Φ from seepage analysis for effective stress analysis	Φ from seepage analysis for effective stress analysis	
External water pressures	Include	Include	Include
Unit weights	Total	Total	Total

For cut slopes, the consolidation process can usually be assumed to be completed prior to construction. For such slopes, the short-term loading condition such as end of construction, rapid drawdown, or seismic loading, is best modeled using the intermediate analysis, whereas the long-term loading condition corresponds to the drained analysis. Occasionally, the undrained analysis may be warranted for cuts in soft cohesive soils where high excess pore water pressures are likely to develop due to construction activities.

The groundwater table within a planned cut section will fall due to the cut. As the water table falls, an increase in safety factor will likely occur. However, for the intermediate analysis, the water table should be modeled at its maximum anticipated elevation. Long-term (drained) conditions are modeled by assuming the groundwater table has fallen and excess pore water pressures have dissipated. The long-term condition is frequently the most critical concern for cuts in soils. Highly plastic clays may experience severe swelling and softening, and commonly exhibit significant potential for sloughing-types of slope

failures. Long-term conditions in these soils may be more accurately modeled by neglecting cohesion altogether.

It is important to identify and accurately model seepage within proposed cut slopes so that adequate slope and drainage are employed. For slope stability analysis requiring effective stress/strength parameters, pore pressures must be known or estimated. This can be done using one of several methods.

- Piezometers (phreatic or confined piezometric surface)
- Manually prepared flow net
- Numerical solution such as finite element or finite difference analysis (provided adequate boundary information is available)

The pore pressure ratio (Ru) can also be used. However, this method is generally limited to use with stability charts or for determining the factor of safety for a single failure surface.

Modeling of Seismically Induced Phenomenon

Model seismically induced inertial forces by conducting a pseudo-static slope stability analysis where the seismically induced inertia force is represented by a seismic coefficient equal to 1/3 of the estimated peak horizontal ground acceleration. Pseudo-static stability analysis can be performed using either the total stress or effective stress analysis. The effect of vertical acceleration on slope stability is ignored.

Model the effect of seismically induced liquefaction of soils underlying the proposed cut slope by assigning post-liquefaction residual soil strengths to the potentially liquefiable zones. The seismic coefficient should be set to zero in this case. The total stress analysis approach is more convenient for post-liquefaction stability analysis as it avoids the estimation of excess pore water pressures induced by seismic loading.

Factor of Safety

The static (or service state) factor of safety of soil cut slopes should generally conform to AASHTO LRFD Section 11.6.2.3 as stated below:

The evaluation of overall stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, Φ, may be taken as:

- *Where the geotechnical parameters are well defined, and the slope does not support or contain a structural element Φ = 0.75* (equivalent to a safety factor of 1.3).
- *Where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element Φ = 0.65* (equivalent to a safety factor of 1.5).

For pseudo-static (seismic) analysis of slopes unrelated to structures use Φ = 0.95 (equivalent to a safety factor of 1.05). For pseudo-static analysis of slopes that involve or are adjacent to walls and structure foundations use Φ = 0.9 (equivalent to a safety factor of 1.1).

For temporary soil cut slopes use Φ = 0.83 (equivalent to a safety factor of 1.2)

These factors of safety should be considered minimum values. The GP should decide whether a higher factor of safety is warranted based on the consequence of failure, experience with similar soils, and uncertainties in the analyses.

Use of Excavated Materials

Soils excavated from roadway cuts may be suitable for placement as structure backfill or embankment. The GP should test soil samples gathered from the sites of planned slope excavations to determine if the generated material will meet the gradation requirements detailed in the Caltrans Standard Specifications.

The GP should provide earthwork factors. Soil excavated from cuts and then compacted for embankment construction typically has a shrinkage factor. Shrinkage values vary based on soil type, in-place density, method of fill construction and compaction effort. Soil waste typically has a swell factor because material is often end-dumped at the waste site. Determine the shrinkage/swell factor for soil that will be reused by proctor tests. Corrections may need to be applied for oversized particles screened out of excavated material. Local experience with similar soils can be used to determine shrinkage/swell factors. Typical shrinkage/swell factors for various soils and rock are in Table 6.

Table 6. Approximate Shrinkage/Swell Factors (from Alaska DOT, 1983)

Material	In Situ wet unit weight (pcf)	Percent Swell	Loose Condition wet unit weight (pcf)	Percent Shrink (-) or Swell (+)	Compacted wet unit weight (pcf)
Sand	114	5	109	-11	129
Sand Gravel	131	5	124	-7	141
Silt	107	35	79	-17	129
Loess	91	35	67	-25	120
Rock/Earth Mixtures					
75% R/25% E	153	25	122	+12	136
50% R/50% E	139	29	108	-5	146
25% R/75% E	125	26	99	-8	136
Granite	168	72	98	-28	131
Limestone	162	63	100	+31	124
Sandstone	151	61	94	+29	117
Shale-Siliceous	165	40	118	+25	132
Siltstone	139	45	96	+9	127

Construction and Performance Monitoring

A construction monitoring program will ensure that the actual soil materials encountered in the field are the same as those considered in the design, the actual material strengths meet the design requirement, and the construction is performed in accordance with contract plans and specifications. If conditions are found that differ significantly from those anticipated, the GP can assist in the development of strategies that will assure slope stability.

Soil cut slope excavation should be carefully controlled during the wet season. Slopes that are susceptible to erosion should be immediately protected as they are exposed. It may be necessary to specify no delay between slope excavation and landscaping or armoring. Temporary drainage and/or erosion control systems are often necessary to protect cut slopes during project construction.

Excavated materials should not be stockpiled near the crest of a cut slope.

Cut slopes may be instrumented and monitored for short-term and/or long-term performance. Slope deformation may be monitored by periodic measurements of survey monuments and slope inclinometers. The function of drainage systems may be monitored by periodic measurements of piezometers. Slope deformation or rising groundwater may signal the need for a renewed evaluation of slope stability.

Reporting

Soil cut slope recommendations must be reported in accordance with the Geotechnical Design Reports module.

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Rock Cut Slopes

Rock cut slope design considers the structural characteristics and strength properties of rock masses to develop designs that address constructability and long-term performance.

The primary goal of a rock cut slope evaluation and design is to determine the steepest stable cut slope angle for a continuous slope, without intermediate slope benches. The design accounts for cut slope performance and safety while optimizing excavation quantities. Slope stabilization and rockfall protective measures may be required to minimize localized or small-scale instabilities within the overall cut. Environmental requirements, right-of-way impacts, and other project goals may at times also warrant slope stabilization measures. The design also provides excavation characteristics (blasting, ripping) and earthwork factors for material disposal or reuse. Three primary references used to develop this module are “Rock Slopes” by Manfakh et. al. 1998 (40), “Rock Slope Engineering” by Wyllie and Mah 4th Edition 2004 (51), and “Rock Slope Engineering” by Hoek and Bray 1980 (34).

Engineering judgment and experience in cut slopes should be applied in the investigation, analysis, and design. Experience with similar rock and slopes and/or case histories should be applied as appropriate

In addition to standard site reconnaissance and geotechnical investigation requirements, rock cut slope design relies heavily upon surface mapping, geomaterial identification, and discontinuity logging. Logging rock structure discontinuities (fracture/joint patterns) and their condition in boreholes and mapping them on surface outcrops is essential to rock cut slope design, as discontinuities strongly influence rock slope stability. Assessment of groundwater conditions in the rock discontinuities, as is true of any slope, is also critical to the assessment of stability (01, 28, 32, 49, and 40). “Rock Slopes” (40) presents an overview of design in Chapter 1 on pages 1-1 to 1-3.

Context sensitive designs are key to developing facilities that fit within the engineered setting, preserving scenic, aesthetic, historic, and environmental resources while maintaining motorist safety and mobility. Developing context sensitive solutions for rock slope design and rockfall mitigation is a stepwise process that should involve all stakeholders early in the scoping and design process. Excavation and mitigation design should consider the area’s scenic concerns, historical significance, and wildlife corridors, as well as the safety, cost, and capacity of its roadways. Design specifications should allow the project staff flexibility to modify the slope geometry and engineer mitigation method(s) that fit regional characteristics, roadway theme, and geological features. Context sensitive projects require frequent communication between the contractor and the project owner to ensure the final product meets all stakeholders’ interests (01).

Site Investigation

Assessment of slope stability requires observation of the site in the field. This can only be accomplished by hiking around, or in some cases accessing the site using ropes. The area must be studied both locally and regionally. A proper investigation consists of a literature search, review of historic and current geologic maps, aerial photo review and a thorough field investigation which must include a surface and may include a subsurface investigation(s).

Literature Search

Knowledge of the geology of the area is critical and can be obtained by examining published geologic maps and reports along with available aerial photographs. Maintenance records and interviews with personnel familiar with the site should be obtained wherever and whenever possible. The maintenance history, including a description of past problems and interim mitigation measures should be obtained. Where there are existing cut slopes obtain the “As-Builts” from the district office and the file of record, in the Geotechnical/District library, for any geotechnical investigations performed.

Field Mapping

For widening projects with existing cuts, reviewing the performance of existing slopes (both cuts and natural) in the area or region must be performed. For new alignments, field observations of naturally formed cuts in drainages or along the ocean and cuts outside the State R/W can provide valuable information where cuts are proposed in previously undisturbed areas.

The areas must always be mapped in detail on both the local and regional level. Geologic maps on a wide variety of scales are available from the USGS, CGS, local county and government entities and private organizations. The Landslide module should be referenced as appropriate. “Rock Slopes” (40) presents a good overview of planning an investigation program in Chapter 2 and in Appendix 2.

Collect the following information:

- Slope length (both horizontal and vertical): physically measure with tape, range finder, or pace.
- Slope Ratio: physically measure with Clinometer, Brunton or Clar Compass.
- Slope surface: smooth, planar, undulatory, and blocky; if undulatory or blocky, note the difference in height from the ‘peaks’ to the ‘valleys’.
- Describe lithology of the soil and/or rock.
- Estimate or physically measure native slope ratios above and below cuts, as well as on nearby slopes of similar geology. Note presence of boulders, including

average size, maximum size, shape, and rock type, as well as their location (top of slope, on slope (embedded or loose), at base of slope).

- Local Stability - Note erosion features, rockfalls, slumps, rills, washes etc.
- Global Stability - Note presence of landslides and rockslides.
- Note moisture features, such as seeps, springs, wet areas, and their likely causes.
- Assess condition of existing slopes.
- Describe vegetation.

Where applicable rock structure data should be collected and recorded. In addition to basic field observations a scan line analysis should be performed on open cut slopes or natural slopes. This information can be used in estimating the Rock Mass Quality (Q) and/ the Geological Strength Index (GSI). An organized table should be developed which includes the following information of the nature of the rock discontinuities. “Rock Slopes” (40) presents a thorough description of the following geological terms in Chapter 2, Section 2.3.

- Location
- Station
- Dip/Dip Direction
- Type of Discontinuity (bedding, jointing, fractures, faulting, etc.)
- Persistence
- Spacing
- Filling
- Asperities
- Roughness
- Rock Quality Designation (RQD)
- Discontinuity Sets
- Discontinuity Roughness
- Discontinuity Alteration
- Discontinuity Presence of water
- Weakness Zones intersecting the excavation

Existing Roadway Conditions

- Number of lanes.
- Lane width.
- Paved shoulder width and unpaved shoulder width, and slope ratio of both.
- Catchment Area (width of roadway between the base of the slope and the edge of traveled way available for rock and debris containment) and back slope ratio.
- Observations of pavement condition, presence and location of cracking, settlement, patching. Make educated guess as to why pavement is in the condition it is in.

Existing man-made structures

- Walls, foundations, building types, culverts, under-crossing, utilities etc.
- Note general conditions of each structure.
- Note if roadway work might impact the structure.

Provide notes on additional geologic investigations that may need to be performed to adequately characterize the site, ie: additional boring locations, geophysical studies, etc.

- Field drawings
    - Develop a scaled field drawing of the area of concern.
    - Develop one or more typical cross sections of the area of concern.
    - Develop cross section with initial mitigation approach if appropriate.
- Photographs of area of concern
    - Photos of problem area (direct/overall).
    - Photos of main features or any other important feature inside or outside of the area.

Subsurface Investigations

Determining the nature of the subsurface material provides valuable information for cut slope design. Two methods to obtain subsurface data which are readily available are drilling subsurface borings both vertically and horizontally and performing geophysical studies. Please also refer to the “Caltrans Soil and Rock Logging” manual (13) and reference (23).

Geophysical Studies

Seismic refraction studies are commonly used to determine rock structure and excavation characteristics, i.e. rippability and earthwork (grading) factors. Some references are provided. These studies should be combined with field mapping to refine estimates of rippability and grading factor. Seismic data may also be used to estimate varying rock quality and degree of weathering with depth when combined with nearby existing road cut observations, subsurface borings and experience. Acoustic televiewer, optical televiewer, and sonic and density downhole logs in a subsurface boring can be useful tools to evaluate the mechanical properties of the rock (03, 12, 14, 19, 32, 44, 45 and 46).

Subsurface Borings

Coring can be obtained by various techniques and is very valuable. Oriented core techniques, optical televiewer and acoustic televiewer downhole logging are valuable in determining the orientation of the rock structure at depth. General coring is valuable in

determining the characteristics of the geomaterials at depth and the changes with depth from the surface. This is especially valuable on new alignment projects where existing cuts do not exist. The geologist/engineer should assess the value of new coring versus the information already available from exposures in existing man-made and natural cuts, as well as existing outcrops, and weigh that with support cost and risk (23). Refer to *Geotechnical Investigations*. Also, FHWA “Subsurface Investigations” (23), Chapter 3, section 3.2 provides a thorough discussion on rock drilling and sampling.

Slope geomorphology/topography

Plan maps and cross sections are required for any project. The plan maps can be acquired by traditional surveying methods and/ or using LIDAR to generate digital terrain maps. Cross sections can be generated from the maps. Cross sections can also be obtained in the field using a tape measure, clinometer or a slope-a-scope. Where this data is not available preliminary slope angles and slope heights can be estimated using topographic maps and cross sections generated from the maps but these should be verified and/or refined based on site (slope) specific data (40).

Rock Strength Properties

The rock suitability for engineering applications is commonly expressed through indices such as the Rock Structure Rating (RSR), Modified Q-Rating System, Rock Mass Rating System (RMR), Unified Rock Classification System (URCS) and the Slope Rock Mass Rating System (SMS). Strength parameters can also be estimated from empirical methods (09, 35, and 49). Refer to the “Geotechnical Laboratory Testing” Section. FHWA “Subsurface Investigations” (23), Chapter 8 for an introduction to laboratory testing for rocks.

Rock Identification/ Rock Classification, (13, 21, 24 and 30).

- Igneous
- Metamorphic
- Sedimentary
- Transitional Materials

Rock Durability

Rock characteristics that affect long-term stability of the slope such as the degree of weathering, potential for weathering (physical, chemical), hardness, erodibility, and slake durability should be considered in the design. Durable rock is considered rock-like in character and relatively unweathered and hard. Nondurable rocks are rock-like in character in place but will weather during the proposed life of the cut to soil like materials. The depth and degree of weathering is dependent on the original material and weathering environment. Indices that might be used that can add to the rock durability quantification are the LA rattler test, and the slake durability test. Moh's hardness scale is also easily applied and evaluated (04, 13, 24, 29, 30, 35, and 40).

Rock Structure

The presence and characteristics of discontinuities, such as joints, foliations, shears, fractures, and faults are important factors in the stability of rock slopes. The orientation, frequency, persistence, and shear strength of rock discontinuities are measured in the field from existing cuts, rock core and/or outcrops. Projections of the dip and dip direction of the measured discontinuities and discontinuity shear strength are typically presented and evaluated on stereonets to determine if failure is kinematically possible. Hoek and Bray's "Rock Slope Engineering" (34) presents a good overview of the relationship between discontinuities and rock slope stability in Chapter 2.

If outcrops do not occur within the project area they should be sought on nearby slopes that are composed of the same formation. If nearby outcrops are not available, the "type section" of the formation is the next most useful resource in determining the rock structure.

When rock properties and discontinuities need to be investigated beyond their ground surface expressions or when no outcrops are available, coring exploration is used to obtain rock cores. Non-oriented cores provide information on the density, persistence, roughness, and infilling of discontinuities. Oriented coring exploration or down hole geophysics may be used to obtain information on the dip and dip direction of discontinuities (10, 20, 30, 34 and 41).

Rock Shear Strength

The shear strength developed along potential rupture surfaces within a slope is an important influence on the stability of rock slopes. The rock mass is the term used to describe the material on the rupture surface. Rock mass strength (UCS) and shear strength along discontinuities must be evaluated to determine potential for failures. If field mapping identifies adversely oriented rock structure it will be necessary to determine the shear strength of the discontinuities (friction angle and cohesion). This information can be obtained by laboratory testing or by reviewing published tables of rock strength parameters. Testing is typically performed using a direct shear box developed to accommodate rock core and similar sized rock samples. There are many tables to empirically estimate rock strength parameters that are available in the references. It is also important to determine the rock strength parameters of rock durability and compressive strength. Durability is typically measured with a slake durability test. This test is commonly used to establish weathering characteristics of the rock mass. Compressive strength can be obtained from rock core by performing an unconfined compressive strength test or a point load test on rock core or similarly sized rock samples (07, 24, 34, 40, 51 and 52). “Rock Slopes” (40) presents a good overview of rock strength properties throughout Chapter 3.

Hydrology

Groundwater and surface water conditions must be evaluated for the design and analysis of rock cut slopes. Hydraulic pressure acting within the discontinuities can cause significant destabilization by decreasing the shear strength due to uplift and/or increasing the driving forces acting on the block. Typically, the groundwater level within a slope can be estimated by observing seepages from and around the rock slope. When groundwater conditions are unknown, and groundwater is expected to influence the stability of the slope, groundwater pressures can be measured using piezometers (34, 40 and 51). “Rock Slopes” (40) presents a good overview of groundwater flow, permeability and pressure throughout Chapter 4.

Analysis

Slope Stability Analyses

The stability of hard rock slopes is typically controlled by discontinuities (joint and joint sets) within the rock. Failures tend to occur as discrete blocks. Discontinuities form intersecting planes of weakness. Without discontinuities, rock slopes, even some composed of relatively weak rock could stand hundreds of feet tall without the potential for failure. Kinematic analysis of the discontinuities is performed to determine the potential modes of failure (wedge, planar, toppling, and circular). This is followed by a static slope stability analyses to determine the factor of safety of potential failure modes. The stability of highly weathered to decomposed rock slopes is often not controlled by discontinuities, but rather by a Mohr-Coulomb failure mode (circular) more typical of soil slopes. Limit equilibrium analysis utilizing the shear strength and cohesion of the rock

mass is performed to determine the most likely failure surfaces and the global stability in terms of a factor of safety. Highly fractured rock slopes in which there is likely no single or distinct discontinuity surface on which movement will occur should also be analyzed in this fashion. These analyses require a value for φ (friction angle) and c (cohesion), which can be determined with lab tests such as a direct shear test (shear box) or by back analysis of similar slopes that have failed (25, 30, 33, 34, 37, 40 and 51). "Rock Slopes" (40) is a recommended reference for its discussion of the mechanics of rock slope stability (Chapter 1, Section 1.4) and plane (Chapter 5), wedge (Chapter 6), circular (Chapter 7) and toppling (Chapter 7) instabilities.

Kinematic Stability Analyses

A kinematic (limit equilibrium analysis) analysis is the first step in evaluating rock slope stability. Kinematics refers to the motion of bodies without reference to the forces that cause them to move (30). This analysis establishes the possible failure modes of the blocks that comprise the slope. The analysis determines if the orientations (dip and dip direction) of the various discontinuities will interact with the cut slope orientation and

inclination to form discrete blocks with the potential to fail. Failure modes typically fall within one of three categories: plane failure, wedge failure, or toppling failure. The analysis involves a comparison of the orientations of the dominant discontinuity sets with the orientation of the cut slope. Where discrete blocks are formed and where the failure surfaces that bound these blocks dip out of the slope (daylight) at an angle steeper than the shear strength along the discontinuity, failure is kinematically possible. A stereonet is used to perform a discontinuity analysis where the discontinuities and slope data are displayed. After the kinematic analyses have identified the most likely mode(s) of failure, the next step is to perform a stability analysis using the shear strength of discontinuities and groundwater conditions. The objective is to calculate the factor of safety of the slope or individual block being analyzed. Limit equilibrium methods for evaluating slope stability analysis focus on the relative magnitude of the resisting and driving forces. Hoek and Bray "Rock Slope Engineering" (34) presents a good overview of the stereonets, the graphical presentation of the data and kinematic analysis in Chapter 3. More advanced stereonet analyses are presented in Goodman (30) Appendix 5.

Discontinuity Analysis

Interpretation of the geological structure data requires the use of stereographic projections onto stereonets that allow the 3- dimensional orientation of data to be represented and analyzed 2-dimensionally. The most commonly used projections are the equal area net and the polar net. An important limitation of stereographic projections is that they consider only angular relationships between lines and planes, and do not represent the position or size of the feature (30, 34, 41, and 40). Chapter 2 in "Rock Slope Engineering" (51) presents a thorough review of a discontinuity and kinematic analysis. Figure 2.18 on page 38 presents a good graphical presentation of the limits of possible sliding for wedge, planar and toppling failures.

Seismic Considerations

Earthquake shaking can result in failures of cut slopes. The common types of instabilities prone to seismic-induced failure in rock cut slopes are rockfalls, rock slides and debris avalanches. The standard procedure for evaluating the stability of a nonliquefiable slope is the pseudostatic analysis, where, while performing a limit equilibrium analysis, a lateral force is applied to the center of gravity of a mass having a failure potential. The selection of shear strength in slope stability analyses involving seismic loadings should be based on short-term undrained shear strengths. The pseudostatic procedure does not provide an estimate of potential seismic deformations. In many instances, the stability of a slope during an earthquake may drop below a factor of safety of 1 for only a brief period of time during the transient shaking. In this case, a pseudostatic analysis would indicate an unacceptable factor of safety below 1, but the actual deformation of the slope may be minimal and the overall performance acceptable. For this reason, the effects of seismic shaking are not regularly considered in rock cut slope design.

When critical facilities are impacted and an analysis including seismic shaking is required one accepted method of estimating seismic deformations of non-liquefiable slopes is the Newmark Sliding Block Analysis. This method uses the yield acceleration of a slide mass and a seismic time history to estimate the permanent seismic deformation. This seismic analysis of slopes, however, is not used on a routine basis (26, 27, 40 and 51). Chapter 6, Section 6.5 in “Rock Slope Engineering” (51) presents a thorough review of the seismic analysis of rock slopes.

Rebound/Relaxation

When a slope is first excavated or exposed, there is a period of initial response resulting from the elastic rebound, relaxation and/or dilation of the rock mass due to changes in stress induced by the excavation (51). Newly excavated slopes commonly experience small local instabilities as the result of changes of the strain due to the removal of overburden. This condition is not necessarily indicative of long term instability, but rather local adjustment. Experience has shown this to occur within days or weeks or, in rare instances, months after construction (22, 30, 37, 43 and 51). “Landslides Analysis and Control” (42; p. 196-197), provides an overview of the loss of strength in the rock mass over time and the impact of residual and induced stresses on stability. Another good discussion can be found in “Rock Slope Engineering” (51) in Chapter 13, Section 13.2.

Convex and Concave Slopes

The radius of curvature can have an important effect on slope stability. In slopes that are concave in plan the horizontal tangential stresses tend to be compressive. In slopes that are convex in plan, the horizontal tangential stresses tend to be tensile. Rock is stronger in compression than in tension (34, 37, 43 and 51). “Landslides Analysis and Control” (42, p. 196), provides an overview of the effects of slope geometry on stability.

Factor of Safety (FS)

Typical target design Factor of Safety (FS) values range from 1.3 to 1.5; however, based on engineering judgment, values outside of this range may be appropriate, depending on the circumstances.

The minimum FS to be used in stability analyses for a specific rock slope depends on factors such as:

1. The degree of uncertainty in the stability analysis inputs; the most important being the amount of intact rock, rock mass strength, discontinuity spacing, discontinuity shear strength and groundwater conditions
2. The level of investigation and data collection
3. Costs of constructing the slope to be more stable
4. Costs, risks to the travelling public, risks to the roadway, and other consequences should the slope fail
5. Whether the slope is temporary or permanent

Rock Slope/ Landscape Integration

A major component of context sensitive design is the final appearance of an engineered slope: how the slope looks on its own as well as how it fits into the surrounding landscape. To create the best and most natural-looking results, engineers and designers typically use a combination of excavation techniques, and rock slope/landscape integration, which uses physical and cosmetic alterations to modify the shape of a slope and give it a more natural appearance by mimicking the surrounding topography. In some cases, safety or cost concerns make it impossible to achieve a completely natural look. However, in any case, the geo-professional and the contractor should strive to develop a slope that is safe, looks natural, and satisfies the interests of all project stakeholders. Factors to consider are slope warping, slope rounding, drainage, catchment ditches, slope angle, slope sculpting, and rock staining (01).

Design

Rock slope design consists of determining (1) the orientation of the cut, (2) the inclination of the cut, and (3) the need for mitigation measures if the resulting factor of safety is deemed too low or the rockfall potential onto the facility is unacceptably high and (4) the effects on the general physical character of the surrounding area of the proposed slope construction (01, 30, 34, 40 and 51).

Selecting Cut Slope Ratio(s)

Several factors affect how steep a rock slope should be cut, including the orientation and strength of the discontinuities within the slope, the height of the cut slope, the strength of the rock mass and discontinuities, the anticipated method of construction,

and whether additional measures will be used to enhance global/local slope stability and/or mitigate potential rockfall. “Landslides Analysis and Control” (43) provides a guide for selecting safety factors in the Existing Natural and Excavated Slopes section on page 197.

The strength and orientation of the discontinuities, relative to the azimuthal orientation and slope ratio of the proposed cut slope, will have the greatest and most direct effect on the Factor of Safety (FS). The height of the cut slope will also have an effect upon the global stability, with the greater height producing more driving force towards failure. If the FS does not meet the project requirements, a flatter slope ratio will be required, unless additional measures to mitigate instabilities are planned. When the primary discontinuities are oriented favorably for a steep cut, but the overall strength of the rock is low due to significant weathering and/or decomposition, a Mohr-Coulomb circular analysis will be required to determine the FS of the proposed slope inclination. Multiple slope ratios on a single cut may sometimes be the best and most efficient solution when the material shear strength increases with depth. For example, a cut slope determined by Mohr-Coulomb limit equilibrium analysis to have a sufficient FS might consist of soil overburden at a 1.2:1 slope ratio, overlying a zone of moderately weathered rock at 0.75:1, which, in turn, overlies a zone of unweathered rock at 0.5:1.

In some cases, steeper cut slopes may be determined through limit equilibrium analysis (kinematic and/or Mohr-Coloumb) to have a sufficient global FS but rockfall potential is found to increase. The slope designer should then determine if the positive effects of increased space at the toe of the cut slope (increased rockfall catchment and sight-distance) gained by steepening are more than sufficient to mitigate the negative effects of increased rockfall.

Some methods of slope construction damage the rock such that the finished cut slope has an increased likelihood of long-term local instabilities (rockfall). Uncontrolled blasting, for example, can cause fracturing and open existing fractures tens of feet into the slope. A finished cut slope can be constructed by excavating the rock using heavy equipment ripping or production blasting techniques. The use of either presplitting (preshear) or trim (cushion) blasting, in conjunction with production blasting, produces a cut slope with significantly less potential for local instabilities (rockfall).

Local experience with similar rock type should be investigated. In some cases, right-of way limitations or other factors, such as economics, may require the design slope to be steeper than desirable. If the resulting factor of safety is determined to be too low, or the potential for rockfall is estimated to be unacceptably high during the design life, rock slope stabilization and rockfall mitigation measures should be included in the design (01, 30, 34, 40 and 51).

Benches

Mid-slope benches are flat catchment areas typically constructed at regular elevation intervals within rock cuts (Highway Design Manual Section 304.3). The primary purpose of these benches is to control the degradation of the slope and contain rockfalls. They

are commonly employed within inter-bedded strata that contain weak, poorly indurated beds, such as shale or coal, that lie between more durable sandstone or limestone layers. For these applications, they are often referred to as lithologic benches. Lithologic benches are typically constructed at the base of the durable strata with variable widths, depending on the thickness of the underlying less-durable strata. The design attempts to account for degradation of the less-durable strata. For effective, long-term rockfall control, the bench needs to be placed directly beneath a near-vertical (0.25H:1V or steeper) section of slope. Like any catchment area, mid-slope benches require cleanout to maintain their effectiveness. In the 1970s, the widespread practice of incorporating mid-slope benches for highway cuts experienced a schism in North America. Prior to the onset of modern rock-slope engineering design, and the use of controlled blasting for the excavation of highway cuts in the early 1970s, slope designs for many high rock cuts used mid-slope benches. Due to a limited awareness of adverse rock-mass conditions and often poor blasting results, many of these slopes deteriorated and these benches filled with debris. Access difficulties, or loss of access due to slope failures, as well as insufficient maintenance resources, prevented their necessary cleanout. As a result, these debris-filled benches began to act as launching surfaces for rockfalls, imparting undesirable horizontal components to their trajectories. With improved characterization of rock-mass conditions and the use of controlled blasting, many designers, particularly in western North America, avoided the use of mid-slope benches, opting for uniform steep cuts that account for potentially adverse structural control, combined with ditches to control rockfall and with localized slope reinforcement, such as anchors or shotcrete. The elimination of mid-slope benches provided the added benefit of reducing excavation quantities. Where inter-bedded limestone/sandstone and shale sequences are common, mid-slope benches remain standard practice with decades of proven effectiveness in containing rockfall, when properly maintained (49 and 51).

Excavation

Excavation of rock cut slopes is done by blasting the rock, removing the rock (ripping) with equipment such as tractors and excavators, or by the use of chemical or hydraulic expanders. Rock excavation for highway construction often requires the formation of cut slope faces that will be stable for many years, and that will also be as steep as possible to minimize excavation volume and land use. While these two requirements are somewhat in conflict, the stability of cuts will be enhanced, and the steepest safe slope ratio increased by using the proper excavation method that does the least possible damage to the rock behind the final face (01).

Ripping

The physical characteristics that affect rippability are:

1. Frequent planes of weakness
2. Weathering

3. Moisture
4. High degree of stratification
5. Brittleness
6. Low strengths
7. Low seismic refraction velocity

Ripping refers to the removal of rock using tractors, excavators and scrapers. Not all materials or formations can be ripped. The factors used to determine rippability are collected during the field investigation. Three important studies need to be done in order to determine rippability: 1) rock mechanics analysis, 2) geologic site inspection, and 3) seismic refraction evaluation. Determine the conditions to be considered in determining whether ripping is appropriate. First and foremost, the geomaterial must be rippable. Secondly, will blasting damage the slope face or cause damage to the surrounding structures or environment? Finally, which is more cost effective, blasting or ripping (01, 31 and 45)?

Blasting

The physical characteristics of a rock mass that favor blasting over other methods of excavation are:

1. Massive
2. Crystalline rock
3. Non-brittle, energy absorbing rock fabrics
4. High strengths
5. High seismic refraction velocity

The preferred method of blasting for rock cut slopes employs lightly loaded, aligned, and closely spaced blast holes that form the final cut slope face in a manner that minimizes the effects of the intense detonation gas pressures caused by production blasting. This technique, which is referred to as presplitting or trim blasting, is performed either before the main production blasting is detonated (presplit blasting) or after the production blasting (trim blasting). In presplit blasting, the row of control blast holes is detonated to form a break in the slope along the final cut slope, which serves to vent production gas pressure and keep it from penetrating and damaging the rock that will form the final cut face. In cushion blasting, the row of control blast holes is detonated last, to trim off the rock outside the cut slope. The cushion blasting technique is most commonly used in weaker rock conditions or wherever the thickness of rock to be excavated is less than 15 feet. Controlled blasting is routinely used for rock cuts that are 0.75H: 1V or steeper. Maintaining blast hole alignments on flat slopes is a limiting factor when considering the use of controlled blasting (01, 15, 36, 40 and 51).

Rock Slope Stabilization

Mitigation measures to enhance stability include rock reinforcement, drainage, rock removal, and protection against rockfalls (01, 25, 34, 40 and 51). “Rock Slopes” (40) serves as a good single reference for stabilization and blasting; however, all the listed references provide information on the various stabilization measures.

- *Reinforcement* - Structural reinforcement can be provided by rock bolts, dowels, and cable lashing. Tensioned rock bolts are used to increase the normal stress along the discontinuity where sliding is possible, thus increasing the shear strength of the discontinuity. They may also be used to anchor potentially unstable rock blocks in place. Dowels are untensioned rock bolts or shear pins used to resist lateral movement of rock blocks by their shear capacity. Cable lashing uses tensioned cable(s) to increase the normal force against the face of an isolated block to increase sliding resistance. Other designs to consider are shear keys and ground anchor walls (01, 02, 46, 49, 40, 51 and 52).
- *Buttresses* - When an overhanging rock is large and it is impractical to remove or reinforce it, buttresses can be used to support the overhanging rock and increase its stability. Buttresses serve two functions: (1) protect or retain underlying erodible material, and (2) support the overhang.
- *Rock Removal* - One method to mitigate an unstable rock slope is to remove the potentially unstable rock by resloping and unloading the rock mass by rock scaling, trim blasting, or other excavation techniques. In the construction of new rock cuts, rock scaling is generally required and treated as incidental to the payment for the type of excavation performed. Rock trimming can be done with light explosive charges, hydraulic splitters, chemical expanders or pneumatic hammers (01 and 49).
- *Drainage* - Dewatering to reduce groundwater pressures acting within the rock slope improves slope stability. Reduced groundwater pressure within a discontinuity increases the shear strength, while lowering the groundwater height within tension cracks reduces the driving force on a rock block. Proper drainage of rock slopes can be achieved by installing drain holes (weep holes, horizontal drains) or vertical relief wells. Various measures, such as construction of surface drains and ditches, minimize water infiltration, thereby preventing the buildup of groundwater pressures (05, 15, 19, 38 and 48).
- *Erosion Protection* – Hard rock is typically resistant to erosion and/or erodes over long periods of time i.e., 100’s or more years. Soils, decomposed rocks, highly fractured rocks, and certain types of rocks that are susceptible to erosion or degradation can erode on the order of days, weeks and months. When hard rock that is resistant to erosion is underlain by an erodible or degradable layer the loss of support of the overlying rock may develop over time through differential erosion, which may create an unstable condition. Stopping this process can be accomplished by applying shotcrete to the surface of the less resistant zones or by applying an anchored mesh to the slope face. Weep holes are installed to prevent the buildup of groundwater pressures behind the shotcrete where the

anchored mesh is free draining. Where applicable (soils and some transitional geomaterials) sustainable erosion control measures can be implemented (01 and 17).

- *Protection Measures*– Draped mesh systems apply limited normal force against the rock face, and primarily serves to control the descent of falling rocks into the roadside catchment area. Barrier systems stop rocks during the rocks' descent. Barrier types can range from rigid concrete, timber, gabion walls to flexible fencing (01, 11, 49, 40 and 51).

Reporting

Rock cut slope recommendations must be reported in accordance with the Geotechnical Design Reports module. Reports must discuss the purpose for the work, field methods used, and techniques used for data processing and interpretation. Names and descriptions of any software applications used for data reduction and interpretation must be presented. Maps and cross-sections must be provided. Features in the geological models that may affect a project must be noted and discussed in the report. Geological interpretations and models must be adequately documented.

Reports specifically pertaining to the rock cut slope design should include the stereo net graphical diagram with the cut slope great circle, the proposed cut slope great circle and the friction angle circle all shown. The report should also include a table indicating the excavation characteristics of the rock, i.e, rippable, moderately rippable, difficult ripping to light blasting, or blasting required. This table should also include earthwork factors as they relate to the seismic velocity layers and excavation characteristics.

Station	Velocity Layer	Thickness	Excavation Characteristics	Earthwork Factors
0+00 to 10+00	V1	10 feet	Rippable	0.9
0+00 to 10+00	V2	10 to 30 feet	Blasting Req.	1.1

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Rockfall

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1. INTRODUCTION

Rockfall investigations may be required for STIP projects, emergency storm damage, and normal slope maintenance operations and can involve phases K, 0, 1, 3 and M1 and M2 tasks. This module provides the process and information needed to complete a rockfall investigation.

Rockfall is the movement of rock of any size from a cliff or slope that is steep enough for the rock to move down slope. Movement may involve any combination of free falling, bouncing, rolling, or sliding. Rockfall may involve more than one rock, but excludes slope failures involving large volumes of rock, such as rock avalanches or landslides (26).

Rockfall has the following characteristics (38):

- The event involves a single block or group of blocks that become detached from the rock face.
- Each falling block behaves independently of other blocks.
- There is temporary loss of ground contact and high acceleration during the descent.
- The blocks attain significant kinetic energy during their descent.

Three publications frequently referenced herein are listed below. These and all other referenced documents are available via hyperlink in the References section at the end of this module.

- Turner and Schuster (2012), *Rockfall Characterization and Control*,
- Brawner (1994) *Rockfall Hazard Mitigation Methods*
- McCauley et.al. (1985) *Rockfall Mitigation*.

Experience, especially with similar rock and slopes, should be considered in rockfall analyses and design of mitigation measures. Case histories in similar conditions should be consulted to provide additional guidance (38). The Geoprofessional should consult with their Office's rockfall technical team representative for all rockfall work.

2. CONTEXT SENSITIVE CONSIDERATIONS FOR ROCK SLOPES

Development of context sensitive solutions for rockfall mitigation is an important consideration in the planning and design process and includes all stakeholders. Mitigation design considers the safety, cost, and capacity of roadways as well as areas of scenic concerns, historical significance, and wildlife corridors (01 & 08).

3. ROCKFALL TYPES AND CAUSES

The first steps in performing an investigation and developing an effective mitigation plan are to (1) identify the type of rockfall, and (2) determine the cause.

Rockfall Type

There are numerous conditions that can affect a slope and produce rockfall. A guide to the types of failures is provided in Turner and Schuster (2013) on pages 25 and 26. Rock failure mechanisms are categorized into four types (07, 26, 30, 37, 38, 42 & 43):

- Planar
- Wedge
- Toppling
- Circular

Rockfall Causes

Rockfall occurs in areas that are subject to various types of external and internal forces. In most instances, more than one factor contributes to rockfall. These factors can be grouped into three categories: structural, environmental, and anthropogenic. Typical causes of rockfall and factors influencing rockfall behavior are listed below (07, 14, 26, 37, 38, & 43):

- Structural
    - Adverse planar features
    - Fractured rock
- Environmental
    - Rain
    - Freeze-thaw
    - Wind
    - Snowmelt
    - Channeled melt/runoff
    - Burrowing animals
    - Differential erosion
    - Tree roots
    - Springs or seeps
    - Wild animals
    - Soil decomposition
    - Earthquakes
- Anthropogenic
    - Poor blasting practices
    - Vibrations from trains and construction equipment
    - Poor slope design

In California, the greatest amount of rockfall occurs during rainstorms when supporting materials are washed away from beneath larger rocks. Loss of supporting material can also be caused by wind, snowmelt, and channeled runoff. The second most common cause of rockfall is freeze-thaw cycles where continued expansion and contraction of water in fractures destabilize blocks of rock (26).

Other rockfall causes may exist that are not a direct result of natural processes but are rather a product of human activity. Examples are the remnant effects of poor blasting practices used in excavating rock slopes (common prior to about 1980), vibrations from trains or heavy equipment and stress relief following excavation (07, 24, 25, 26, 37, 38, & 43).

Factors Influencing Rockfall Behavior

Numerous factors influence the behavior of rocks after they have begun traveling from their points of origin, including:

- Slope height
- Slope angle
- Slope roughness
- Vegetation
- Slope geology
- Natural or manmade topography
- Rock soundness
- Rock size
- Rock angularity
- Rock elasticity

These factors influence trajectory, which includes travel speed, mode of travel, distance traveled, and resistance to disintegration, and have direct bearing on the design of mitigation measures (26).

4. ROCKFALL HAZARD RATING SYSTEMS

Slope rating systems are typically adopted to provide a relative ranking of potentially unstable slopes. The main goal is to obtain consistent and comparable information for a series of locations to plan for safety improvements, optimize the benefits from investing limited safety budgets, and reduce maintenance and other operational costs. As a planning tool, slope rating systems facilitate a proactive approach, but they can also be used in response to a rockfall event on a single slope to both describe the event and compare it to other rockfall events in a consistent manner. To provide a uniform standard for evaluating slopes, rockfall sites must be analyzed using the procedures in the Rockfall Hazard Rating System (RHRS) (30). Characterization of rockfall and assignment of a RHRS score requires consideration of the following attributes (30, 37, 38, 42 & 43):

- Rock slope geometry
- Vehicular traffic volumes
- Roadway geometry
- Geologic conditions
- Climatic conditions
- Presence of water on the slope
- Rockfall history

Other published methods, many of which are described in Turner and Schuster (2012), might be more applicable on a site-specific basis. While their use is not discouraged, they should be used in conjunction with and not in place of RHRS.

5. ROCKFALL RISK ASSESMENT AND RISK MANAGEMENT

The practical application of formal risk-based approaches is difficult to achieve in real-world situations because of the investment in time and personnel. Turner and Schuster (2012) discuss a variety of approaches in use today. Use the simple risk-based approach as described in the RHRS manual.

6. SITE INVESTIGATION

Characterization of site stability is determined by a field investigation, which includes a surface investigation and may include a subsurface investigation (07, 26, 37, 38, & 43). A site investigation must begin with a literature search, review of historic and current geologic maps, and aerial photo review. Refer to *Geotechnical Investigations* and *Rock Cut Slopes* for additional instruction on investigations.

(For storm damage response the site investigation procedures herein may need to be abbreviated to provide quick response to the District. Use the Rockfall Storm Damage Evaluation Form at the end of this module to collect the critical site information necessary to provide first response recommendations to the client. Always bring this module with you to the field and try to collect as much of the required information presented in this Section 6. A Rockfall Technical Team member must be consulted in all storm damage situations so that proper recommendations are made given the abbreviated site investigation.)

Rockfall mitigation design uses the structural characteristics and strength properties of the geomaterials. Accurate design of rockfall mitigation relies heavily on surface mapping, geomaterial identification, evaluation of erosion susceptibility, and discontinuity logging. Logging rock structure discontinuities (fracture/joint patterns) and their condition on surface outcrops is essential in the analysis for rockfall mitigation. Determination of groundwater presence, as is true of any slope, is also critical to the assessment of stability.

A rockfall investigation, at a minimum, must answer these six questions:

1. Is there a rockfall problem and where is it located?
2. What is the nature and frequency of past rockfall events at these sites?
3. Where are the source area(s), travel path(s), and runout zone(s) and what are the geological properties of each?
4. What is the likely motion of the rock traveling down the slope (rolling, bouncing, or sliding)?
5. What size rocks typically reach the base of the slope (or point of interest) versus their size in the source zone?

6. How far do the rocks roll past the base of the slope (or point of interest)?

Existing Slopes and New Slopes

Most rockfall investigations are performed on existing slopes where rockfall has been identified. However, in some cases proposed steepened cuts to existing stable slopes may give reason to perform a rockfall assessment. This assessment should be based on several features and characteristics including RQD, fractures in core, proposed catchment areas (or lack thereof), and the presence of river deposits further up the slope that may be or become unstable.

Literature Search

Knowledge of the geology of the area is critical and should be obtained by examining published geologic maps and reports and aerial photographs. District Surveys typically have aerial vertical and oblique photos. Air flights can be arranged with the Highway Patrol to fly sites and take photographs. For existing cut slopes, obtain the “As-Builts” from the District Office and the file of record and/or Document Retrieval Services (DRS). See the *Geotechnical Investigations*.

Climate

Most rockfall occurrences are typically associated with heavy rainfall, rapid snow melt, freeze thaw cycles, wind, channeled runoff, and groundwater (07, 26, 38, 42 & 43). Therefore, historic precipitation and temperature data must be obtained and reviewed. Graphic representations of rockfall history versus temperature, precipitation, and wind at or near the site correlate rockfall frequency to these factors and define trends (07, 41, 42, & 43). The *National Oceanic and Atmospheric Administration* records and maintains weather data around the State and is available on-line.

Maintenance History

The history of a slope or section of road can identify trends in rock slope failure, and most importantly, the rockfall occurrence and frequency. Review maintenance and/or legal records for rockfall accident history, which provides insight into potential rockfall risk. Speak with the District’s Maintenance Engineer. Maintenance personnel can provide estimates of the rockfall size, shape, and frequency which can be used to determine the source area and the possible failure mechanism. Evidence of pavement impacts provides information on free falling distribution as well as impact energies. Caltrans Maintenance time reporting system has a category for scaling and rock/mud removal. Accessing this data can be yet another valuable resource to determine rockfall frequency and seasonal trends. Finally, direct communication with the maintenance supervisor and/or maintenance lead workers who have worked in the area may provide the best insight into the history, nature, and impact of rockfall problems at the site.

Rockfall Characteristics

There are four travel modes of a falling rock: free falling, bouncing, rolling, and sliding. In any one or combination of these modes rockfall occurs as either individual rocks or as a group of rocks. An individual falling rock, or a single rock that falls and hits other rocks, causing more rocks to fall, is considered an individual rockfall event. Such an event could also result in a group of individual rocks falling within seconds of each other, or one large group falling simultaneously. A group of rocks might also fall when an unstable rock cluster fails, resulting in a group of rocks falling together as one mass.

Characterizing the rockfall event is important in developing a strategy for determining the trajectory and kinetic energy at the base of the slope. For example, individual rocks might have low mass and high velocity when compared to a larger, more massive group of falling rocks, which might have a lower velocity but a higher mass (07, 26, & 38).

Size and Shape of the Rockfall

The following information must be collected in and around the base of the slope, at Maintenance's nearby rockfall disposal sites and/or below the highway:

- Average rock size
- Maximum rock size
- Typical rock shape (tabular, spherical, disc shaped)
- Dimensions of the rock along three principle rock axes (x, y, and z)
- Specific gravity

This information will be used to estimate the weight of the rock and the moment of inertia of the falling rock body. If carefully measured, this procedure is typically within 10% of the actual weight of the rock (36). Alternatively, weigh the fallen rocks at the base of the slope directly using a load cell, available from the Foundation Testing Branch, attached to the bucket of a loader.

Rockfall Source and Run Out/Impact Zone

Determine the location from where rocks fall (the source area). Determine the location where rocks first impact the ground at the base of the slope (impact zone), and where the rocks come to rest (run out zone). Knowing the beginning and final locations of the rockfall helps identify the path the rock travels. This information is important in determining the rockfall bounce height and cross-section used in the analysis. Also, from careful inspection of the rockfall path, determine information on the trajectory of the rockfall by identifying impact locations on the slope and at the base of the slope, or observing where rocks have hit trees or other obstructions.

Measure the size of rocks in the run out/impact zone and at the source to understand about how the rocks break up during the rockfall. If large rocks are measured at the source but only fragments reach the run-out zone, the rocks are breaking up either as they fall on the slope or when they hit the road. If the former is true, then a smaller rock size would be used in the analysis, which will affect the analysis and mitigation measures (07 & 38).

Field Mapping

Assess the performance of the existing slopes (both cut slopes and natural slopes) to determine the limits of the unstable area. Field observations of naturally formed cuts in drainages or along the ocean and cuts outside the State Right of Way (ROW) are particularly important in providing the characteristics of a slope and rockfall (07, 26, 37 & 38). Refer to the *Field Mapping* section of the *Rock Cut Slope* module for a list of data and information to collect.

The mapped area should include the active rockfall zone and any adjacent area that may influence or be influenced by the rockfall. Geologic maps on a wide variety of scales are available from the USGS, CGS, local, county and government entities, and private organizations.

Collect or note the following information:

- Slope length (measured along the slope angle and vertical height): physically measure with tape, range finder, or pace.
- Slope angle: physically measure with Clinometer, Brunton or Clar Compass.
- Slope surface: smooth, planar, undulatory, and blocky; if undulatory or blocky, note the difference in height from the ‘peaks’ to the ‘valleys’.
- Describe lithology of the soil and/or rock.
- Estimate or physically measure native slope ratios above and below cuts, as well as on nearby slopes of similar geology. Note presence of boulders, including average size, maximum size, shape, and rock type, as well as their location (top of slope, on slope (embedded or loose), at base of slope).
- Local stability - Note erosion features, rockfalls, slumps, rills, washes, etc.
- Global stability - Note presence of landslides and rockslides.
- Moisture features, such as seeps, springs, wet areas, and their likely causes.
- Assess condition of existing slopes.
- Describe vegetation.

Where applicable, rock structure data should be collected and recorded. In addition to basic field observations, a scan line analysis should be performed on open cut slopes or natural slopes. This information is used in estimating the *Rock Mass Quality* (Q) and the *Geological Strength Index* (GSI). A table should be developed to include the following rock discontinuity information. “*Rock Slopes*” (40) presents a thorough description of the following geological terms in Chapter 2, Section 2.3.

- Location
- Station
- Discontinuity orientation (Dip/Dip Direction)
- Type of Discontinuity (bedding, jointing, fractures, faulting, etc.)
- Persistence
- Spacing
- Filling
- Asperities
- Roughness
- Rock Quality Designation (RQD)
- Discontinuity Sets
- Discontinuity Roughness
- Discontinuity Alteration
- Discontinuity Presence of water
- Weakness Zones intersecting the excavation

Existing Roadway Conditions:

- Number of lanes
- Lane width
- Paved shoulder width and unpaved shoulder width, and slope ratio of both.
- Catchment Area (width of roadway between the base of the slope and the edge of traveled way available for rock and debris containment) and back slope ratio.
- Observations of pavement condition, presence and location of cracking, settlement, patching.

Existing man-made structures:

- Walls, foundations, building types, culverts, undercrossing, utilities, etc.
- Note general condition of structures.
- Note if roadway work might impact the structure.

Field Drawings:

- Develop a scaled field drawing of the area of concern
- Develop one or more typical cross sections of the area of concern
- Develop cross section with initial mitigation approach if appropriate

Photographs of the area of concern:

- Photos of the problem area (direct/overall)
- Photos of main features or any other important features inside or outside of the area

Identify additional geologic investigations that may need to be performed to adequately characterize the site, such as additional boring locations and/or geophysical studies.

The Geoprofessional must also collect data required for the *Rockfall Hazard Rating System* (RHRS) (30 & 33) and the *Colorado Rockfall Simulation Program* (CRSP) (02 & 23). The field data necessary for RHRS and CRSP are also useful in addressing other aspects of a rockfall investigation.

RHRS requires:

- Slope height
- Ditch effectiveness
- Vehicle risk
- Sight distance
- Roadway width
- Geologic character
- Block size
- Climate
- Rockfall history

CRSP requires:

- Slope geometry
- Slope inclination
- Slope length
- Surface roughness
- Lateral variability
- Slope material properties
- Slope coefficients
- Falling rock coefficients
- Falling rock geometry
- Falling rock size
- Falling rock shape
- Falling rock material properties
- Falling rock durability
- Falling rock mass

When possible, investigate rockfall source zone instabilities with analytic tools and engineering-mechanics-based approaches (05). Several easily quantifiable rock mass properties are typically used to predict rock mass behavior. The required analysis, done largely during the field mapping, focuses on identifying likely source zones, the rock mass characteristics of these zones, and the relative potential for each zone to generate rockfall. Use this approach to identify areas that need attention and to assess the effectiveness of potential mitigation measures (38). The recommended procedures are the *Rock Quality Designation* (15 & 38), the *Rock Mass Rating* (05) and the *Modified Q-Rating System* (04).

7. INSTRUMENTATION AND MONITORING

Monitoring observes changes to surface exposures, or measures movements or strains within the rock mass. A typical monitoring program seeks to:

- Maintain safe operation of the transportation facility and to protect the traveling public;
- Provide advance notice of any instability, thus allowing for the early introduction of appropriate mitigation strategies;
- Provide for the protection of personnel and equipment during construction operations in the vicinity of, and for the modification of, any excavation activities to minimize the impact of the instability;
- Provide geotechnical information in order to analyze slope failure mechanisms, design appropriate remedial measures, or conduct a re-design of the slope.

Rockfall monitoring work is performed by the Geoprofessional with the assistance of District personnel. Another option is a service contract, working through Maintenance or Construction to obtain monitoring services.

Monitoring Slope Movement

The techniques available to measure rock deformation belong to two general categories:

- Surface measurements: techniques range from simple visual observations, rock patrols and photographic recording to more sophisticated approaches that include optical and electronic surveying, use of differential Global Positioning System (GPS), terrestrial photogrammetric approaches, and the newest technologies—laser scanning (LiDAR) and radar scanning using differential interferometry.
- Ground displacement measurements: techniques include the application of manual, mechanical and electronic instruments, such as extensometers, strain gauges, and crack gauges or simply measure the distance between two points with a tape measure (07 & 38).

Monitoring Rockfall Characteristics

Rockfall factors to monitor are (1) Location, (2) Frequency, (3) Quantity, (4) Size, (5) Time of the year, and (6) Weather.

Monitoring is typically accomplished utilizing Maintenance forces and their routine patrolling of the roadways. Over the years various forms have been developed for Maintenance personnel to record rockfall activity at a site or sites. Sample forms are available from the Rockfall Technical Team. The most successful forms have been the simplest. This information is invaluable and continued collection and record keeping should be encouraged.

8. ROCKFALL MECHANICS

Kinetic Energy

Kinetic energy is the most common parameter for describing rockfall in engineering design. Rockfall motion is the sum of translational and rotational kinetic energy and is expressed mathematically as:

$$Total\ KE = 1/2\, mv^2 + 1/2\, I\omega^2$$

Where m is the mass of the rock, v is the translational velocity, I is the moment of inertia and ω is angular velocity.

Potential Energy

The potential energy of a rock is:

$$PE = mgh$$

Where m equals mass, m equals the acceleration of gravity and h equal height above the impact zone.

When a rock experiences free fall, its initial potential energy is progressively converted into kinetic energy. The maximum potential vertical velocity of a rock free-falling from a source is:

$$v = \sqrt{2gh}$$

Where v equals velocity just before impact, m equals gravitational acceleration and h is the maximum vertical height of the fall.

Refer to Turner and Schuster (2012) for a complete discussion on rockfall mechanics.

9. ROCKFALL ANALYSIS

Rockfall analysis helps obtain an understanding of the rockfall behavior, which is used to assess potential mitigation measures. Every slope is unique and has variable slope angle, rock type, slope height, topography, soil cover, and vegetation. These features must be characterized because they affect rockfall energy and trajectory.

The various methods used to analyze rockfall energies and trajectories include the quantification of rockfall velocity, impact energy, and bounce heights. The three principle approaches to analysis shown here are discussed in Turner and Schuster, 2013 (38).

- Mathematical Analysis (Table 1)
    - Rock mass stability analysis
    - Rockfall trajectory analysis
        - Rockfall simulation analysis (computer analysis)
        - Empirical analysis
        - Field testing

All these approaches are based on, to differing degrees, actual field rock rolling data, and are used as tools to assist in the design and the assessment of mitigation measures (07 & 38). Where possible, all three methods should be used to analyze the rockfall trajectories at the site. Use of the large body of empirical rockfall data is strongly encouraged.

Rock Mass Stability Analysis

The analysis of a source of rockfall or the stability of an entire rock slope has two goals: (*a*) identification of kinematically unstable blocks and (*b*) investigation of the factors influencing the stability of such blocks. Stereographic, kinematic, and limit equilibrium analytic methods are discussed in *Rock Cut Slope*s and summarized in Table 1.

Table 1: Conventional Methods of Mathematical Rock Slope Analysis (38)

Analysis Method	Critical Input Parameters	Advantages	Limitations
Stereographic and Kinematic	Critical slope and discontinuity geometry; representative shear strength characteristics.	Simple to use and show failure potential. Some methods allow analysis of critical key-blocks. Can be used with statistical techniques to indicate probability of failure and associated volumes.	Suitable for preliminary design or for non-critical slopes, using mainly joint orientations. Identification of critical joints requires engineering judgment. Must be used with representative joint/discontinuity strength data.
Limit Equilibrium	Representative geometry, material/joint shear strength, material unit weights, groundwater and external loading/support conditions.	Much software available for different failure modes (planar, circular, wedge, toppling, etc.). Mostly deterministic but some probabilistic analyses in 2-D and 3-D with multiple materials, reinforcement and groundwater profiles. Suitable for sensitivity analysis of Factor-of-Safety to most inputs.	Factor-of-Safety calculations must assume instability mechanisms and associated determinacy requirements. In situ stress, strains, and intact material failure not considered. Simple probabilistic analyses may not allow for sample/data covariance.
Rockfall Simulation	Representative slope geometry and surface condition. Rock block sizes, shapes, unit weights, and coefficients of restitution.	Practical tool for siting structures and catch fences. Can utilize probabilistic analysis. 2-D and 3-D codes available.	Limited experience in use relative to empirical design charts.

(From “Rockfall: Characterization and Control ©”, National Academy of Sciences. Materials used with permission of the Transportation Research Board.")

Rockfall Trajectory Analyses

When possible, use the following three methods to analyze rockfall trajectories. The first is to rely on empirical data that characterize the behavior of rockfall for different slope characteristics. The second is to perform field tests where rocks are rolled and the behavior of the falling rocks is observed. The third is to construct a rockfall simulation using the analytical models developed for that purpose.

Empirical Analysis

Using data from field testing, researchers developed tables and charts to describe rockfall behavior given slope parameters such as slope angle and height. For rockfall impact and runout analysis use the following three methods.

a. Rockfall Shadow Zone

For planning studies, use the *Rockfall Shadow Zone* (22) to determine the maximum distance rocks can travel from the base of the slope. A minimum shadow angle of 27.5° should be used for the preliminary estimation of the maximum rockfall runout zone (22).

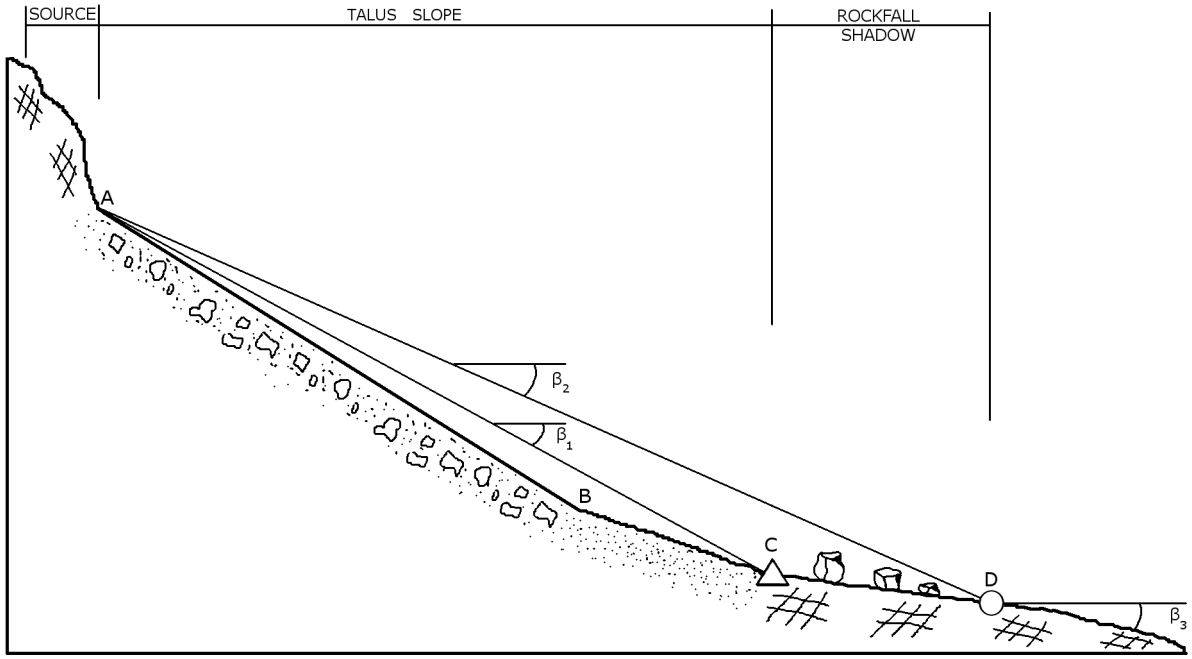


Figure 1: Schematic diagram of the characteristic rockfall path profile. A-C is the talus slope with mean angle β_1 and C-D is the rockfall shadow with a shadow angle β_2, β_3 is the substrate angle (22).

b. Ritchie Criteria

Use the *Ritchie Criteria* (33) for slopes from 80 to 130 feet vertical, and slope angles near vertical to 1¼: 1 to determine rockfall catchment ditch dimension.

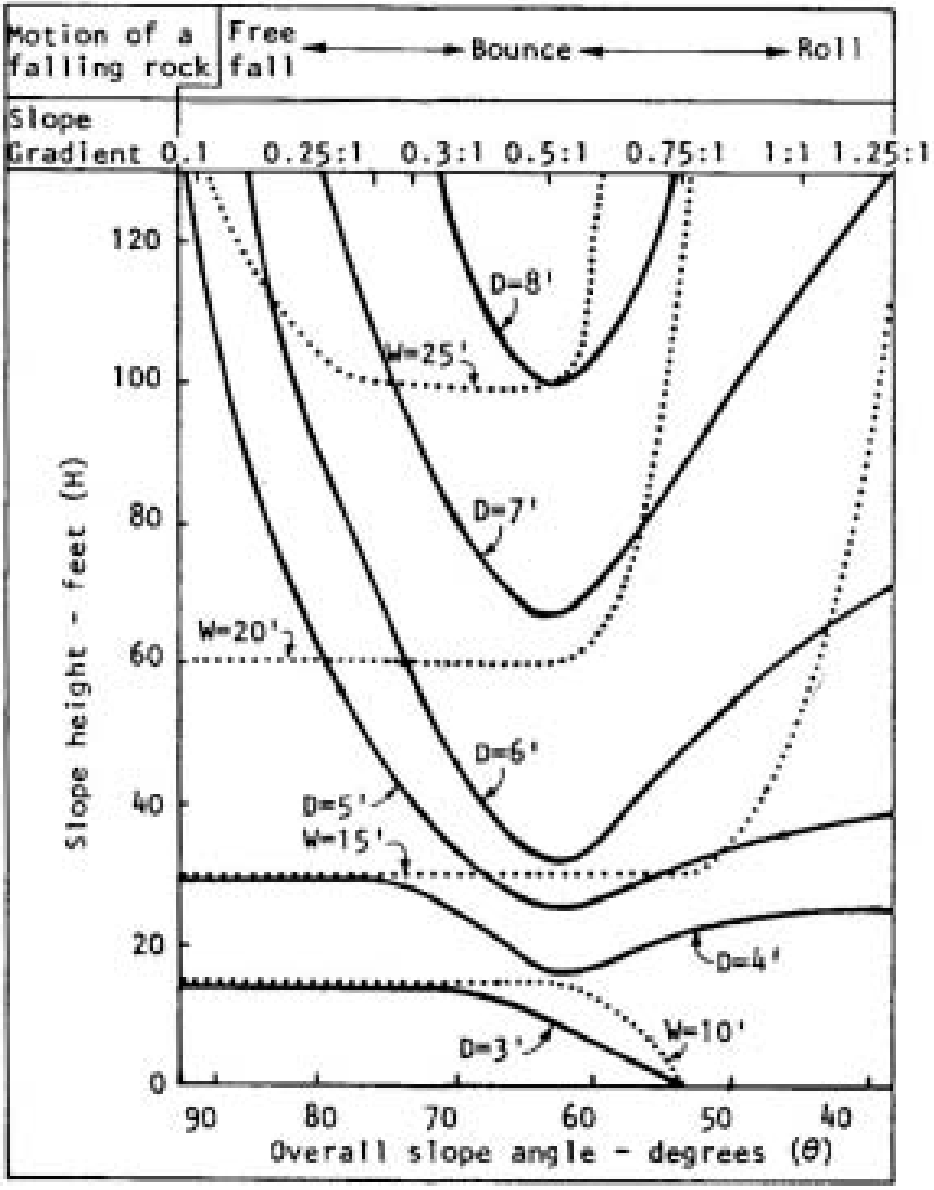


Figure 2: Ritchie's rockfall catch ditch design chart (32)

c. Rockfall Catchment Area Design Guidelines

Use the *Rockfall Catchment Area Design Guidelines* (30 & 32) for slopes less than or equal to 80 feet vertical and slope angles from vertical to 1V:1H to determine rockfall impact and run out zones. From these charts, a designer selects the most appropriate ditch dimension and inclination required to provide adequate catchment.

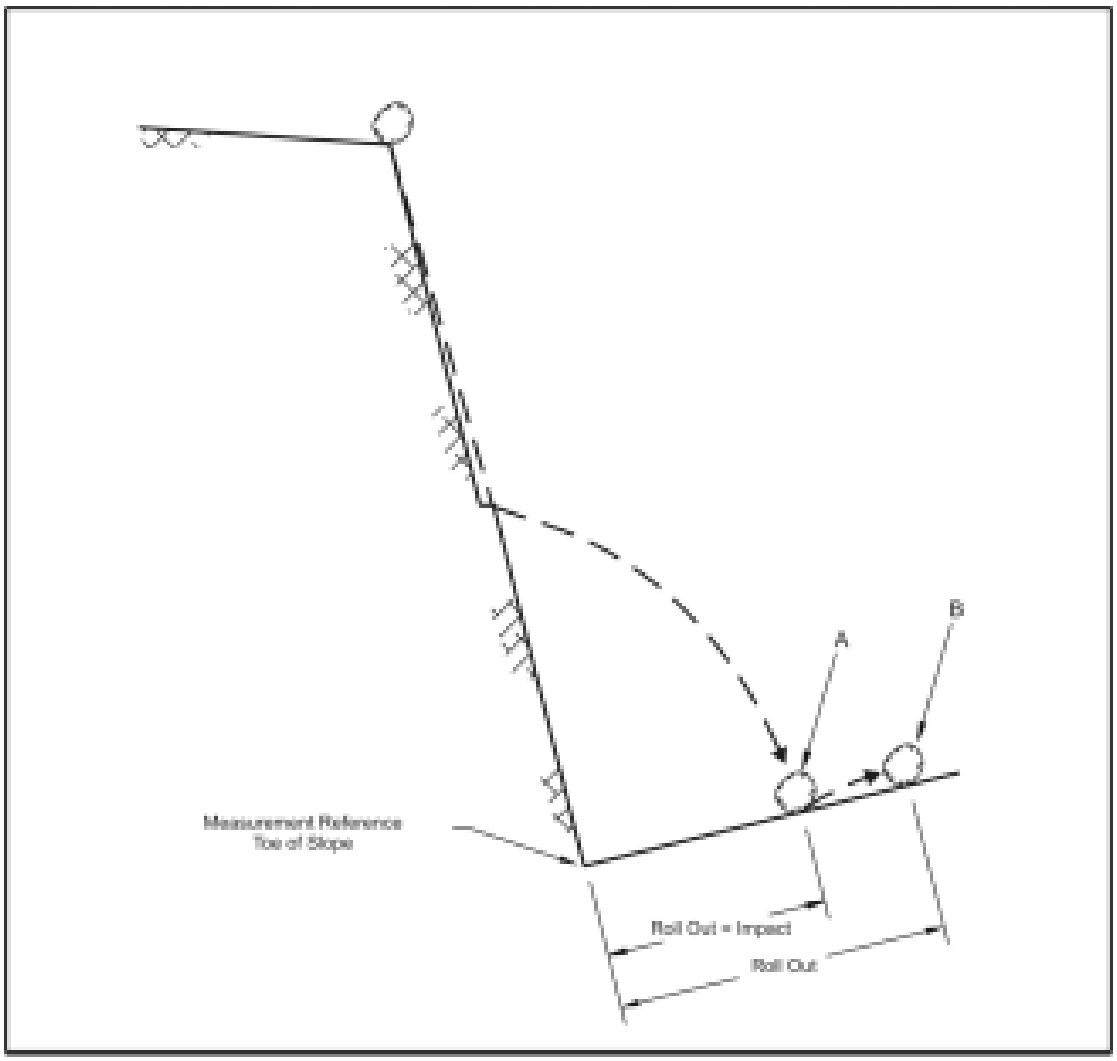


Figure 3: Definition of impact location and roll out distance (22).

Although there is no widely accepted standard for the retention rate, many current designs achieve at least an 85% to 90% retention rate (38). On the other hand, project limitations such as right of way or excavation quantities may prevent designs with such high retention rates. The Department may have to accept an improvement in retention in lieu of ideal retention.

Field Testing

An attempt should be made to roll, film, and analyze actual rockfall as part of the analysis and mitigation design. The preferred method is to perform rock rolling tests at the site, as this will provide the most accurate information for rockfall analysis. However, this is not always possible due to safety, economic, and environmental limitations. Field testing works best if the test area can be prepared without impacting the travelled way. An alternative is to conduct the rock-rolling tests on a similar slope of similar rock type and characteristics away from the road or on a road with less traffic (07 & 38). The rocks being rolled from the source zone (i.e. scaled) or similarly sized rocks on the top of the slope can be used. When planning a test, there are three principle requirements to consider:

- Accessible test rocks
- Visible and accessible test slope
- Data collection equipment and placement

Caltrans developed a Rockfall Barrier Testing Guideline in 1999 (16) and it should be used to develop the testing program. Caltrans has also developed a new technique to film and measure rolling rocks (34). Caltrans Office of Audiovisual Communications will provide video support for rock rolling tests. Turner and Schuster (2012) and Brawner (1994) present thorough discussions on performing field tests. Included in these references are some useful rock rolling data collection forms.

Rockfall Simulation

Computer modeling is used as a tool to study the behavior of rockfall, determine the need for mitigation, and aid in the design (07). Modeling should be used in conjunction with field data and good judgment. Computer modeling allows designers and investigators to observe dozens or even hundreds of simulated rockfall events.

Mathematical modeling analysis must be performed using the *Colorado Rockfall Simulation Program* (CRSP) (02 & 23). Other published models, many of which are described in Turner and Schuster (2012), may prove to be applicable on a site-specific basis. While their use is not discouraged they should only be used in conjunction with CRSP.

Rockfall simulation relies on field observations and assumptions on the starting and ending points of a rockfall, rock size, and actual slope data (listed in 6.8, "*Field Mapping*"). The model is run and various input coefficients are calibrated until the model reflects actual field observations. Another approach to determining the values for input coefficients is to perform field rock rolling tests and then model the observed rockfall trajectories to calibrate the various input coefficients. In either case, the calibrated model is then used as a predictive tool for modeling different slope configurations and rockfall sizes.

10. MITIGATION SELECTION

The Geoprofessional must consider four potential mitigation strategies. Each strategy is weighed against the purpose of the project and potential costs. The strategies are presented below in order of cost and effectiveness (Tables 2 to 6):

1. ***Avoidance*** measures rely on relocation, or realignment of a road away from the rockfall source. Avoidance is always the best solution but is typically by far the most expensive.
2. ***Stabilization*** measures include changes to the slope, or engineered features, to reduce the likelihood of occurrence of a rockfall. Stabilization generally has more moderate costs, but, requires maintenance and has associated risk.
3. ***Protection*** techniques are used to control rockfall once the rocks destabilize. Protection is less expensive, but has higher maintenance costs and increased risks.
4. ***Management*** includes warning signs, monitoring, and rock patrols. Management has low costs, but the risk may be appreciably greater.

Each strategy must be considered for each project and in many cases combinations of strategies are used as the final solution. Consideration should be given to the level of improvement for each strategy. For example, 100 percent mitigation may not be feasible, but improvement of 50 percent may significantly increase safety. All viable strategies should be presented to the client to help in the decision-making process.

Mitigation measures are commonly evaluated based on (38):

- Complexity
- Effectiveness
- Durability
- Constructability
- Environmental
- Aesthetics (context-sensitive solutions)
- Cost
- Ongoing maintenance requirements

The Rockfall Technical Team maintains an ongoing inventory table for each District's rockfall mitigation measures (12). The inventory is organized by location and year constructed and should be used, along with your Rockfall Technical Team representative, to find similar projects.

Stabilization, protection, and avoidance measures make up most rockfall mitigation work. Table 2 provides a qualitative comparison of each of the mitigation measures. The comparisons are based on routinely considered criteria including complexity, effectiveness, durability, constructability, traffic impacts, environmental and aesthetic effects, cost, and maintenance requirements.

Table 2: Qualitative Comparison of Mitigation Measures (38).

MITIGATION MEASURE	SELECTION CRITERIA								
	Complexity	Effectiveness	Durability	Constructability / Special Expertise	Road Closure/ Traffic Restrictions	Environmental Limitations	Aesthetic Impacts	Cost	Maintenance Requirements
	Complexity	Effectiveness	Durability	Constructability/ Special Expertise	Road Closure/ Traffic Restrictions	Environmental Limitations	Aesthetic Impacts	Cost	Maintenance Requirements
AVOIDANCE									
Tunnels	VH	VH	H	H	P	L-M	L-H	VH	M-H
Realignment	M-VH	M-VH	H	M	P	M-H	L-H	H-VH	M-H
Elevated Structures	M-VH	M-H	L-H	M	P	L-H	L-M	VH	M-H
STABILIZATION									
Removal:									
Scaling	L-M	L-H	L-M	M	Y	L	L	L-M	L-M
Blast Scaling	M-H	L-H	M	M-H	Y	L-M	L	L	L-M
Trim Blasting	M-H	M-H	M-H	M-H	Y	L-H	L-H	M	L-M
Re-sloping	L-H	M-H	H	L-M	Y	L-H	L-H	M-H	L
Reinforcement:									
Dowels	M	M-H	H	H	P	L	L	M-H	L
Shear Pins	M	M	M	H	P	M	M	M	L
Rock Bolts	M-H	M-H	H	H	P	L	L	M-H	L
Shotcrete	M-H	M-H	M-H	H	P	M-H	H	M-H	L
Buttresses	M-H	H	H	M	P	L-H	H	M-H	L
Cable Lashing	M-H	L-M	L-M	M	P	L-H	M	M	L-M
Whalers/Lagging	M	M	L-M	M	P	L-H	M-H	M	L
Drainage:									
Weep Drains	L	L-H	M	L	P	L-H	L	L	H
PROTECTION									
Mesh /Cable Nets:									
Slope Protection	L-M	M-H	M-H	M	Y	M-H	M-H	L	L-M
Anchored Mesh	M	M	M	M	P	M	H	L-H	M-H
Suspended Systems	L-M	M-H	M-H	M	Y	M-H	M-H	L-M	L-M
Catchment Areas/Sheds:									
Ditches/Berms	L	M-H	H	L	P	L-M	L-M	L-H	H
Rockfall Sheds	VH	H	M-H	H	P	H	H	H	L-M
Barriers:									
Rigid Barriers	L	M-H	L-M	L	P	L	L	L	M-H
Flexible Barriers	M	M-H	M-H	M	P	L	L	M	M-H
MANAGEMENT									
Warning Signs	L	L-M	na	na	N	L-M	L	L	L-M
Road Patrols	L	L-M	na	na	N	na	na	L-H	H
Scaling	L-M	L-H	L-M	M	Y	L	L	L-M	L-M
Ditch Cleaning	L	L-H	na	na	Y	L-M	na	L-M	H
Monitoring	M-H	L	L-H	M-H	N	L	L	L-H	L-H

L = low, M = medium, H = high, VH = very high, N = no, Y = yes, P = possibly, na = not applicable

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Avoidance

Avoidance entails moving an existing or planned facility away from a rockfall zone. The available alternatives require a shift in either horizontal or vertical position. Realignment of a highway away from the base of an unstable slope or tunneling through the slope requires a horizontal shift of the facility. Viaducts, stacked lanes, and separated grades using various retaining wall options rely on a vertical shift, which elevate facilities to avoid or reduce exposure to rockfall (38).

Avoidance measures, while costly, are a viable solution for Caltrans. Many examples exist throughout the state most notably, the Confusion Hill realignment (01-Men-101-PM 99), Emerald Bay Viaduct (03-), Devils Slide Tunnel (04-SM-001-PM 38.5/40) and Pitkin's Curve Bridge (05-Mon-001-PM 21). Many smaller avoidance projects have been completed in the State, the majority of which utilize viaducts to move the roadway away from the slope and allow rocks to pass under the roadway. Table 3 presents a description, purpose, and limitations of avoidance measures.

Table 3: Avoidance Measures (38)

AVOIDANCE MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS
Tunnels	Avoid unstable slopes by moving the roadway inside the rock mass away from external rockfall sources.	Problems associated with traffic in confined space. Long tunnels require lighting and special ventilation. Expensive.
Realignment	Full road realignment or facility relocation to move away from rockfall area.	Often the old road must be maintained for existing access. Commonly there is limited space for this option. Expensive.
Elevated Structures	Used to span the anticipated rockfall paths allowing rockfalls to pass beneath.	The structure must completely span the active area to avoid being damaged by rockfalls. Expensive.

Stabilization

The goal of stabilization is to reduce the potential for rockfall by preventing or inhibiting material from dislodging at the source area through either a reduction in the driving forces, an increase in the resisting forces, or both (02, 19, 26, 27, 35, 38, 39, 40, 42 & 43). Table 4 presents a description, purpose and limitations of each stabilization measure.

Table 4: Stabilization Measures (38)

STABILIZATION MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS	SPECIFICATION
Removal:			
Scaling	Removal of loose rock from slope by means of hand tools and/or mechanical equipment. Commonly used in conjunction with most other design elements.	A temporary measure that usually needs to be repeated every 2 to 10 years as the slope face continues to degrade.	NSSP available from your RTT representative.
Rock Removal/ Blast Scaling	Removal of loose rock or large rock blocks from slope by means of blasting or chemical expanders.	Damage from fly rock and rockfall, and possible undermining or loss of support by key-block removal.	Rock Excavation (SSP 19-4-X1) Controlled Blasting (SSP 19-4-X2)
Trim Blasting	Used to remove overhanging faces or protruding knobs that may act as launch features on a slope.	Difficulties with drilling, debris containment, and safety.	Rock Excavation (SSP 19-4-X1) Controlled Blasting (SSP 19-4-X2)
Re-sloping	Cutting the rock slope at a flatter angle to improve slope stability and rockfall trajectories.	May have right-of-way or environmental issues.	Earthwork
Reinforcement:			
Dowels	Untensioned steel bars/bolts installed to increase shear resistance and reinforce a block. Increases normal-force friction once block movement occurs.	Passive support requires block movement to develop bolt tension. Slope-access difficulties.	NSSP available from your RTT representative.
Shear Pins	Provides shear support at the leading edge of a dipping rock block or slab using grouted steel bars.	Cast-in-place concrete needed around bars to contact leading edge of block. Access difficulties.	NSSP available from your RTT representative.
Rock Bolts	Tensioned steel bars/bolts used to increase the normal-force friction and shear resistance along potential rock-block failure surfaces. Applied in a pattern or in a specific block.	Less suitable on slopes comprised of small blocks. Difficult to access slope.	NSSP available from your RTT representative.
Shotcrete	Pneumatically applied concrete requiring high velocity and proper application to consolidate. Primarily used to halt the on-going loss of support caused by erosion and spalling. Also helps retain small supporting rock blocks.	Reduces slope drainage. Can be unsightly unless sculpted or tinted. Wire mesh or fiber reinforcement required. Needs a minimum 2-inch (5-cm) thickness to resist freeze/thaw. Quality and durability are very dependent on nozzleman skills.	
Buttresses	Provide support to overhanging rock or lateral support to rock face using either earth materials, cast concrete, or reinforcing steel.	Height limitations. May form a roadside hazard and be unsightly.	
Cable Lashing	Anchored, tensioned cable(s) used to strap a rock block in place. May be used in conjunction with cable nets or wire mesh. Also used as a temporary support during rock-bolt/dowel drilling activities.	Due to slope and/or block geometry, typically movement must occur for full cable resistance to develop.	NSSP available from your RTT representative.
Drainage:			
Weep Drains	Reduces water pressures within a slope using horizontal drains or adits. Commonly used in conjunction with other design elements.	Difficult to quantify the need and verify the improvements achieved.	

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a. Scaling

Scaling is considered both a stabilization and management measure. In some instances scaling removes the unstable section and requires no additional work. However, in most instances slope scaling needs to be performed periodically. There are many variables that dictate how often a slope needs to be scaled. If a slope requires frequent scaling, every 1 to 2 years or less, then other measures should be considered to mitigate rockfall at the site.

Caltrans has a hand scaling program, which includes experienced scaling crews, a code of safe operating practices, in-house training, slope assessment procedures, and scaling operation guidelines (10, 11 &13). Caltrans crews are typically utilized to perform hand scaling in emergencies and as inspectors during contract scaling operations. Several Districts have management programs and annually hand scale slopes as a preventative measure.

When hand scaling operations are considered, a *Slope Scaling Assessment Form* must be completed as part of the investigation and prior to any scaling operations (11).

Mechanical scaling may be recommended when slope stability is unpredictable. In this situation the “no scaling” alternative must be considered. A common method is to drag a heavy object across the slope face with a crane or cable system. This technique removes loose rock without anyone needing to be on the slope. It is both difficult to estimate the volumes that will be generated and to evaluate the condition of the final slope face. Many references recommend hand scaling be done following mechanical scaling (7, 26, and 38). Therefore, upon completion of mechanical scaling operations the geoprofessional should evaluate the need to perform hand scaling or other mitigation methods.

b. Blasting

Controlled blasting may be used to selectively remove individual rocks or rock clusters that cannot be removed by hand scaling (9). The recommended procedure is to place light charges in strategically drilled holes. Alternatives to explosives are the Boulder Buster™, chemical expanders and hydraulic splitters (9). Blasting under cable mesh drapery is the recommended method to blast on a slope face when there are strict requirements to control fly rock and debris.

Caltrans has a blasting program for breaking rocks and trim blasting. The program is supported by an in-house training program, which includes a blasters manual (9). The blasters manual is an excellent resource and reference for contract blasting operations.

c. Resloping

Resloping redesigns the cut slope to increase stability, improve roadway geometrics, and increase rockfall catchment. The *Rock Cut Slope Design* module must be used when resloping rock cuts.

d. Reinforcement

Rock bolts, dowels, and shear pins are effective methods for slope stabilization. Use of these designs requires a complete understanding of the rock structure of the slope. Measuring and evaluating the rock structure is thoroughly discussed in *Rock Cut Slopes.*

e. Shotcrete

When using shotcrete, provide adequate drainage behind the shotcrete by using weep holes. Plain shotcrete is considered unsightly, but today it is common practice to color and sculpt the shotcrete to blend in to the natural background.

f. Buttresses

Two common buttressing approaches are covering the slope or providing support to a rock or overhang. Slopes can be covered with rock slope protection (RSP) to prevent erosion of the slope face. There are some limitations on slope height and slope angle but it is a very effective measure. Another method of covering a slope is to build retaining structures, single or multi-tiered, with backfill up to the top of the slope. More conventional buttresses provide underlying support via structural members to overhanging rocks and outcrops.

g. Cable Lashing

In many instances a large rock or group of rocks on a slope face are key blocks supporting material above. Removal of these blocks could destabilize a larger mass. In these cases, it is often best to stabilize the rock or rocks in place. While this can be done with rock dowels and shear pins, cable lashing can be done with minimal worker exposure on the slope. Anchors are constructed up-slope, from which cables, used as straps, are wrapped across the rocks and secured to the anchors. Covering the rock face with cable nets and attaching the cable straps to the nets is a technique often used to cover more surface area. An example of calculating the force to prevent overturning of a rock block is shown and is used to determine the number of cables straps and anchor capacity.

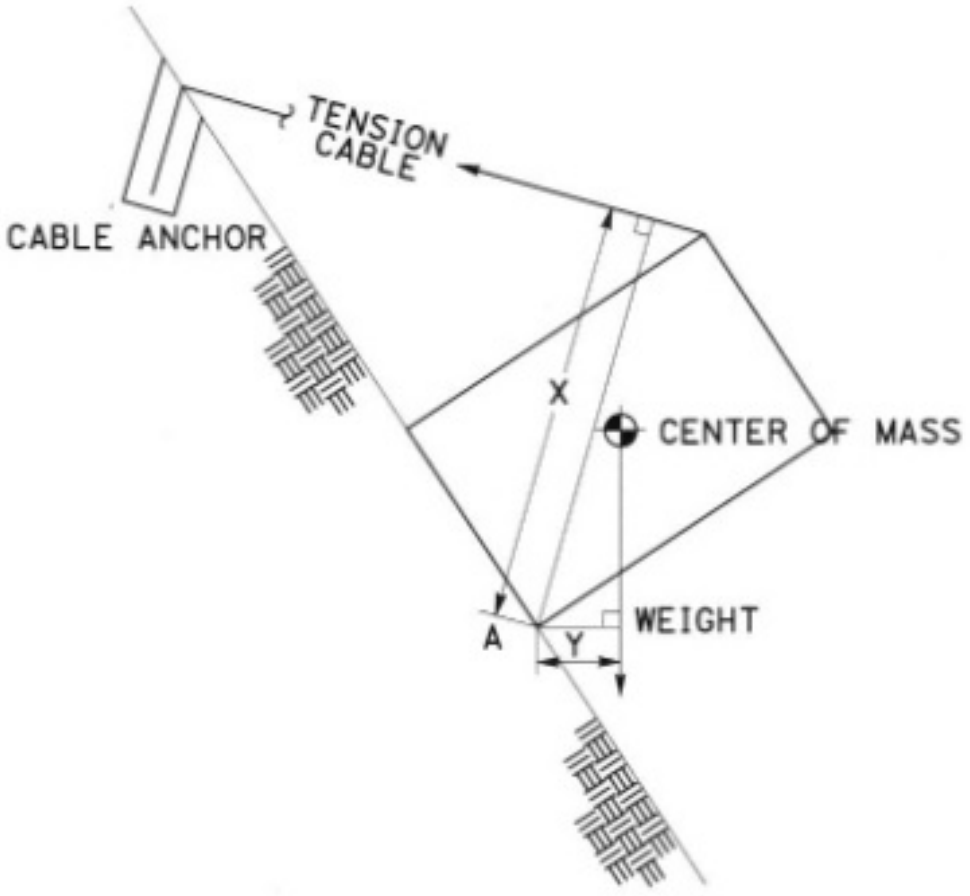


Figure 4: Method to determine required restraining force and anchor capacity. ∑MA=> Wy = Tx, T=Wy/x.

Protection

Protection measures stop, control, or deflect falling rock. Conditions that typically warrant protection measures include (38):

- Rockfall source areas lie beyond Caltrans right of way.
- The extent or nature of the source area is impractical or excessively costly to stabilize. Stabilization is deemed infeasible.
- Avoiding the unstable slope by facility relocation is not practical or is excessively costly. Avoidance is deemed infeasible.

Table 5 presents a description, purpose, and limitation of protection measures.

Table 5: Protection Measures (38)

PROTECTION MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS	SPECIFICATION
Mesh/Cable Nets:			
Draped Mesh Slope Protection	Hexagonal wire mesh, cable nets, or high-tensile-strength steel mesh placed on a slope face to slow erosion, control the descent of falling rocks, and restrict them to the catchment area.	Requires a debris collection ditch area. Must consider debris and snow loads on anchors. Typically limited to 4-ft-minus (1.2-m) rocks.	NSSP available from your RTT representative.
Anchored wire mesh/cable nets/ high tensile strength steel mesh	A free draining, pinned/anchored-in-place net or mesh. Used to retain rocks on a slope.	May form pockets of loose rock as rockfall debris accumulates. Can be difficult to clean out.	NSSP available from your RTT representative.
Suspended Systems (Attenuators)	Wire or cable mesh draped by fence posts or suspended across chutes. The fence (impact zone) intercepts and attenuates falling rocks initiating upslope, and directs them beneath the mesh and into the roadside catchment area.	Require debris-collection ditch areas. Must consider debris and snow loads on anchors. Typically limited to 4-ft-minus (1.2-m) rocks.	NSSP available from your RTT representative.
Catchment Areas/Sheds:			
Ditches/Berms	A shaped catchment area at the base of a slope used to contain rockfall.	Tall slopes require wide fallout areas. May have right-of-way or environmental issues.	
Rockfall Sheds	A covered structure used to intercept and divert rockfalls.	Expensive, hazards associated with traffic in confined space. Must consider downslope issues.	
Barriers:			
Rigid Barriers (with or without fence extension)	Rigid barrier walls used to intercept falling rock and restrict them to a prescribed fallout area. Examples include Jersey barriers, guardrails, and other concrete, gabion, or mechanically stabilized earth (MSE) walls.	Rigid systems are more prone to damage by higher-energy events. Complicates debris cleanout and snow plowing.	
Flexible Barriers	Wire ring, high-strength wire mesh or cable-net panels with high-energy absorption capacity supported by steel posts and anchor ropes with braking elements. Typically proprietary systems.	Require room for barriers to deflect during impacts. Must be cleaned out periodically. Complicates snow plowing.	NSSP available from your RTT representative.

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a. Flexible Rockfall Fences

Caltrans has over 100 flexible rockfall fences throughout the state (12). Flexible rockfall fences are suitable for energies under 1000 kJ (38). Above 1000 kJ, the fences typically require maintenance after a single design load impact. When rockfall impact energies are above 1000 kJ, careful consideration should be given to the suitability of using a fence. Consider other protection measures (earthen berms, timber walls, attenuators, etc.) or avoidance or stabilization measures. In general, fences for high impact energies should be used where the higher impact energies are infrequent.

Standard fence testing guidelines test at two energy levels; service energy level (SEL) and maximum energy level (MEL). SEL represents the energy capacity (rating) of system from two impacts and the fence does not require maintenance. MEL represents the maximum energy capacity (rating) of a fence from a single impact, regardless of the condition of the fence as long as the rock is stopped. Standard testing procedures require the MEL to be 3 times the SEL. When choosing a fence, consideration should be given to frequency of SEL and MEL rockfall impacts as this will directly correspond to maintenance requirements and construction costs. For most typical highway rock cuts, designing for SEL is appropriate.

Caltrans flexible rockfall fence specifications specify post spacing, post dimension, mesh type, and deflection limits. The specification does not require that a fence be tested by a specific test. Instead, proof of performance is required which can be a combination of testing, calculations, and case histories.

Use the *Rockfall Catchment Area Design Guide* (32) and the potential energy formula to design flexible rockfall fences. This manual is applicable for slopes less than or equal to 80 feet vertical, between 45 and 90 degrees, and rockfall dimensions less than 3 feet.

For example, consider a site with a vertical slope height of 60 feet, a slope ratio of ½:1, a 6-foot dirt shoulder, an 8-foot paved shoulder, and a rockfall size of 2 feet in dimension.

The potential energy is:

PE=mgh= weight x height (imperial units)

w = (2’x2’x2’) (165pcf) = 1320 pounds

PE=1320 pounds x 60 feet

PE=39.6 foot- tons (106.9 kJ)

Determine fence height by using the *Rockfall Catchment Area Design Guide* by simple geometry. For example, as determined from catchment charts in the *Rockfall Catchment Area Design Guide* for 99% containment, the catchment width needs to be 11 feet. Therefore, the design fence, to have 11 horizontal feet between the top of the fence and the slope face, is 10 feet. In this example the fence should be a 110-kJ energy system, 10 feet in height.

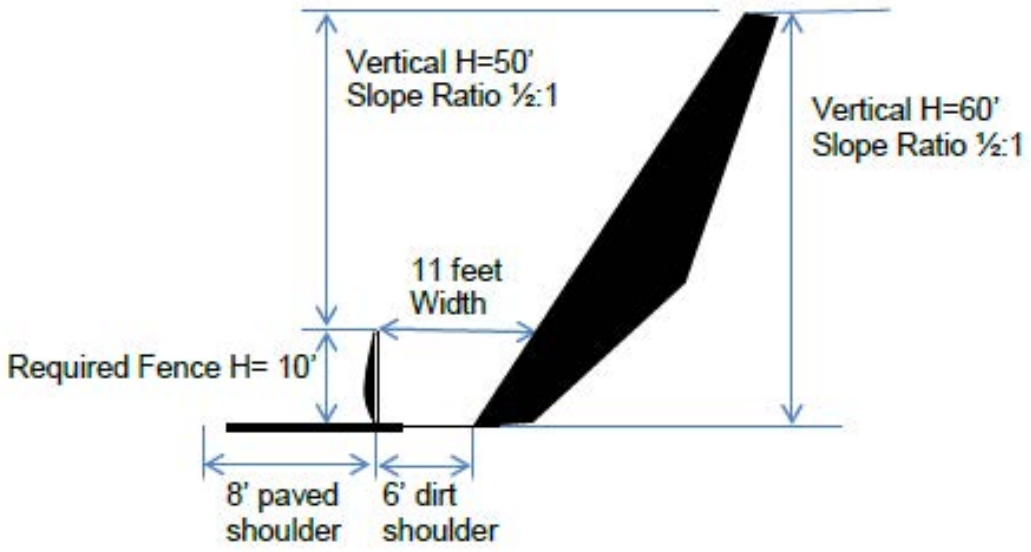


Figure 5: Using the Rockfall Catchment Area Design Guide to determine required fence height as a function of containment.

b. Cable and Wire Mesh Drapery

Covering a slope with wire or cable mesh drapery is widely used throughout the State (12). Drapery systems must be designed in accordance with *Design Guidelines for Wire Mesh/Cable Net Slope Protection* (29). Of note are the appendices where ground anchor spacing and ground anchor load requirements are determined based on slope height, slope angle, and mesh type. Interface friction between the mesh and the ground surface must be used when designing anchor loads and spacing. Snow loading, ground anchor design, and anchor testing are also well documented. Draped meshes need some catchment at the base of the slope to contain material as it migrates downward and accumulates at the base of the slope. Where there is little or no catchment, anchored mesh should be considered (Section 10.3.3).

Mesh selection criteria is based on rock size. Use double twisted wire mesh (38) for slopes with rockfall sizes less than 2 feet maximum dimension. For rockfall larger

than 2 feet, but less than 5 feet, use cable mesh (38). Larger rockfall sizes require a special drapery design and may warrant an alternative strategy.

In some cases, the top of the drapery needs to be elevated to intercept rockfalls sourced upslope of the installation, where natural topography channels rockfall, or along abrupt slope convextities.

For example, consider a slope that is 100 feet high (vertical) with a 1:1 slope ratio and maximum rockfall sizes of 18 to 20 inches. Because the rock size is less than 2 feet, double twisted wire mesh is selected. Anchor load and spacing for the project is determined from Appendix A, Figure A-1 in the *Design Guidelines for Wire Mesh/Cable Net Slope Protection* (29). Anchor spacing, often dependent on topography, supports 40 foot spacing. For a 40-foot anchor spacing and a 100-foot high slope the individual anchor load will be 600 pounds.

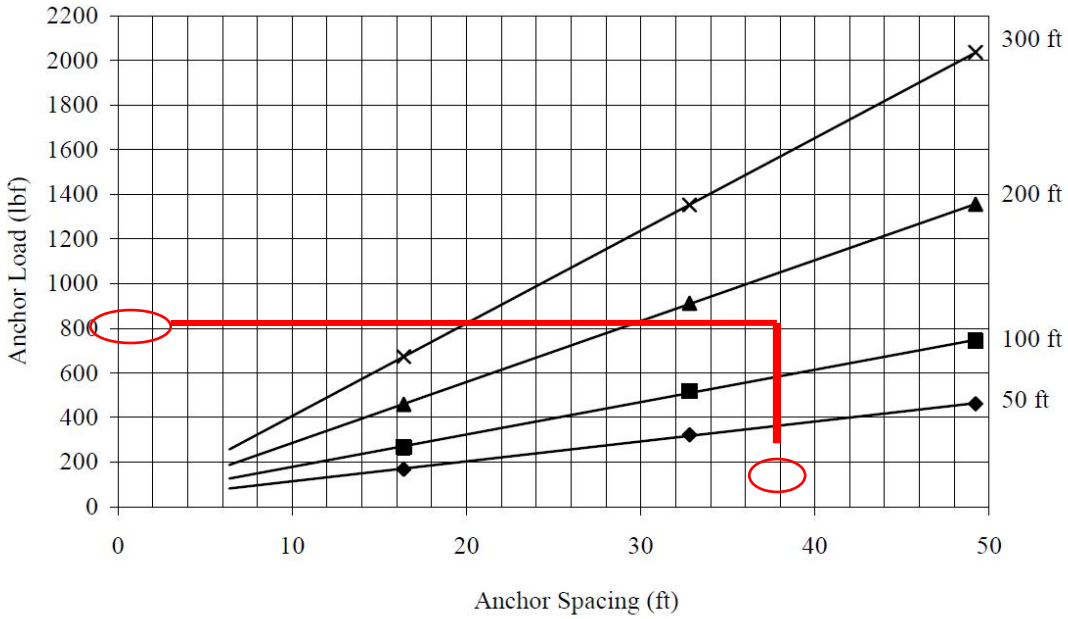


Figure 6: Graph plots anchor load vs. spacing for double-twisted wire mesh for a planar, 45° slope ranging in height from 50 to 300 ft (29).

c. Anchored Cable and Wire Mesh

Anchored meshes for rockfall are different from anchored meshes used for slope stabilization. Use anchored mesh designs when there is little to no catchment at grade. In this case the standard drapery design is employed with the mesh anchored to the slope so the rocks cannot migrate down slope. At present there are over 25 anchored mesh installations in the State (12). Anchored meshes do not increase stability by imparting an active force on the slope face, but instead are passive systems that stop rocks from falling to the base of the slope by containing them behind the mesh (06). Caltrans uses double twisted wire mesh and cable mesh in its

anchored systems. The Geoprofessional should use the punching shear strength of the mesh in the determination of the anchor loads. The design load of the dowels need not exceed the material punching shear.

d. Cable and Wire Mesh Attenuators

Suspended wire and cable mesh barriers are a hybrid of drapery systems and flexible rockfall fences. Also referred to as attenuators, these systems catch rolling and bouncing rocks, attenuate the energy, suppress the trajectory, and guide the rock to the base of the slope into a catchment area. However, in many instances in practice and in testing, the velocity of the rock at the base of the slope is such that the rock can exit into the roadway, sometimes requiring a secondary barrier at grade. Colorado DOT did an extensive study of attenuator systems from which a standard design was developed for energies up to 500 kJ (03). Other studies also support the capabilities of these systems (38). To date all attenuator testing has been performed for energies less than 500 kJ. Caltrans currently has over 15 attenuator systems throughout the state, all of which are performing per design (12, 18). Attenuator development is rapidly progressing as these systems combine the best maintenance attributes from draperies and fence systems, making these systems very popular with maintenance personnel.

e. Catchment Ditches

Properly dimensioned catchment ditches and berms are the most effective and maintainable rockfall mitigation measures available. The orientation of the foreslope has a dramatic influence on rollout distances. Rollout distance drops dramatically as the foreslope angle increases. A vertical foreslope section effectively stops rolling rocks; however, it poses safety hazards to passing vehicles and is not applicable to many roadway geometrys. Catchment design requires space, and as corridor width becomes more restricted, maximizing ditch and berm design for 100 percent containment is rarely achievable. However, increasing catchment effectiveness is still a viable and appropriate alternative. For example, consider a site where there is an existing 6 feet of catchment and, due to limited corridor widths, only 6 additional feet of catchment are available. Standard design criteria require 15 feet for 99% containment. Although the proposed catchment is only 80% of the required width for 99% containment, the catchment width has been doubled and both the impact and runout containment have been improved. This can be quantified by calculating RHRS scores for both the existing and proposed catchment widths. This approach can be used by the design team to balance all of the factors involved in the project and document the improvement in safety achieved by the design. Design of catchment ditches less than 80 feet vertical must be in accordance with the pooled fund study report *Rockfall Catchment Area Design Guide* (32). For slopes above 80 feet vertical the *Ritchie Criteria* must be used for design and supplemented with CRSP (38).

f. Rock Sheds

Rockfall sheds are not often used in California but, due to corridor restrictions and increasing material disposal difficulties, are becoming more viable. Currently there is one rock shed at Rain Rocks (05-MON-1-PM 21) on the Big Sur coast. Another shed is currently being designed at the Ferguson Rockslide (10-MPA-140-PM 42).

g. Rigid Barriers

Rigid barriers, which include timber and concrete lagging walls, earthen berms, Jersey barrier, k-trail, and gabion basket barriers, are being used more infrequently in favor of flexible barriers. Rigid barriers are primarily used where rockfall and mudflows occur at the same location, and there is zero tolerance for the mud passing through. District 7 maintains many rigid barrier systems along the Malibu Coast and has developed a set of plans for timber and concrete barriers which are available through the Department Rockfall Technical Team.

Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS) barriers, although categorized as rigid, deform to absorb energy and are becoming more popular worldwide, especially with the increasing demand for higher energy capabilities. Considerable testing and research has been done to develop barriers with steep slopes that have minimal width requirements and high-energy capacity (38).

Management

Long-term mitigation of rockfall at a single location or along an entire transportation corridor may not be achievable within a reasonable time frame for financial, environmental, or other reasons. In such cases, a variety of maintenance and/or monitoring activities may provide a significant reduction of the risk and improve public safety by reducing the exposure of road users to unstable slopes during critical periods of slope instability. Risk-reduction measures include cautionary signs, observational or instrumentation monitoring, warning systems, regular maintenance patrols, and preemptive closure of the facility. Interim stabilization or protection measures are also valuable tools for reducing rockfall at a specific location or for a short duration. These risk-reduction measures principally improve safety by increasing traveler awareness or by reducing exposure during critical periods of slope instability (38).

Table 6: Management Measures (38)

MANAGEMENT MEASURE	DESCRIPTION/PURPOSE	LIMITATIONS
Warning Signs	Alert users to the potential for rockfall and for fallen rocks to be encountered on the roadway.	Users become accustomed to presence of signs and ignore warnings.
Road Patrols	Roadway inspections during periods of higher rockfall activity to find and remove fallen rocks.	May encompass too many miles of road to manage in a timely manner.
Scaling (see protection measures)	Removal of loose rock from slope by means of hand tools and/or mechanical equipment. Commonly used in conjunction with most other design elements.	A temporary measure that usually needs to be repeated every 2 to 10 years as the slope face continues to degrade.
Ditch Cleaning	Removal of accumulated rockfall debris from ditches to maintain their effectiveness in capturing rocks.	Temporary measure requiring repeated action by maintenance crews and a suitable disposal site.
Monitoring (see 7.2)	Use of instruments to detect incipient rockfalls.	Lead time to events can be short.

(From “Rockfall: Characterization and Control ©”, National Academy of Sciences. Materials used with permission of the Transportation Research Board.")

a. Scaling and Blasting

See Scaling and Blasting (pg. 23).

b. Ditch Cleaning

Obtaining records of ditch cleaning frequency, quantities, and ditch effectiveness helps to understand the rockfall behavior at a site. Maintenance has a reporting code on the LMMS time reporting system for this activity that can be used to collect this data. More frequent ditch cleaning can improve catchment effectiveness and is a relatively inexpensive management mitigation measure.

c. Monitoring

See Rockfall Monitoring Characteristics (pg. 11).

11. ROCKFALL MAINTENANCE AND MANAGEMENT PROGRAMS

A rockfall management program consists of a multifaceted “cradle-to-grave” process that takes into consideration:

- Slope identification
- Hazard and risk evaluation
- Mitigation options and cost estimates
- Project prioritization and project programming
- Final mitigation design
- Construction contract preparation
- Construction of the mitigation project

Consistency throughout this process is important. Each step in the process should contain milestones, decision points, or both. All elements of a rockfall management program are detailed, discussed, and referenced in Turner and Schuster (2012). Such programs can be instituted on a statewide basis, district basis, or on a single project scope.

12. REPORTING

Rockfall investigations and mitigation recommendations are presented in the Geotechnical Design Report. Geological interpretations and models must be adequately documented and must discuss the purpose of the work, field methods used, and techniques used for analyses and interpretation. Names and descriptions of software applications used for data reduction and interpretation must be presented. Maps and cross-sections delineating the nature and extent of the project must be provided. Features in the geological models that may affect a project must be noted and discussed All reports should include an estimation of the rockfall trajectory and each report should include a RHRS rating.

Table 7 presents mitigation measures available for rockfall mitigation and the corresponding information that must be reported in the DPGR or GDR. Mitigation methods should be presented in increasing order of cost and effectiveness.

Table 7: Rockfall Reporting

Recommendation	Information to include in the Geotechnical Report
AVOIDANCE	
Tunnel	Possible portal locations, Slope conditions above the portals, Q, GSI, Fracture spacing and orientation, Lined or unlined
Realignment	How far the roadway needs to be from the slope for adequate catchment area
Structures	Position of structure so it is situated far enough away from the slope to achieve an adequate catchment area or elevate the structure to allow the rocks to freely pass under the structure
STABILIZATION	
Removal	Standard reporting
Scaling	Slope Assessment Form
Blasting	Areas to be removed, Individual rock assessment, Slope Assessment form
Re-sloping	Seismic Velocities, Cut slope angle
Reinforcement	Standard reporting
Rock Bolts, dowels, and shear pins	Spacing, bonded and unbounded length, load, location
Shotcrete	Area to be covered, weep hole location
Buttresses	Location, dimensions
Cable lashing	Sections to be lashed, cable anchor locations, cable and cable anchor loads
Whalers/Lagging	Location, dimensions
PROTECTION	
Mesh/Cable Net	Rockfall sizes, area to be covered, type of mesh, application ie drapery, suspended systems, anchored.
Catchment Areas	Rockfall sizes, energy, bounce heights, ditch dimension, ditch location, percent retention, application ie ditches, berms, rock sheds
Barriers	Rockfall sizes, energy, bounce heights, barrier location, application ie flexible rockfall fence, earthen berm, timber or concrete lagging wall.
MANAGEMENT	Rockfall frequency, seasons, rockfall sizes, ditch effectiveness

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Rockfall Storm Damage Evaluation Form Page 1 of 2

Project Information

District _____ County _____Post Mile _____
EA _____
EFIS ID# _____
Project Manager _____
Project Name _____
Geoprofessional _____
Date Reviewed _____

District Assessment Information
DAF Number _____
Description
_____
Proposed Repair Type _____
Total Cost _____
Priority Category _____ Geotech
_____
Priority Level

Problem Description

_____
_____
_____
_____
_____
_____
_____

Site Severity Rating: Severe Moderate Slight

Photo Attached Yes No

Repair Strategy Options	Risk	Geotech Complexity
Clean up//Minimal Repair	High Medium Low	High Medium Low
Avoidance	High Medium Low	High Medium Low
Stabilization	High Medium Low	High Medium Low
Protection	High Medium Low	High Medium Low
Management	High Medium Low	High Medium Low

Preliminary Preferred Repair

_____
_____
_____
_____
_____

Rockfall Storm Damage Evaluation Form Page 2 of 2

Field Data Collection List

- Current slope condition
- Rockfall hazard None Imminent Long term
- Slope height – vertical
- Slope ratio
- Slope length
- Roadway characteristic
- Roadway width
- Catchment dimensions
- Rock Size Average Maximum
- Rollout distance Average Maximum
- Potential Energy
- Rockfall source
- RHRS Score
- Slope assessment form
- Local knowledge
- Detours and closures situation

Plan View Sketch

Cross Section Sketch

1.0 Ground Modification

Ground modification technologies are geotechnical construction methods used to improve poor ground conditions when removal and replacement, avoidance of such conditions, or the use of deep foundations is infeasible or too costly. Ground modification may be used to:

- Mitigate liquefiable soils.
- Improve loose or soft soil to reduce settlement, increase bearing capacity, shear, or frictional strength as well as overall improvement of stability for embankment and structure foundation.
- Improve slope stability for mitigation of landslides.
- Increase density.
- Decrease imposed load.
- Form seepage cutoff or fill voids.
- Accelerate consolidation.
- Control deformation.
- Provide/increase lateral stability.
- Reduce earth pressures.

There are three strategies available to accomplish the above functions:

1. Increase shear strength, density, and/or decrease compressibility of foundation soil,
2. Reduce the applied load on the foundation soil by the use of lightweight fills,
3. Transfer the load to a more competent (deeper) foundation soil.

Ground Modification Methods, Volumes I and II, FHWA NHI-16-027 and FHWA NHI-16-028, April2017 are frequently referenced in this module. The Geoprofessional should consult each volume for details concerning a specific ground modification method. Also be aware of new and innovative ground modification methods. If a new or innovative ground modification method is to be considered on a Caltrans project, the method should be discussed with the Project Development Team including Construction. Sometimes it will become necessary to initiate a construction evaluation project to measure the effectiveness of a new or innovative ground modification technique prior to its use.

A web-based information and guidance system, *Geotechnical Solutions for Transportation Infrastructure* (*Geotech Tools*), presents information on geoconstruction technologies and provides a tool to assist in deciding which technologies are potentially applicable to site-specific conditions. The following ground modification techniques are addressed in *Geotech Tools*:

- Aggregate columns/Stone Columns/Rammed Aggregate Piers
- Beneficial Reuse of Waste Materials

- Bio-Treatment for Soil Stabilization
- Blast Densification
- Bulk-Infill Grouting
- Chemical Grouting/Injection Systems
- Chemical Stabilization of Subgrades and Bases
- Column Supported Embankments
- Combined Soil Stabilization with Vertical Columns (CSV)
- Compaction Grouting
- Continuous Flight Auger Piles
- Deep Dynamic Compaction
- Deep Mixing Methods
- Drilled/Grouted and Hollow Bar Soil Nailing
- Electro-Osmosis
- Excavation and Replacement
- Fiber Reinforcement of Slopes
- Geocell Confinement in Pavement Systems
- Geosynthetic Reinforced Construction Platforms
- Geosynthetic Reinforced Embankment
- Geosynthetic Reinforcement in Pavement Systems
- Geosynthetic Separation in Pavement Systems
- Geosynthetics in Pavement Drainage
- Geotextile Encased Columns
- High-Energy Impact Rollers
- Helical Soil Nails
- Hydraulic Fill with Geocomposite and Vacuum Consolidation
- Injected Lightweight Foam Fill
- Intelligent Compaction
- Jet Grouting
- Lightweight Fill, EPS Geofoam, Low Density Cementitious Fill
- Mass Mixing Methods
- Mechanical Stabilization of Subgrades and Bases
- Mechanically Stabilized Earth Wall System
- Micropiles
- Onsite Use of Recycled Pavement Materials
- Partial Encapsulation
- Prefabricated Vertical Drains and Fill Preloading

- Rapid Impact Compaction
- Reinforced Soil Slopes
- Sand Compaction Piles
- Soil Nail Wall
- Shoot-In Soil Nailing
- Shored Mechanically Stabilized Earth Wall System
- Traditional Compaction
- Vacuum Preloading with and without PVDs
- Vibro-Concrete Columns
- Vibrocompaction

2.0 Selection Process

Selection of an appropriate ground modification technology requires consideration of technologies and site-specific project goals and challenges. Use the steps in Table 1 to select the appropriate method.

Table 1: Selection Process (after Ground Modification Methods)

Step	Process
1	Identify potential poor ground conditions, including extent and type of negative impact
2	Assess remove and replace and avoidance options – if infeasible or too expensive consider ground modification
3	Identify or establish performance requirements
4	Identify and assess any space or environmental constraints
5	Determine subsurface conditions – Type, depth, and extent of poor soil as well as groundwater table depth and assessment of shear strength and compressibility
6	Make preliminary selection – consider performance criteria, limitations imposed by subsurface conditions, schedule and environmental constraints, and the amount of improvement required (Table 2 should be used in this selection process)
7	Perform preliminary design
8	Compare and select – selection is based on performance, constructability, cost, and any other relevant project factors

Ground modification categories, functions, methods, and applications are summarized in Table 2.

Table 2: Ground Modification Categories, Functions, Methods and Applications (after Ground Modification Methods)

Category	Function	Method	Application
Consolidation	Accelerate consolidation and increase shear strength	1- Prefabricated vertical drains 2- Surcharge	Viable for normally consolidated clays. Can achieve up to 90% consolidation in a few months
Load Reduction	Reduce load on foundation and reduce settlement	1- Geofoam (EPS) 2- Foamed (Cellular) Concrete 3- Lightweight fill	Density varies from 6-76 lb/ft3. Granular fills usage subject to local availability.
Densification	Increase density, bearing capacity, and friction strength of granular soils. Decrease settlement and increase resistance to liquefaction	1- Vibro-Compaction 2- Dynamic Compaction by falling weight impact	Vibrocompaction viable for clean sands with up to 15% fines. Dynamic compaction limited to depth of about 33 feet, but is applicable for a wider range of soils. Both methods can densify granular soils up to 80% Relative Density. Dynamic Compaction generates vibrations for a considerable lateral distance.
Reinforcement	In soft foundation soils, increases shear strength, resistance to liquefaction, and decreases compressibility. Internally reinforces fills and/or cuts.	1- Stone Column 2- Rammed Aggregate Piers 3- MSE retaining walls 4- Soil Nail walls	Soil Nailing may not be applicable in soft clays or loose fills. Stone columns applicable in soft clay profiles to increase global shear strength and reduce settlement.
Chemical Stabilization by Deep Soil Mixing	Physio-chemical alteration of foundation soils to increase their tensile, compressive, and shear strength; to decrease settlement; and/or provide lateral stability and/or confinement	1- Wet mixing methods using primarily cement 2- Dry mixing methods using lime-cement	Applicable to soft to medium stiff clays for excavation support where the groundwater table must be maintained or for foundation support where lateral restraint must be provided or to increase global stability and decrease settlement. Required significant QA/QC program for verification.

Category	Function	Method	Application
Chemical Stabilization by Grouting	To form fill voids, increase density, increase tensile, and compressive strength	1- Permeation Grouting with particulate chemical grouts 2- Compaction Grouting 3- Jet Grouting 4- Bulk filling 3- Injected Lightweight Foam Fill	1) Permeation grouting to increase shear strength or for seepage control. 2) Compaction grouting for densification and 3) Jet grouting to increase tensile and/or compressive strength of the foundations, and 4) Bulk filling of any subsurface voids. 5) Inject lightweight foam to fill voids and lift pavements and slabs w/o adding weight.
Load Transfer	Transfer load to deeper bearing layers	5- Column Supported Embankment (CSE) on flexible geosynthetic mats	Applicable for deep soft soil profile or where a tight schedule must be maintained. A variety of stiff or semi-stiff piles can be used.

Geotech Tools contains a technology selection assistance tool that provides solution options. The following information is available for each method:

- Technology Fact Sheets
- Photographs
- Case Histories
- Design Guidance
- Quality Control/Quality Assurance
- Cost Information
- Specification Guidance
- Bibliography

3.0 Design Parameters for Ground Modification Analyses

Specific geotechnical data that will need to be developed during the investigation depends upon the ground modification technique chosen. Ground Modification Methods should be referred to for specific geotechnical data needed for various types of ground modification techniques.

4.0 Design Requirements

The following documents should be used for design:

- *Ground Modification Methods*, Reference Manual Volume I and II, FHWA NHI-16-027 and FHWA NHI-16-028, April 2017
- *Geotech Tools, Geo-Construction Information and Technology Selection Guidance for Geotechnical, Structural, and Pavement Engineers*, SHRP2, Transportation Research Board
- *Design and Construction of Stone Columns – Vol. I*, Federal Highway Administration, FHWA/RD-83/026, Barksdale, R. D., and Bachus, R. C., 1983.
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- *Prefabricated Vertical Drains – Vol. I: Engineering Guidelines*, Federal Highway Administration, FHWA/RD-86/168, Rixner, J. J., Kraemer, S. R., and Smith, A. D., 1986.

5.0 Ground Modification Methods used by Caltrans

Caltrans has used the following ground modification methods (see Appendix 1):

- Prefabricated Vertical Drains (PVDs) and Fill Preloading
- Lightweight Fills (natural volcanic, cellular concrete, Expanded Polystyrene (EPS), Expanded Shale, Shredded Tires, and Saw Dust)
- Geosynthetic Reinforced Embankments
- MSE Walls and Reinforced Soil Slopes
- Soil Nailing
- Stone Columns/Rammed Aggregate Piers
- Compaction Grouting
- Injected Lightweight Foam Fill
- Permeation Grouting
- Deep Soil Mixing
- Micropiles

Sections 5.1 through 5.3 provide details (Introduction, Investigation, Design Methods, Reporting, and Considerations for Construction) on three ground modification techniques commonly used by Caltrans: Prefabricated Vertical Drains, Lightweight Fill (Expanded Polystyrene and Cellular Concrete) and Stone Columns/Rammed Aggregate Columns.

5.1 Prefabricated Vertical Drains (PVD) and Surcharge

Prefabricated Vertical Drains (PVD) (formally “wick drains”) are band shaped (rectangular cross-section) geocomposite products consisting of a geotextile filter material surrounding a plastic drainage core. PVD are used to accelerate the settlement and shear strength gain of saturated, soft foundation soils by shortening the drainage path length. PVD are commonly coupled with surcharge fills to facilitate accelerated embankment construction with minimal post-construction settlement.

Advantages of PVD with surcharge are:

- Decreased construction time
- Low cost versus other ground modification technologies
- No spoil
- High production rate
- Durable
- Relatively straightforward and simple QC/QA procedures

Projects that have used PVD are:

- ALA-80 SFOBB Oakland Touchdown (OTD) Geofill (EA 04-01205)
- SOL-37 Widening project (EA 04-0T141)
- ALA-880- 5th Avenue Bridge Seismic Bridge Replacement (EA 04-1706U)
- SJ-4 Widening (EA 10-0H04U)
- MEN-101 Willits Bypass (EA 01-26200)
- SJ-12 Bouldin Island (EA 10-0G800)
- SD- 5 (EA 11-0301U)

Installation of PVD requires site preparation, construction of a drainage blanket and/or a working mat, and installation of the PVD. Site preparation includes removal of vegetation and surface debris, and obstacles that would impede installation of the PVD. It may be necessary to construct a working mat to support construction equipment, which can later serve as the drainage blanket. There are many different ways of installing PVD, but most methods employ a steel covering mandrel that protects the PVD material as it is installed.

All methods employ some form of anchoring system to hold the drain in place while the mandrel is withdrawn following insertion to the desired depth. The mandrel is penetrated into the compressible soils using either static or vibratory force.

Design considerations include drain spacing (typically triangular from 3 to 8 feet spacing with 3 to 6 ft common), flow resistance, and installation disturbance. Quality control tests usually relate to the material properties of the drain and the measurement of settlement and dissipation of excess pore water pressures during consolidation.

5.1.1 Investigation

The investigation for PVD is similar to the investigation for embankment stability and settlement (see *Embankment* module). PVD do not require any special considerations during the field investigation.

5.1.2 Design Methods

The Federal Highway Administration (FHWA) has design documents for both of the preferred design procedures for this technology:

- Prefabricated Vertical Drains, FHWA-RD-86-168, 1986
- Ground Modification Methods, Volume 1, FHWA-NHI-16-027, April 2017

Design parameters include the selection of the drain type, drain spacing, drain length, and the amount of preload needed to achieve a specified consolidation within an allotted time.

The design begins with traditional settlement analyses without PVD to determine the total magnitude and time rate of settlement under final project loads. Then the use of PVD is analyzed to reduce the time to reach the final consolidation settlement.

The first step of the design process is to establish project time requirements, anticipated service loads, and the acceptable amounts of post-construction settlement. A subsurface investigation and laboratory soil-testing program are then performed to provide information about the soil stratigraphy and engineering properties of the compressible soil. Based on this information, the amount of total settlement due to primary consolidation and secondary consolidation can be estimated as well as the time for this settlement to occur. If the time to reach 90 to 95 percent of the total project settlement is too long, PVDs should be considered to reduce the time required for consolidation settlement.

PVD spacing should be determined using the Barron-Hansbo relationship that relates the time to achieve a desired average degree of consolidation to drain diameter, drain spacing, and coefficient of consolidation.

An example calculation of staged fill construction is provided in the Washington Department of Transportation (WASHDOT) Geotechnical Design Manual, Appendix 9A.

5.1.3 Reporting

The GDR should include sections that:

- Justify the use of PVD with surcharge fill if required. State predicted settlement and time to achieve both with and without PVD. Discuss foundation bearing capacity failure or slope instability during staged construction and associated staging requirements. (See *Embankment* module)
- Justify the selection of PVD as the preferred treatment strategy, including such considerations as constructability, cost, and overall project specific effectiveness.

- Provide layout and cross sections of ground modification area showing limits, PVD pattern, and depths of PVDs, stability berm height and location (if used), loading rates, and settlement period.

5.1.4 Field Instrumentation and Considerations for Construction Including QA/QC

Considerations typically include:

- Requirements for field splicing and connecting PVD to drainage pipes and/or drainage blankets/working platforms as required;
- Site accessibility issues for heavy equipment including working platform or ground pressure limitations for very soft surficial ground conditions;
- Difficult PVD installation due to presence of obstructions which may require pre-auguring;
- Confirming that PVD are installed to correct depth in field by appropriate field observations (both to ensure not too short or too long);
- Coordination with District Environmental on any site-specific requirements for pore water discharge (if applicable); and
- A comprehensive geotechnical instrumentation program to confirm settlements and/or stability of embankment is achieved (whether by CT personnel or contractor provided) with clearly defined scope and reporting requirements and sufficient redundancy to handle potential for equipment malfunction/damage and adequately cover planned construction staging.

A key consideration of the geotechnical instrumentation program is to layout the required type, location and depth of monitoring taking into account the proposed construction staging with sufficient redundancy of monitoring points. This is particularly needed for contractor supplied, installed, and monitored instrumentation as less control over data quality is exercised. If the project has stability concerns and controlled loading rates, more detailed and comprehensive instrumentation may be required.

5.1.5 Specifications

Refer to *Geotech Tools* to create a project-specific NSSP.

5.2 Lightweight Fills

Lightweight fill materials are used to reduce the magnitude of the applied loads to:

- Eliminate or significantly reduce embankment settlement.
- Reduce active pressure behind retaining walls and abutments.
- Reduce driving force in landslide repair.
- Increase an embankment’s resistance to seismic loads.

Lightweight fills have primarily been used at Caltrans for reducing embankment settlement at bridge approaches and to reduce the driving force of landslides.

In cases where a soft soil deposit is very thick, partial excavation of the native material directly below the embankment (and backfill with lightweight material) will help to balance the total imposed load. The amount of excavation depends on the unit weight of the material to be excavated and the unit weight of the lightweight fill to be used. The lighter the material the less excavation would be required. Sometimes it is not possible to use lightweight fill to completely offset an additional embankment load, however, it can reduce the additional load to a tolerable amount.

Common lightweight fill materials used by Caltrans are:

- Expanded Polystyrene (EPS) or Geofoam
- Cellular Concrete (Foamed Concrete)
- Natural (volcanic) lightweight materials

Expanded shale, wood fiber (saw dust), and shredded tires have been used by Caltrans. Expanded shale is seldom used in Caltrans for embankment construction due to its relatively high cost. Wood fiber (saw dust) is seldom used in Caltrans for embankment construction due to its lack of availability in large quantity. Shredded tires have been used in three Caltrans projects and its use is encouraged by the California Department of Resources Recycling and Recovery in their effort to reduce stockpiles of disposed tires. FHWA issued an Interim Guideline to limiting the maximum layer thickness for shredded tire fills to 10 feet.

Consider the following when selecting a lightweight fill:

- Availability of lightweight fill materials;
- The engineering properties of the lightweight fill material for use in both settlement and slope stability analysis. For example, for granular lightweight fill, the geoprofessional must evaluate the density, the angle of shearing resistance or cohesion of the lightweight fill. Whereas, for EPS and cellular concrete, in addition to the density, compressive strength must be evaluated;
- The durability, water absorption potential, corrosion potential, and other unique characteristics;
- Costs for using lightweight fill versus conventional construction.

Table 3 provides a list of various lightweight materials with the range of densities, and specific gravities:

Table 3: Lightweight Fill Materials

Lightweight Fill Type	Range of Density (pcf)	Range of Specific Gravity
Natural (Volcanic) Material	50 to 75	0.80 to 1.2
Expanded Polystyrene (EPS)	0.8 to 2	0.01 to 0.03
Cellular (Foamed) Concrete	20 to 61	0.3 to 0.8
Wood Fiber (Saw Dust)	34 to 60	0.6 to 1.0
Shredded Tires	37 to 56	0.6 to 0.9
Expanded Shale	37 to 65	0.6 to 1.0
Fly Ash	70 to 90	1.1 to 1.4
Boiler Slag	62 to 109	1.0 to 1.8
Air-Cooled Slag	69 to 94	1.1 to 1.5

For more information regarding design parameters (density, angle of shear resistance, permeability and compressibility), environmental considerations, design and construction of granular lightweight fill such as Wood Fiber, Air- Cooled blast Furnace, Fly Ash, Boiler Slag, Expanded Shale and Shredded Tires refer to August 2006 FHWA NHI-06-019, Tables 2 through 7.

5.2.1 Expanded Polystyrene (EPS)

The most comprehensive design, material, and construction guidelines on the use of Expanded Polystyrene (EPS) for highway construction have been summarized in NCHRP 24-11 for embankments and 24-11(02) for slope stability projects. Additional design information is summarized by Horvath (1995).

5.2.2 Cellular Concrete (Foamed Concrete)

Cellular concrete consists of cement, water, a foaming agent, and optional admixtures. Cellular concrete is self leveling and can be pumped up to 3300 feet, and will begin to harden between 2 to 6 hours after production. Cellular concrete can be pumped at 100 cubic yards per hour. The density of cellular concrete typically ranges from 25 pcf to as high as 65 pcf. Relative to soil the shear strength is much higher. If significant differential settlement is anticipated the designer should be aware that cellular concrete (due to its relatively brittle nature) could crack, losing much of its shear strength. The unit cost of cellular concrete can be high especially for small quantities.

Caltrans has used cellular concrete in several large and small projects to reduce embankment settlement, to reduce landslide driving forces, and to reduce active pressures behind retaining walls. Cellular concrete has also been used as backfill for tunnel, waterlines and sewers, to provide shock absorption in earthquake zones, and to fill voids in silos and abandoned mines.

The advantages of using cellular concrete compared to other types of lightweight materials are:

- Easily placed by pump or gravity for rapid installation
- Broad range of densities and compressive strengths
- Durable and noncorrosive
- High slump and self leveling
- Absorbs shock waves
- High freeze-thaw resistance
- Low water absorption and permeability
- No compaction is required
- Cost is comparable or even less than most granular lightweight materials

The disadvantages of using cellular concrete compared to other types of lightweight materials are:

- The cost of cellular concrete increases with cast density.
- The cost is relatively high for small jobs.
- Requires qualified cellular concrete contractors and their suppliers.

5.2.3 Investigations (EPS and Cellular Concrete)

The field exploration and laboratory testing should include:

- Determining the thickness of soft foundation soil by drilling or by CPT sounding.
- Performing in situ strength testing using Cone Penetration Test (CPT) or Vane Shear Test (VST).
- Determining the groundwater level (monitoring may be required).
- Obtaining undisturbed soil samples for laboratory testing using the modified California sampler, Shelby tubes, and pitcher barrel.
- Performing laboratory tests on samples of soft foundation soil to determine particle gradation, moisture contents, unit weight, void ratio, shear strength (Su) unconfined compressive strength (q_u), coefficient of consolidation (Cv), and permeability.
- Use of geophysical testing methods maybe considered for determining thickness of soft layers. PS Suspension logging maybe used for determination of in situ strength.

5.2.4 Design Method (EPS)

EPS is approximately $1/100^{th}$ the weight of conventional fills and therefore is highly effective at reducing driving forces or settlement potential. EPS dissolves in gasoline and other organic fluids or vapors and therefore must be encapsulated in a gasoline resistant geomembrane where such organics could potentially reach the EPS. Other design considerations for EPS include creep, flammability, buoyancy, moisture absorption, photo-degradation, and differential icing of pavement constructed over EPS.

The EPS design process includes:

- Design for external (global) stability. This includes consideration for settlement, bearing capacity, and slope stability under the projected loading conditions.
- Design for internal stability within embankment mass. The designer must insure the EPS geofoam can support the overlaying pavement and traffic loads without immediate and time dependent creep compression.
- Design of the appropriate pavement system over the EPS.
- Design to protect the EPS to resist hazards like fire and gasoline leakage- This can be done by using gasoline resistant geomembrane.
- Design for uplift pressure. This is necessary if high groundwater exists and if the 100-year flood level creates high head in surrounding areas. In some cases where uplift is an issue, the use of a cutoff wall may be necessary.
- The foundation under the EPS must be prepared to create a smooth surface and dry condition. In cases where groundwater exists, dewatering may become necessary.

External stability analyses generally follow traditional geotechnical procedures, although stress distribution must consider a non-homogenous embankment. For shear strength, NCHRP- 24-11 recommends using only ¼ of EPS geofoam compressive strength.

Internal stability analyses are based on the properties of the EPS type selected to support the imposed loads from overlying pavement and traffic. The design approach for internal stability is a deformation-based methodology using the total stress from all loads on EPS blocks, elastic limit stress, and the initial tangent modulus to evaluate load-induced deformations. Refer to FHWA-NHI-16-027, Table 3-2 for the minimum recommended values of elastic limit stress for various EPS densities.

NCHRP- 24-11 provides detailed design methods, examples, typical construction details, and design charts for external, internal, and pavement design. FHWA-NHI-16-027, Table 3-3 summarizes the range of design parameters and design considerations associated with the use of EPS.

Regarding environmental considerations, Table 3-3 of FHWA-NHI-16-027 states that there are no known environmental concerns regarding EPS and no decay of the material occurs when placed in the ground.

5.2.5 Design Method (Cellular Concrete)

The design of cellular concrete must balance the need for load reduction with compressive strength requirements. Due to high air content in cellular concrete, it generally has much lower strength than conventional concrete. Applications that require high compressive strength, such as foundations, should use higher density cellular concrete. For many applications, such as flowable fill in trench lines or behind retaining walls, the compressive strength can be as low as 100 psi. For use as lightweight fill in

embankment construction, a compressive strength ranging from 80 to 200 psi would be sufficient.

The design process for cellular concrete in embankments should include:

- Design for external (global) stability. This includes consideration for settlement, bearing capacity, and slope stability under the projected loading conditions.
- Design for internal stability within embankment mass. The designer must insure the cellular concrete can support the overlaying pavement and traffic loads without cracking and creep compression.
- Design of the appropriate pavement system over cellular concrete. Communicate with District Materials regarding the most appropriate pavement design.
- The lower compressive strength mixes are affected by freeze-thaw cycles. The product should be used below the zone of freezing or a higher compressive strength used. Densities greater than 37 pcf have reported excellent freeze-thaw resistance.
- Design for uplift pressure. Necessary if high groundwater exists and if the 100-year flood level creates a peizometric head in surrounding areas. In some cases where uplift is an issue, the use of a cutoff wall may be necessary.
- The foundation under the cellular concrete must be prepared and compacted to create a smooth surface and dry condition. In cases where groundwater exists, dewatering may become necessary. In addition, a layer of permeable material (8 to 12 inches) wrapped in filter fabric including a layer of geomembrane on top directly below cellular concrete may become necessary when excess groundwater is present.

Table 3-4 of FHWA-NHI-16-027 states that there are no known environmental concerns regarding cellular concrete.

5.2.6 Reporting (EPS and Cellular Concrete)

The GDR should include sections that:

- Justify the use of a ground modification method, such as excessive predicted settlement, foundation bearing capacity failure, or slope instability.
- Justify the selection of type of lightweight fill to be used, including such considerations as constructability, cost, and effectiveness.
- Provide detailed layout, profile and cross sections of ground modification to be treated with lightweight fill. The profile and cross sections should show limits, depth of excavation to be backfilled with lightweight material, and height of the lightweight fill material and supporting engineering results.
- If lightweight material is to be used as backfill behind retaining walls, justify its use such as reduction in active pressure and elimination of settlement and provide detailed cross sections.

- Provide instrumentation and monitoring plans and specifications (either Caltrans approved SSP and/or NSSP).

5.2.7 Considerations for Construction (EPS)

FHWA-NHI-16-027, Table 3-3 summarizes a list of important considerations such as:

- Subgrade preparations before placement of EPS blocks;
- Placement and interlocking of EPS blocks when multiple layers are used;
- Mechanical connections between EPS blocks;
- Covering of EPS blocks to prevent exposure to sunlight and displacement from wind or buoyancy.

For monitoring and construction control for EPS blocks, field monitoring should include measurements of the density and compressive strength of the materials supplied. For EPS blocks, the density and compressive strength will be a function of the grade delivered with appropriate manufacturer QC documentation. Samples should be obtained for QA testing. Observations of the placements of the blocks should also be made to confirm that the blocks are placed without a continuous joint and that shear transfer plates are installed between successive lifts of the blocks. The gasoline resistant geomembrane covering the blocks should be measured to confirm thickness and complete enclosure of the blocks. The seams within the geomembrane should be sealed properly.

5.2.8 Considerations for Construction (Cellular Concrete)

FHWA-NHI-16-027, Table 3-4 summarizes a list of important considerations such as:

- Required a staging area for batching, mixing, and placing on site;
- Required forming for placement of cellular concrete in stages;
- The lift thickness of each pour should be measured to ensure that it does not exceed the maximum thickness specified in the specifications;
- Adequate time as specified in the specifications should be allowed for cellular concrete to harden sufficiently prior to placement of the next lift. The materials must support foot traffic prior to casting subsequent lifts.
- Samples of the freshly mixed fill should be obtained at the point of placement in a manner similar to concrete testing for performance of density and compressive strength tests.

5.2.9 Specifications (EPS and Cellular concrete)

Non-standard specifications from previous Caltrans projects are available on the Caltrans intranet at the DRS and/or OE advertised projects web page. In addition, typical and sample specifications are available in FHWA NHI-06-027 and Geotech Tools website.

5.3 Stone Columns and Rammed Aggregate Columns

Stone columns and rammed aggregate columns (RAC) use aggregate to create stiff columns to increase bearing capacity, shear strength, rate of consolidation, and liquefaction resistance, and to reduce settlement. Rammed aggregate columns can be designed to provide uplift capacity. Stone columns do not provide uplift capacity. Example applications of these methods include:

- Support for roadway or bridge approach embankments over unstable soils. Examples: SON-101/Airport Blvd I/C project (EA 04-3A23U1) and MRN/SON-101 Marin Sonoma Narrows B-3 project (EA 04-264091),
- Support for structures, such as bridge approaches and retaining walls. Example: SON-101/Airport Blvd I/C project (EA 04-3A23U1, and ALA-92/880 Interchange project (EA 04- 01611)
- Slope stabilization. Example: SF-1 Mt. Lake project (EA 04-1A9021) and ALA-580, Widening project (EA 04- 4A0701),
- Liquefaction mitigation. Example: SF-1 Mt. Lake project (EA 04-44010), Seismic Retrofit project ALA-260 (EA 44010) and SD-5 (EA 11-0301U)

Stone columns are formed with gravel or crushed rock in a pattern to create a composite foundation of the columns and surrounding soil. The stiff columns carry a larger load than the surrounding soil resulting in increased bearing capacity and reduced settlement. Stone columns can be installed by either vibro-replacement (a water jetting, top feed method), or vibro-displacement (an air jetting, top or bottom feed method). However, due to environmental considerations, approval of the vibro-replacement method may be difficult to obtain in California. In both installation methods, cylindrical vibrating probes are jetted into the ground to form holes, which are backfilled with gravel or crushed rock. Pre-augering can be used to reduce the ground displacement and vibration during construction.

Rammed aggregate columns (RAC) consist of aggregate-filled drilled holes that form stiff, high density piers. However, unlike a stone column a high-energy beveled tamper typically mounted on an excavator is used to compact the aggregate. As the aggregate is rammed to form the columns, the aggregate is forced laterally into the sidewalls of the hole, partially densifying the surrounding soil. To provide uplift capacity, a metal frame anchored between the bottom lifts is included in the pier.

Both methods have the advantages of:

- Rapid installation
- Cost effectiveness compared to other foundations options
- Creating a shortened drainage path to accelerate consolidation
- Allowing for high level of compaction
- Efficient QC/QA procedures

Although both methods have similar ranges of applications, the vertical ramming force applied on RAC can develop much higher bearing capacity in the columns. RAC are more expensive, and may be subject to proprietary constraints.

Table 4 presents factors to consider when selecting either stone columns or rammed aggregate columns.

Table 4: Design Considerations for Stone Columns and Rammed Aggregate Columns

	Stone Columns	**Rammed Aggregate Columns**
Suitable materials for treatment	Clays, silts, and loose silty sands (shear strength c=300 to 2000 psf)	Soft organic clays, stiff to very stiff clays, loose silty sand, medium dense to dense sands, uncompacted fill.
Unsuitable materials for treatment	Peat, organic soil, very soft clay (c<200psf) with layer thickness greater than 1~2 column diameters	Very soft clays (c<300 psf), very loose sands (SPT<1)
Treatment depth	20-30 ft typical, up to 90 ft	7 – 30 ft
Load bearing capacity	40-60 kips typical, 110 kips max.	50-150 kips
Settlement	Reduced by 30-50% of unimproved ground	Reduced to less than 1”
Backfill material	Vibro-replacement: uniform, round to subangular gravel (1 to 2.5 inches) Vibro-displacement: well-graded gravel/cobble (3/8 to 4 inches)	Uniform gravel (2 to 3 inches)

Alternatives to stone columns and rammed aggregate columns include site preloading, excavation and replacement, driven piles, deep-soil-mixing columns, jet grout columns, and drilled shafts.

There are other emerging alternatives to stone columns and rammed aggregate columns, including vibro-concrete columns, geotextile encased columns, gravel drains, sand compaction piles, and rammed stone columns. These alternatives may prove applicable where stone columns / rammed aggregate piers are not. For more details about these alternatives, refer to Ground Modification Methods Manual.

5.3.1 Investigations

Table 5 lists design parameters for stone columns / rammed aggregate columns that should typically be obtained from field exploration. Note that not all listed parameters may be needed for a project.

Table 5- Design Parameters for Stone Columns/Rammed Aggregate Columns

Parameter	Field Exploration Method
Thickness of layer to be treated	Boring (SPT, CPT, soil tube)
$(N1)_{60}$ of untreated and treated soil*	SPT boring
Normalized tip resistance $(q_c)_1$ of untreated and treated soil	CPT boring
Shear Strength (S_u) of untreated and treated soil	• Pocket Penetrometer Test (PP) • Vane Shear Test (VS) • Torvane (TV)
Shear wave velocity (Vs) of untreated and treated soil	• Seismic CPT boring • Correlations with $(N_1)_{60}$ / $(q_c)_1$ / Su • Geophysical methods
Modulus of subgrade reaction (k)	Field plate load test

* Untreated soil: Soil that has not had ground modification treatment; treated soil: soil that has had ground modification, e.g. soil mass before and after stone column installation.

Design parameters and data that should be obtained from laboratory tests for untreated soil are:

- Particle gradation
- Unit weight
- Void ratio
- Shear strength (Su)
- Compressibility
- Coefficient of consolidation
- Permeability

Design parameters that are usually obtained from correlations with other parameters include friction angle, elastic modulus, and Poisson's ratio (see EPRI, 1990).

5.3.2 Design Methods

Support of embankments and support of structures applications requires designs that provide adequate bearing capacity and/or uplift capacity, tolerable settlement, and reduced liquefaction potential. Slope stability applications may require a ground modification design that provides specified minimum shear strength and reduces the liquefaction potential.

In general, analysis and design approaches are similar for stone columns and rammed aggregate columns. For both methods, design procedures are available in *Ground*

Modification Methods and *Geotech Tools*. Additional discussion and design examples can be found in Stone Column Design Manual and the Geopier manual.

The main design parameters to be considered include:

- Limits of treatment area
- Depth of treatment
- Replacement ratio
- Pattern of column layout
- Column diameter
- Column spacing

The effectiveness of ground treatment design is verified by in-situ geotechnical testing and/or load tests. In-situ geotechnical testing, such as CPT and SPT, are more appropriate where densification of the matrix soil is anticipated. Geophysical methods such as PS Suspension Logging and Full-Waveform Sonic Logging have been successfully used for verification of densification. Load tests usually provide more reliable verification. For both ground modification methods, verification load tests may include short-term test for ultimate bearing capacity, long-term test for consolidation settlement, and short-term horizontal shear test. Unique to rammed aggregate columns are modulus test and Bottom Stabilization Test (BST) (Fox and Cowell, 1998). The modulus test is essentially a plate load test to obtain the modulus of subgrade reaction of a test column. The BST is performed on top of the bottom bulb to verify that the column being installed has achieved general stabilization prior to the completion of installation. It is a method to determine whether a production column is comparable in quality to load test columns.

5.3.3 Reporting

The GDR should include sections that:

- Justify the use of a ground modification method, such as excessive predicted settlement, foundation bearing capacity failure, suspected liquefaction hazard, or slope instability.
- Justify the selection of stone column or rammed aggregate column method as the treatment strategy, including such considerations as constructability, cost, and effectiveness.
- Provide layout and cross sections of the ground modification area showing limits, pattern, spacing, and depths of the treatment columns, and supporting engineering calculations;
- Provide instrumentation and monitoring plans, and specifications (SSP and/or NSSP).

5.3.4 Considerations for Construction

A partial list of situations that may be encountered during construction include:

- Site clearance of underground and overhead utilities.
- Site accessibility for heavy equipment.
- Potential impact of ground movement, vibration, and noise to neighboring properties. Monitoring of the neighboring properties before, during, and after construction may be required.
- Difficult installation due to presence of rubble, concrete, abutment foundations, utilities, and other buried materials.
- For rammed aggregate columns, the presence of high groundwater combined with loose sandy material may cause caving of the drilled hole. Temporary casing may be needed to keep the holes stable.

In case the column verification testing fails to meet required performance criteria, consider adding more columns, increasing column depth, or adjusting column spacing.

5.3.4 Specifications

Non-standard specifications from previous Caltrans projects are available for both stone columns and rammed aggregate columns on Caltrans intranet. Contract specifications are also discussed in Ground Modifications Methods. In general, the specifications should include provisions on:

- Method specification (Materials, equipment, and construction procedure)
- Performance specification and acceptance criteria
- Verification testing
- Ground movement, vibration, noise control, and monitoring
- Field Inspection

6.0 References

1. *Geotechnical Solutions for Soil Improvement, Rapid Embankment Construction, and Stabilization of the Pavement Working Platform*, SHRP2 Renewal Research, Transportation Research Board.
2. *Geotech Tools, Geo-Construction Information and Technology Selection Guidance for Geotechnical, Structural, and Pavement Engineers*, SHRP2, Transportation Research Board
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5. Elias, V., Welsh, J., Warren, J., and Lukas, R., 2000, *Ground Improvement Methods Demonstration Project 116*, Federal Highway Administration, FHWA-SA-98-086.
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9. Rixner, J. J., Kraemer, S. R., and Smith, A. D., 1986, *Prefabricated Vertical Drains – Vol. I: Engineering Guidelines*, Federal Highway Administration, FHWA/RD-86/168.
10. *Guideline and Recommended Standard for Geofoam Applications in Highway Embankments*, NCHRP Report 529, Transportation Research Board, 2004.
11. *Guidelines for Geofoam Applications in Slope Stability Projects*, National Cooperative Highway Research Program, Transportation Research Board, 2011.
12. *Foundation and Soil Reinforcement Manual*, Geopier, 1998

Appendix

(from 2014 version)

Ground Improvement Methods Used by Caltrans

Method	Dist-Co-Rte	PM	Date	EA	Purpose	Information Available
Wick Drains	01-MEN-101	43/51	2010-2014	01-26200	Embankment	GDR
Wick Drains	10-SJ-12	0.2/6.8	2007-2013	10-0A840	Embankment	GDR
Wick Drains	11-SD-005	(missing data)	2002	11-0301U	Embankment	GDR, Plans, Calculations
Wick Drains	04-Sol-37	8.0/10.5	(missing data)	04-0T1411	Embankment	GDR, Plans, Calcs, Spec
Wick Drains	04-CC-680	35/36	1999	04-006091	Embankment	GDR, Plans, SSP
Lightweight Fill-Cell Concrete	04-CC-680	38/39	1998	04-254504	Embankment	GDR, Plan, SSP
Lightweight Fill-Cell Conc./EPS	04-CC-680	38/40	2002	04- 006054	Embankment	GDR, Plan, SSP
Lightweight Fill-Cell Concrete	04-CC-680	38/39	2007	04- 0060A4	Embankment	GDR, Plan, SSP
Lightweight Fill – EPS	01-Men-101	37.7/39.6	2000-2002	01-293501	Embankment	GDR, Plan, Specs
Lightweight Fill - EPS	01-Men-101	35.6/38.9	2008-2009	01-474001	Slide Repair	GDR, Plan, Spec
Lightweight Fill – TDA	01-Men-101	98.5/100.9	2005-2010	01-397511	Embankment	GDR, Plan Spec
Lightweight Fill – TDA	08-Riv-215	(missing data)	2008	(missing data)	Retaining Wall	Ret Wall backfill
Lightweight Fill-Natural and expanded shale	04-Ala-80 Emeryville	(missing data)	(missing data)	(missing data)	Emb. & MSE	GDR, Plan, SSP
Lightweight Fill-Geogrid	04- ALA-80 Frontage Rd	(missing data)	(missing data)	(missing data)	Emb. & RSP	(missing data)
Lightweight Fill-Geogrid	04- Son-116	13.7	(missing data)	(missing data)	Slide Repair	(missing data)
Lightweight Fill-Geogrid	04-SON-116	40/41	2001	04-1S0601	Slide Repair	GDR, Plan, SSP
Lightweight Fill-Cell Concrete/MSE	SCL-87	11/13	2000	04-4874R1	Embankment	GDR, Plan, SSP
Lightweight Fill-Cell Concrete	04-Mrn-101	7.4	2009	04-4S5501	Embankment	GDR, Plan, SSP
Lightweight Fill-Cell Concrete	04-SF-101	8.0-9.8	2008	04-163701	Embankment	GDR, Plan, SSP
Lightweight Fill-Geogrid	04-SON-1	20.8-1.2	1998	04- 196461	Slide Repair	GDR, Plans, SSP
Lightweight Fill-Geogrid	04-SON-1	27.0	(missing data)	(missing data)	Slide Repair	GDR, Plans, SSP
Lightweight Fill-Terramesh	04-SON-1	26.5	(missing data)	(missing data)	Slide Repair	GDR, Plans, SSP
Lightweight Fill-Soldier Pile wall	04-SON-1 Miller Creek	40.1	(missing data)	(missing data)	Soldr Pile wall Slide Reapai	GDR, Plans, SSP
Lightweight Fill-Soldier Pile wall	04-NAP-128	(missing data)	(missing data)	(missing data)	Soldr Pile Wall Slide Repair	GDR, Plans, SSP
Lightweight Fill - Geogrid	04-SON-116	13.7	(missing data)	(missing data)	Slide repair Temp Wall	GDR, Plans, SSP
Stone Column-Liquefaction	04-ALA-260	1.1-1.7 (KP)	2000-2002	04-44010	Embankment	Seismic retrofit report
Stone Column-Slope Stabilization	SF-001	6.0-9.7	2011	04-1A9021	Embankment	GDR, Plan, SSP

Method	Dist-Co-Rte	PM	Date	EA	Purpose	Information Available
Stone Column-Slope stabilization	04-ALA-580	4.9-8.9	2010	04-4A0701	Embankment	GDR, Plan, SSP
Stone Column-Liquifaction	11-SD-005	(missing data)	2002	11-0301U	(missing data)	GDR, Plans, Calculations, Spec, QA results
Rammed Aggregate Piers-Embankment and RW	SON-101Airport Blvd	(missing data)	2013	04-3A23U1	Appr. Emb Ret. wall	GDR, Plan, SSP
Rammed Aggregate Piers	MRN/SON-101	(missing data)	2014	04-264091	Embankment	GDR, Plan, SSP
Compaction Grouting	SCL-87	7.5	2007	04-930322	Sink hole/ settlement	GDR
Compaction Grouting	SCL-87	4.84	2007	04-4396U4	Sink hole/ settlement	GDR
Compaction Grouting	SCL-880	2.0/ 2.1	2010	(missing data)	Sink hole/ settlement	Grouting Plan
Compaction Grouting	SON-1	10.4	2008	04-4S3604	Settlement	GDR/FR
Compaction Grouting	07-LA-105	(missing data)	2004	07-18830	(missing data)	GDR, Plans, Calculations, Spec
Compaction Grouting	07-LA-091	(missing data)	2005	07-18220	(missing data)	GDR, Plans, Calculations, Spec, QA results
Permeation Grouting	07-LA-213	(missing data)	2003	07-4L020	(missing data)	Specifications
Jet Grouting	Oakland	(missing data)	2003	Pose/ Webster Tube	Fill Voids	Consultants-Liu done Inspection
Jet Grouting	04-ALA-260	1.1-1.7 (KP)	2000-2002	04-44010	Fill Voids	Seismic retrofit report
Injected Lightweight Foam Fill	04-ALA-580	43.2	2012	04-1F3903	Fill Voids	(missing data)
Injected Lightweight Foam Fill	04-ALA-680	0.9-1.6	2013	04-4G7003	Fill Voids	(missing data)
Injected Lightweight Foam Fill	04-CC-80	(missing data)	(missing data)	(missing data)	Fill Voids	(missing data)
Injected Lightweight Foam Fill	CC-4	5.4	2012	04-2G6504	Fill Voids	GRD/ Plan
Injected Lightweight Foam Fill	SCL-85	0.27	2010	04-4S0601	Fill Voids	GDR/Plan
Injected Lightweight Foam Fill	SON-128	5.4	2010	(missing data)	Fill Voids	GDR/Plan
Injected Lightweight Foam Fill	SOL-680	11.9	2010	04-1F6404	Fill Voids	GDR/Plan
Deep Mixing	04-SF-101	8.0-9.8	2008/ present	(missing data)	Embankment	GDR, Plan, SSP
Deep Mixing	04-SON-101	4.0-5.2	2009	(missing data)	Embankment.	GDR, Plan, SSP

Method	Dist-Co-Rte	PM	Date	EA	Purpose	Information Available
Micropiles	01-Hum-101	111.5	2010-2011	01-488303	Slide repair	Plan
Micropiles	Son-1	21.7	1995	04-193961	Slide repair	GDR, Plan, SSP
Micropiles	Son-1	21.5	2001	04-1S2801	Slide repair	GDR, Plan, SSP
Micropiles	04-92/280	10.2	2011	04-1A7701	(missing data)	GDR, Plan, SSP
Micropiles	07-LA-027	(missing data)	2001	07-45460	(missing data)	GDR, Plans, Spec

Geosynthetics

1.1 Overview

Geosynthetics are man-made polymeric materials used for geotechnical application. They are used in lieu of conventional materials and often are more cost effective with equal or improved engineering performance. The intent of this module is to provide general guidance to the practicing geotechnical designer on the proper use of geosynthetics for both design and construction. The key assumptions of this module are that the user has sufficient overall geotechnical experience and is already familiar with general civil engineering and construction practices and uses professional judgment in the decision to use and specification of geosynthetic materials. This module closely follows the design and construction procedures provided in the FHWA Publication No. FHWA-NHI-07-092 “Geosynthetic Design and Construction Guidelines” (Holtz, et al., 2008) hereafter referred to as the FHWA manual with other references cited as needed within appropriate sections of this module with a complete list given in Section 4.

The following subsections in Section 1 establish a systematic design approach, develop uniform nomenclature, and define the concept of primary geosynthetic function with associated applications. Section 2 explains design criteria and procedures and construction considerations including specifications. Section 3 explains quality control (QC), quality assurance (QA), conformance test, and construction inspection.

1.2 Design Approach

The selection of an appropriate geosynthetic is a complex undertaking that depends upon integration of available knowledge and experience and involves a number of project specific and site-specific factors. In general, the geotechnical investigation conducted for a project without geosynthetics will be adequate which is logical as most products replace conventional materials. However, specific geotechnical data such as site-specific test of subsurface/geosynthetic interaction may be needed during the investigation depending on the product function and critical nature of the application.

A systematic approach to design is needed due to the potentially large variety of geotechnical uses and available geosynthetic products. The FHWA manual recommends the following systematic approach to designing with geosynthetics:

1. Define purpose and scope of project;
2. Investigate and establish geotechnical site conditions;
3. Establish application criticality, severity, and performance criteria and identify external influences on performance;
4. Formulate trial designs and compare alternatives;
5. Create design models and parameters and perform analyses;
6. Select most effective design based on analyses and considering cost, construction feasibility, etc.;

7. Prepare detailed geosynthetic specifications and plan/details including key property requirements and installation procedures;
8. Hold preconstruction and construction follow-up meetings with Caltrans field inspectors and contractor(s);
9. Review and comment on acceptance of proposed geosynthetic materials based on manufacturer's certification and/or laboratory testing;
10. Carefully monitor construction paying particular attention to potential damage to geosynthetics caused by field operations; and
11. Inspect finished application after any significant events which could affect performance.

A systematic approach will allow a geosynthetics design which is technically robust, cost effective, constructible, and meets long term performance requirements.

1.3 Nomenclature

Due to the multitude of geosynthetic products available it is imperative to use a consistent terminology to allow a comprehensive comparison of alternatives. The FHWA manual should be consulted for more thorough definitions and detailed descriptions of geosynthetic manufacturing processes and identifying terms. For clarity some commonly used terms are defined here as follows.

Geosynthetic – A planar product manufactured from polymeric material used with soil, aggregate, or other geotechnical engineering materials as an integral part of a civil engineering project.

Geotextile – a permeable geosynthetic comprised solely of textile materials (nonwoven or woven comprised of various synthetic polymers and manufactured by numerous processes).

Geogrid – a geosynthetic formed by a regular network of tensile elements and apertures typically used for reinforcement functions (may be uniaxial or biaxial in reinforcement direction).

Geomembranes – an impermeable geosynthetic, typically used to control fluid migration (composed of various polymer compositions, surface textures, and welding/overlap methods).

Geocomposite – a geosynthetic material manufactured of two or more geosynthetic materials (e.g., a drainage geocomposite formed by heat bonding a filter geotextile and a drainage geonet together).

A good source of geosynthetic product data using the above commonly used terms is the annual December issue of the Geotechnical Fabrics Report (GFR) magazine published by the Industrial Fabrics Association International (IFAI) which is formatted as a specifier's guide and is available at www.geosyntheticsmagazine.com.

1.4 Functions and Applications

A key concept for geosynthetics design is to define the primary function of the product as this determines the necessary properties to be specified. The FHWA manual defines the six primary geosynthetic functions as follows (along with common geosynthetics used for each one):

1. Filtration (geotextile typically nonwoven but some specially manufactured wovens).
2. Drainage (geonets, drainage geocomposites (geonet-geotextile), sheet/wall drains and prefabricated vertical drains (wicks)).
3. Separation (geotextile – typically woven for less elongation and nonwoven for better drainage properties).
4. Reinforcement (geogrids, geotextiles – typically woven but some high strength nonwovens if more elongation is allowable).
5. Fluid Barrier (geomembranes, geosynthetic clay liners (GCLs)).
6. Protection (non-degradable and degradable rolled erosion control products (RECPs) such as mats and blankets, geocells, geotextiles both woven and nonwoven).

Any geosynthetic may also have secondary functions but it is usually the primary function that controls the design property requirements as outlined in tables in section 1.0 of the FHWA manual. It is important to note that the design for geosynthetic applications rely on both index and engineering properties of the geosynthetics. Index properties are not the properties that directly satisfy design demands but provide indirect measurements from which the geosynthetics can be evaluated, mostly for survivability. Engineering properties are the material properties that must satisfy the engineering demand under anticipated design conditions, such as tensile strength.

Specifications for geosynthetics commonly used for geotechnical projects can be found in 2015 Standard Specifications (Section 96) and SSPs, or NSSPs sponsored by Geotechnical Services (GS). These should be used if meeting the FHWA manual design and construction requirements for the intended use but should be checked for construction survivability which is often critical to geosynthetics long term performance.

Section 2 of this module provides a comprehensive review of the various geosynthetic use categories and associated functions that can help in deciding essential design properties of the product.

Geosynthetic Design Criteria and Procedures

2.1 Overview

The intent of this section is to identify the general design criteria and procedures by examining the main geotechnical use categories and required functions outlined in the FHWA manual (sections 2.0 through 10.0 with pavement overlay (section 6.0) omitted as subject covered by Caltrans Materials). The organization of this section closely

follows the FHWA manual so the user of this module can quickly find the key parameters needed for geosynthetics and refer to more detailed design information as needed.

It should be mentioned that geosynthetics design is often an iterative approach where various properties are selected and checked against the primary function and then modified to optimize the overall performance. Another element of design common to all products is that the final geosynthetics properties specified should be checked that they are commercially currently available from at least three sources (a useful reference is the GFR annual specifier's guide available at www.geosyntheticsmagazine.com).

It should be also noted that a critical element of all geosynthetic use is the ability to survive construction (survivability) which is much more an issue than with conventional geotechnical materials. For this reason, Section 3 of this module addresses the construction considerations that must be covered with the related subjects of specifications, conformance testing, and perhaps most importantly, installation monitoring and inspection.

2.2 Subsurface Drainage

This category covers one of the major geosynthetic uses related to subsurface drainage applications including retaining walls, prefabricated vertical drains (wick drains), blanket drains, trench/french drains, interceptor/toe drains, and seepage control. These applications use the primary functions of filtration and flow capacity (both perpendicular as measured by geotextile permittivity and in-plane as measured by geonet or sheet/wall drain transmissivity) and these two primary functions basically replace granular filters or granular drainage materials respectively.

The filtration function replaces a granular filter and as such must allow water to flow through the geotextile (typically nonwoven unless specially manufactured woven) while retaining soil particles to prevent piping and protect the drainage media. The filter must also perform for the life of the drainage system by resisting clogging. The design procedures are given in detail in the FHWA manual but key considerations are the tradeoffs between flow through the geotextile (as measured by geotextile permittivity) and retention ability of the geotextile (as measured by Apparent Opening Size (AOS)) which must be weighted depending on the actual applications (e.g., retaining wall application with drain pipe would favor flow capacity over retention criteria while a geocomposite blanket use would favor retention over flow capacity). Clogging resistance is enhanced by the largest porosity for nonwoven geotextiles or percent open area for woven geotextiles. In general, nonwoven geotextiles are most effective at filtration while woven geotextiles should only be used when specially manufactured for that function.

AASHTO provides drainage geotextile strength requirements based on severity of installation anticipated with Class 1 for severe, Class 2 (ASAHTO default selection) for less severe and Class 3 for light trench applications. Generally, the Caltrans Filter

Fabric standard nonwoven geotextile may be used (similar to AASHTO Class 2) if checked versus filter design criteria with soils to be filtered and applies for construction survivability under most conditions.

The drainage function of in-plane flow capacity replaces a granular drainage media and the key geosynthetic property is therefore in-plane transmissivity. The products commonly used are geonets, sheet/wall drains, or geocomposites (nonwoven filter geotextile heat bonded to one or both sides of a drainage geonet). A key consideration is that although the geosynthetics may have equal flow capacity its resistance to fines blocking its flow path is far less than a comparable gravel layer so the retention part of the attached filter may favor retention over permittivity. Therefore, the complete drainage system operation must be looked at during geosynthetics design. Geocomposite wall drain Standard Specifications is available and should be used if it meets all design requirements including construction survivability.

Detailed procedures and examples of filter and drain design are provided in section 2.0 of the FHWA manual. Another good source of information is to contact geosynthetic manufacturers (available in GFR annual specifier's guide) directly for case study and often even software to assist in design.

Prefabricated vertical drains (commonly known as wick drains) are used during surcharge preloading of soft foundation soils to accelerate settlement during construction and prevent bearing type failure (to allow stable embankment placement) and efficiently replace sand drains. For wick drain design, FHWA manual No. FHWA/RD-86/168 "Prefabricated Vertical Drains-A design and Construction Guidelines Manual" (Rixner, et al., 1986) should be used.

2.3 Rock Slope Protection (RSP) and Permanent Erosion Control

This category covers many important geosynthetic uses related to erosion/scour/rock slope protection applications, including shoreline or cut/fill slope protection beneath riprap or other hard armor, structure scour, and other permanent erosion control systems for ditches/streams/slopes. These applications use the primary function of protection against erosive forces under static and dynamic flow conditions.

The same criteria used for filtration design apply with generally more severe survivability and durability requirements due to the harsh and critical nature of the applications. The need for permanent erosion protection requires higher survivability and products should be selected that are permanent in nature as they must perform for the life of the protection system. Detailed procedures and examples are provided in section 3.0 of the FHWA manual.

Typical products include geotextiles (both woven and nonwoven), geocells, non-degradable rolled erosion control products (RECP) such as mats or blankets. Design may be under the direction of the landscape architect or hydraulics but it is important to

understand the design procedures and construction specifications. Improper selection of these permanent measures can severely affect overall project performance.

Standard Specifications is provided under RSP fabric which can generally be used after checked versus design requirements and evaluating the critical nature of application and meeting the extreme survivability needs anticipated.

2.4 Temporary Erosion and Sediment Control

This category covers many temporary or construction period geosynthetic uses related to sediment and erosion control/slope protection applications including silt fences, turbidity curtains, soil retention blankets, geotextile on ditches/culvert outfalls/slopes, and other non-permanent erosion control systems. These applications use the primary function of protection against erosive forces under static and dynamic flow conditions. The products selected must function for a limited period of time before permanent vegetation or other permanent erosion control measures are established.

The key consideration is the ability of the selected product to withstand maximum anticipated run-off velocities which may be under the design direction of the landscape architect and/or hydraulics. The application is included in this module for completeness and since improper selection can adversely affect other geotechnical parameters by undermining walls or cut slopes/embankments. Detailed procedures and examples of design are provided in section 4.0 of the FHWA manual. Typical products include geotextiles (both woven and nonwoven), and degradable rolled erosion control products (RECPs) such as mats or blankets.

Specifications are provided under silt fence fabric, sediment filter bag, and temporary cover in Standard Specifications Section 96, which can be used for most temporary applications.

2.5 Roadway Separation and Subgrade Enhancement

This category covers probably some of the most common geotechnical geosynthetic uses related to roadway applications including weak subgrade stabilization and protection of aggregate materials from subgrade intrusion. Due to the fact that Caltrans Materials designs permanent roadway structural section, the discussion here is limited to temporary roads such as detours, haul/access roads, working platforms, as well as placement and compaction of fill over weak subgrades. These applications use the primary function of separation by keeping weak subgrade soils from pumping through overlying fill or preventing contamination of select fill by intrusion into the subgrade. This may allow stable construction over soils that may otherwise require expensive ground improvement technologies.

The key criteria used for separation design is survivability and durability requirements due to the nature of the applications. Typical products include geotextiles (both woven and nonwoven), geogrids, or geocomposites such as a combination of geogrid and

geotextile. Detailed procedures and examples of filter and drain design are provided in section 5.0 of the FHWA manual.

Specifications is provided under subgrade enhancement geotextile, in Standard Specifications Section 96, which can generally be used after checked versus design procedures and meeting the survivability needs anticipated.

2.6 Reinforced Embankments on Soft Foundation

This category covers another significant group of geosynthetic uses related to reinforced embankments over soft foundations. These applications use the primary functions of reinforcement to allow stable construction over otherwise unsuitable conditions by preventing bearing failure in soft foundation cases. The nature of the application requires that all applications are critical even temporary slopes or embankments during construction with only reduced factors of safety used versus permanent long term stability analyses. Generally, conventional geotechnical analyses of bearing capacity, slope stability, and settlement must be performed to determine the optimal selection of primary geosynthetic reinforcing elements including vertical spacing and secondary facing reinforcement if needed for construction and long term performance. Many commercial software packages are available for these analyses which allow modeling of geosynthetic elements.

Embankments over soft foundations utilize horizontal geosynthetic reinforcing layers over the entire area of the embankment in single or multiple layers with key parameters being the Long Term Design Strength (LTDS) based on the ultimate tensile strength as measured by the wide width tensile strength test for geotextile or geogrids with a reduction factor applied (based on design life creep, backfill materials installation damage, and long term durability). Other important geosynthetic properties for reinforced embankments are the sewn seam strength, soil-geosynthetic friction (for sliding resistance). The ability of the geosynthetic to withstand severe installation conditions is also important and is well summarized in tables in section 7.0 of the FHWA manual. Also, the soils to be used for the embankment should be carefully specified to allow specified compaction even under challenging site conditions. Construction monitoring as discussed in Section 3 of this module is also very important to overall system performance. The FHWA manual section 7.0 should be utilized for full design procedures and example calculations along with typical specifications.

Key design considerations are the internal and external slope stability (using conventional geotechnical analyses methodology with the addition of tensile reinforcing elements) and ability to handle anticipated settlements and overall durability over the life of project. In general, woven geotextiles are most effective at reinforcing functions while nonwoven geotextiles may be used if secondary functions of drainage/filtration are required and/or additional elongation of reinforcing material is not a concern (i.e., typically secondary facing reinforcement use). Generally, the specifications for Geotechnical Subsurface Reinforcement described in Standard Specifications Section

96-1.02D may be used if checked versus design criteria and applies for construction survivability under most conditions.

The products commonly used are geogrids (uniaxial and biaxial) or geotextiles (typically woven but nonwoven may be used for secondary facing reinforcing uses) and reinforcing geocomposites (woven geotextile used with a geogrid over very soft foundations spoils to prevent subgrade intrusion). A key consideration is that the geosynthetic material chosen must have sufficient construction survivability which includes strength of sewn seams or sufficient overlap if sewn seams are not required.

Another good source of information is to contact geosynthetic manufacturers (available in GFR annual specifier's guide) directly for case study and often even software to assist in design. Many commercially available slope stability software programs can be used which allow modeling of geosynthetic reinforcement materials.

2.7 Reinforced Slope and Mechanically Stabilized Embankment Wall

This category covers another significant geosynthetic use related to reinforced soil slopes (RSS) and mechanically stabilized embankment (MSE) retaining walls. Reinforced soil slopes are defined as those up to 70 degrees in slope angle. MSE walls are defined as those geosynthetic reinforced structures with facing slope angles from 70 to 90 degrees. These applications use the primary functions of reinforcement to allow stable construction at steeper face angles than possible without reinforcement. A brief discussion is provided below but the user should carefully follow the FHWA manual sections 8.0 and 9.0 for full design procedures. The nature of the application requires that all applications are critical even temporary RSS and MSE walls during construction with only reduced factors of safety used versus permanent long term stability analyses. Generally, conventional geotechnical analyses of bearing capacity, slope stability, and settlement must be performed to determine the optimal selection of primary geosynthetic reinforcing elements including vertical spacing and secondary facing reinforcement if needed for construction and long term performance. A key feature of RSS and MSE walls is that permanent structures usually have some type of facing to protect the exposed elements. Many commercial software packages are available for these analyses which allow modeling of geosynthetic elements.

RSS and MSE walls utilize horizontal geosynthetic reinforcing layers from the face of the wall for a minimum required reinforcement length with key parameters being the Long Term Design Strength (LTDS) based on the ultimate tensile strength as measured by the wide width tensile strength for either geotextile or geogrids. Other important geosynthetic properties are soil-reinforcement interaction (for pullout resistance and interface friction), and overall system creep resistance. These are discussed briefly here but the FHWA manual should be used for full design procedures and example calculations along with specifications. Additional references for MSE and RSS applications are included in the Section 4.

The design procedures are given in detail in the referenced FHWA manual but key considerations are the internal and external slope stability (using conventional geotechnical analyses methodology with the addition of tensile reinforcing elements) and ability to handle anticipated settlements and overall durability over life of project. The specifications for Geotechnical Subsurface Reinforcement in Standard Specifications Section 96-1.02D may be used if checked versus design criteria and applies for construction survivability under most conditions.

The products commonly used are geogrids (uniaxial and biaxial) or geotextiles (typically woven but nonwoven may be used for secondary facing reinforcing uses) and reinforcing geocomposites (geotextile used with a geogrid). A key consideration is that the geosynthetic material chosen must have sufficient construction survivability which includes sufficient overlap or connection methods.

Detailed procedures and examples of RSS and MSE wall design are provided in sections 8.0 and 9.0 of the FHWA manual and in the FHWA Demonstration Project 82 (FHWA publication No FHWA-SA-96-071). Another good source of information is to contact geosynthetic manufacturers (available in GFR annual specifier's guide) directly for case study and often can provide software to assist in design. Generic programs are also available from FHWA which can allow rapid checks of various geometries and material properties for fine tuning of design.

2.8 Geomembrane and Fluid Barrier

This category covers a wide variety of potential uses related to roadway applications including fluid barrier around lightweight geofoam (to prevent hydrocarbon damage), cut-off walls, tunnel linings, control of moisture in expansive subgrade soils, waterproofing of walls/bridge abutments, and detention/retention basin liners. Because this application is very specialized, the discussion is limited and the FHWA manual should be consulted for more design and specification information.

The key criteria used for barrier design is leakage prevention as well as survivability and durability requirements due to the nature of the applications as well as friction between subgrade and material selected (textured surfaced products are available for increased friction). Typical products include geomembranes (comprised of various polymeric compounds including polyethylene (High Density Polyethylene (HDPE) and non-HDPE, polypropylene, and Polyvinyl (PVC) being the most common) and Geosynthetic Clay Liners (GCLs) (composed of bentonite clay bonded to a geomembrane or sandwiched between geotextiles commonly used in environmental containment applications). Each material has specific methods of welding/overlapping sheets together for water tightness and highly developed QC/QA protocols. Detailed procedures and examples of fluid barrier design are provided in section 10.0 of the FHWA manual.

CONSTRUCTION CONSIDERATIONS

3.1 Specifications

Although this Section covers the specifications which are prepared during design and could therefore be discussed in Section 2, it is during construction that their effectiveness is tested. This is particularly important for geosynthetic materials as their overall function is heavily influenced by how they are installed and their susceptibility to damage during construction. It is probably the one true disadvantage of geosynthetics use in that improper installation is the weakest link in their performance and therefore must be mitigated by well prepared specifications and construction monitoring covering all aspects of installation. The FHWA manual provides an excellent discussion of the six elements that should be included in all geosynthetic specifications as follows.

1. General Requirements (such as storage and handling requirements including ultraviolet resistance).
2. Specific Geosynthetic Properties (both specified index and performance tests if needed).
3. Seams and Overlaps (as applicable for design function and product selected).
4. Placement Procedures (including minimum cover and maximum equipment ground pressures if required).
5. Repair Procedures (if damaged during placement).
6. Product Acceptance and Rejection Criteria.

These are discussed at length in section 1.0 of the FHWA manual and guide specification examples for each category are provided in appropriate design sections. Standard Specifications Section 96 should be used for most applications but must be checked to ensure that they have the key elements of geosynthetic specifications outlined above and meet design requirements. The following section discusses an additional element that must be included in the specifications for larger projects and critical uses and involves procedures to accept or reject geosynthetic materials proposed for use.

3.2 Quality Control, Quality Assurance, and Product Conformance Test

For large projects and critical applications special Quality Control (QC), Quality Assurance (QA), and product conformance test requirements may be needed.. For these critical use projects, acceptance criteria should be clearly and concisely stated in the specifications. For smaller projects, a manufacturer’s certification letter may be satisfactory for non-critical applications and this should be clearly stated in the specifications. The FHWA manual provides an example of geosynthetic specification conformance example per ASTM 4759. The following section covers the most critical consideration for use of geosynthetics in geotechnical practice which is the need for vigilant construction inspection.

3.3 Construction Inspection

This section covers what is perhaps the most critical element of successful geosynthetics use. Many problems with geosynthetics use are directly attributable to improper installation or damage caused by contractor operations. The FHWA manual provides a field inspection checklist (table in section 1.0) which is a good starting point but it is most important to actually observe the installation and placement procedures used by the contractor in the field. In this regard, it is very important to educate Caltrans construction personnel on what to look for during placement of geosynthetics and backfill as it must be performed continuously which will require full-time knowledgeable inspectors with regular visits by the geotechnical designer.

The key criterion is construction survivability which varies depending on the geosynthetics, foundation and backfill materials, applications, construction equipment, and construction methods. In any case, geosynthetics must survive construction and provide primary and secondary (if needed) functions through the design life of the project. The more critical the nature of the application warrants more careful construction inspection. Detailed procedures and examples of proper construction for various products are provided in the appropriate sections of the FHWA manual.

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Emergency Response

Emergency conditions are defined in this document as damage to the state highway system that are recent, unexpected, and event driven. Events may include storms, floods, slides, fires, earthquakes, terrorist activity, and unusual accidents that cause or threaten to cause a closure of a transportation facility *(California Transportation Commission Resolution G-00-11)*. Caltrans Geoprofessionals (GP) providing emergency response (ER) service will aid clients in the remediation and repair of damaged highway facilities due to events such as: rockfall, landslides, slipouts, embankment washouts, culvert distress, sinkholes, slope erosion, fire damage, debris flows, flash flood, tsunamis, scour, structure collapse, foundation damage, ground rupture, mud pots; and rapid ground settlement due to lateral spreading, mine shafts, buried wells, and unauthorized excavations by the homeless.

During ER work, the GP will rely on applicable sections of the Quality Management Plan (QMP) and Geotechnical Manual. The procedures detailed in the QMP and Geotechnical Manual may be expedited or abbreviated depending on the site conditions, severity of damage, and urgency of restoring operation. For example, when responding to collapse of a bridge on an interstate freeway in a rural area with no surrounding development, mobilization for exploratory drilling may begin while the Site Assessment Questionnaire or County Well Permit are being processed. When requesting the necessary clearances, the GP should vocally express the urgency of the circumstance to Caltrans or County staff who have the authority to expedite clearances. The Underground Service Alert process may be shortened by contacting utility company executives and arranging a next day field meeting.

Emergency response projects are dynamic and fast-paced relative to programmed projects, so it is essential to address the priorities of Geotechnical Services (GS) clients quickly. The top priorities are assuring safety, maintaining or establishing access for emergency vehicles, determining the scope of necessary repairs to restore the facility, developing geotechnical reports to convey design information, and providing field support as repairs are constructed.

Communication and Contacts

Ideally, requests for emergency support are made by the District Emergency Response Coordinator to the GS Office Chief that serves the District. However, requests for support and information concerning an unfolding emergency may be made directly to GPs through numerous conduits. Regardless of the source or receiver of the request, the Design Office Chief shall be notified of all emergency support requests within their region. The Office Chief will ensure there is proper coordination between the assigned Branch, clients, and stakeholders. The Office Chief will notify the Deputy Division Chief of all significant emergencies requiring GS support, and provide status updates as work progresses.

For each emergency, the GS Branch Chief will open an ER Project Tracking Record. The Tracking record shall be created at the first available opportunity after a request for emergency support has been received and will include preliminary information on the

location and nature of the emergency. The Tracking Record provides GS Management with information on projects that are actively receiving emergency support from GS, and on the scope and status of support.

Each GS Branch shall become familiar with the Emergency Operations Plan and protocols for the Districts they serve. District Emergency Operations Plans are typically available through District Maintenance, the District Emergency Response Coordinator, or the District Intranet site. GPs shall adhere to District protocols and paths of communication developed for emergencies. Emergency contact information for each GS Design Office will be provided to District staff.

Emergency Management and Support

The Districts are the owners of all emergency projects within their jurisdictions. Management of emergencies will be directed by the Districts and supported by the GS Design Offices upon request. The emergency procedure and response will vary between Districts and vary depending on the size and complexity of the emergency.

The Branch Chief will assign a qualified GP to provide emergency support and ensure immediate response to all emergencies. Branch Chiefs and GPs should become familiar with the highways, problematic sites, and Maintenance personnel in the regions they serve; this practice will facilitate client service and interaction.

During ER efforts by the GS Design offices, OGS will prioritize services to emergency projects. OGS will recognize the need to expedite the delivery of geotechnical products and collaborate flexibly with GPs providing direct support to the Districts. OGS will expedite services including exploratory drilling, laboratory testing, and drafting of LOTB.

During ER efforts by the GS Design offices, OGDPP will prioritize and provide expedited reviews of reports produced in support of emergency projects. For more challenging emergency efforts, OGDPP staff may be enlisted to serve as technical experts on the Project Development Team or as geotechnical design staff assigned to the project.

GS Emergency Response Procedures

GS will provide timely and consistent responses to requests for emergency support, however, the response will vary somewhat depending on the emergency condition, the needs of the client, and the ER effort coordinated by the District. GPs will participate on Damage Assessment Teams and PDTs. In general, for non-structural damage, GS will provide evaluations and recommendations to the District Maintenance Engineer. For damage affecting structures, GS will provide evaluations and recommendations to Structure Design (SD).

Typical steps for emergency response:

A. Initial request and preparation for field work

1. Receive request from client. Use the checklists presented at the end of this document as a guide to gathering pertinent information. Review relevant portions of the Geotechnical Manual as a guide to gathering information on specific features.
2. Notify the appropriate person (e.g., Branch Chief) that an emergency support request was received. In conjunction with the Office Chief, the Branch Chief will determine which qualified staff will respond to the emergency. When emergency support is anticipated, such as before an approaching storm, a Branch Chief may predesignate staff to respond to anticipated emergency sites.
3. The Branch Chief or designee will create a Project Tracking Record, fill in the information found in the GS Storm Damage Status spreadsheet as thoroughly as possible, and update the Tracking Record throughout the project.
4. Gather pertinent information such as as-built plans, utility plans, drainage plans, right-of-way maps, Google Earth views, and archived geotechnical documents relevant to the site.
5. Create and begin electronic file storage according to Section 10.3 Project Records Management System of the Quality Management Plan (QMP).
6. Obtain a state vehicle and gather field gear. Wear personal protective equipment at the site and carry your Caltrans ID badge. The GP should travel to the site in a State vehicle because law enforcement and Caltrans Maintenance will more readily let you through road closures, and you will be more readily identified by those who are monitoring the site or who need to interact with you.

B. Initial field visit

7. At the site, sign in if necessary. Meet, record, and interview contacts. Question Maintenance staff on site history and behavior. Thoroughly review the site. Take photos and measurements of the site and affected features. Make sketches (with dimensions) of distress features, site layout, and cross sections. Consider possible detours (some detour options may influence your recommendations). If a Contractor has been retained, determine the Contractor's capabilities. Anticipate pressure to provide quick recommendations or preliminary design, and only offer recommendations after thorough site review and evaluation. It may be necessary to delay repair recommendations until after a formal site investigation.
8. Determine if the road should be opened or closed. Consider the safety of the public, Caltrans forces, and construction crews. Consider construction activities necessary for repair that may avoid disrupting live traffic, if possible. Discuss with the District the possibility, duration and implications of construction repairs during night shift. Consider the duration of repair activities.

9. Determine the effect or severity of the emergency: e.g., scour failure of a bridge on a major freeway or interstate is far more consequential than washout of a rural highway with low Average Daily Transit and available local street detours. Obtain the needed geotechnical resources using appropriate means given the urgency and magnitude of the situation.
10. While in field, consider potential repair strategies. Use the Relocation-Avoidance /Stabilization/Protection evaluation protocol as outlined in the Landslide Module. Consider repair strategies that align with Contractor's capabilities. Consider immediate action vs. permanent restoration. Convey immediate repair recommendations to the Caltrans person in charge (likely a Resident Engineer (RE)). Stay at the site as necessary to provide support and oversee immediate repairs. If necessary, get support from your supervisor or other team members.

C Preparation/ back in office

11. The determination of both immediate and long-term repair strategies may not be possible in the field. Back at the office, develop and evaluate repair alternatives with your supervisor and team members. Recognize that GS often takes the lead in developing repair strategies, particularly for embankments, retaining walls, slope stability, and rockfall. Consider all stakeholder interests and site constraints. A decision matrix may be useful (see Landslide module for example decision matrix). Interact with the PDT. While some ER guidelines promote the concept of "replace in kind," repaired features should conform to current design standards (e.g. shoulder width). Betterments may be permitted if they prevent recurrence.
12. Determine the additional necessary work including subsurface investigation, surveying, instrumentation and monitoring. Typical site investigation requirements may be abbreviated if the highway or freeway is closed because of the emergency condition. Consider the investigative resources and equipment that are available. If an emergency event has caused the closure of a major freeway or interstate with no reasonable detour, the site investigation may proceed concurrently with some construction activities or may be minimized using conservative design assumptions. Develop a site investigation commensurate with the conditions being considered.
13. Determine which functional units will be involved including District Maintenance, Design, Hydraulics, Right-of-Way, Traffic, Environmental, and Construction. Contact the stakeholders and consider repair strategies that best address their concerns. Coordinate through a single focal point such as the District Emergency Response Coordinator, Project Manager, or Project Engineer.
14. If a structure solution is necessary, GS will inform other DES functional units such as Structure Maintenance and Investigations, SD, and Structure Construction.
15. Determine GS deliverables. Develop and provide appropriate draft or preliminary reports to client as necessary to expedite project progress.
16. Working with District and Structure partners, decide on the final repair strategy and help the team to determine if the work will proceed by Emergency Force Account,

Emergency Limited Bid, or as a Design Project (PS&E). Relatively small repairs and projects may involve relatively few stakeholders and functional units while large repairs and projects may involve a Project Development Team.

17. Finalize reports. Document decisions that lead to the selected repair strategy. Document reasons why geotechnically preferred strategies are not implemented (such as environmental considerations). Provide cost estimates for solutions that may not be familiar to clients, such as reinforced embankments, rockfall protection systems, or debris flow barriers. Given the accelerated delivery schedules and urgency of repair, some emergency projects may be constructed based on the findings and recommendations contained in preliminary geotechnical reports. Final geotechnical reports may be prepared near the completion of construction to document site conditions that were discovered, and designs that were implemented, as emergency repairs progressed.
18. Provide GS construction support through completion of all geotechnically significant project components. For many emergencies there are no formal Project Plans or Special Provisions. Other emergencies will have only abbreviated Project Plans. The RE and Construction Inspectors may direct the Contractor's work with the support of the assigned GP. Often, REs and Inspectors assigned to the ER effort will have little or no prior emergency support experience while the GP will have worked on numerous emergency projects. In this circumstance, the GP will add valuable experience and confidence to the team.
19. Continue support until construction is complete to ensure proper construction, function and protection of geotechnical elements of the repair (e.g. ensure culverts do not outlet onto new fill slopes).
20. Document progress and decisions in Project Tracking throughout the emergency effort.

Emergency Response to Regional Events

Events such as earthquakes, heavy and/or prolonged rainfall, or fires may adversely affect numerous structures, roadways, and slopes throughout a wide region. The District Emergency Response Coordinator will coordinate the Caltrans response during widespread emergency efforts. One GP will coordinate the GS emergency response effort and serve as the GS Emergency Response Coordinator. The GS Emergency Response Coordinator will serve as the focal point of communication between the District, GP, supervisors, and managers; and DES Management. The GS Storm Damage Status spreadsheet is attached to assist with coordination and communication of the GS effort.

During large scale emergency events, experienced GPs from unaffected regions may be needed to assist the local staff; the Office Chiefs from all GS offices will work cooperatively as necessary to effectively staff the emergency effort. Some GPs may assist in the field while other staff perform analyses and produce reports at their home office. GPs providing field response to large scale emergencies must be prepared for difficult conditions that include numerous closed routes, difficult site access, difficulty

finding lodging, shortages of equipment and material, frustrated citizens, long work hours, and physical hazards.

GS Emergency Support Products

Just as the GS emergency response will vary based on several factors, the products provided to GS clients will vary. The emergency effort may unfold rapidly, recommendations may be conveyed verbally, and communication can often be haphazard or misinterpreted. Verbal communication should be followed by written documentation either by email or memorandum. Final emergency assessments and recommendations shall be documented and conveyed in a report. Preliminary or draft reports help facilitate communication and progress during the response effort.

Maintenance Support

For work performed as Maintenance Support Tasks M2 and M3, a Maintenance Support memorandum will be prepared using an abbreviated GDR format as determined to be appropriate by the Branch Chief. At a minimum, the report should include an introduction, description (with history), assessment, recommendations, and cost information that is not otherwise readily available. The report should be titled Maintenance Support, followed by a few-word description of the emergency condition (e.g. Maintenance Support: Interstate 8/Hill Street Landslide). The Maintenance Support memorandum should be completed as soon as possible after the support request and site review.

Design Support

For non-structure emergency work performed as a Capital Project, a more detailed Maintenance Support memorandum, GDR, or FR will be prepared as determined appropriate by the Branch Chief.

The document guiding the ER of SD is Structure Design Emergency Director's Order Project Delivery Procedure – Projects Requiring Division of Engineering Services. SD will participate in the ER efforts involving the design and construction of new structures in two distinct ways:

1) SD will review non-standard retaining wall plans developed by contractors and their design consultants. GS may provide geotechnical information or reports for the design of retaining walls to the emergency contractors and design consultants.
2) SD will provide a complete design package for the construction of new structures (retaining walls and bridges). GS will provide preliminary or final Foundation Reports as necessary. Preliminary information, evaluations, and opinions of GPs will factor heavily into appropriate structure type selection. SD team members can proceed with the design process as investigative information and final reports are developed by GS.

Construction Support

GS will provide construction support to the Maintenance Engineer, Structure Construction Representative, or RE charged with administering emergency repairs. A qualified GP will be at the project site to act as a technical resource and to assure proper construction of geotechnically significant features, whether constructed with or without the benefit of plans and special provisions. The GP will not serve as a permanent Construction Inspector but may fill in as a temporary inspector under special arrangement with the District.

Construction support involves observations, direction, agreements, and verbal communications in the field which require follow-up documentation in an email or memo. The documentation provided by the GP should summarize any conversation in the field and be specific enough to produce the desired outcome. The correspondence should also have enough information to act as a stand-alone document.

Definitions

State Highway Operation and Protection Program (SHOPP):

SHOPP is the “fix it first” program that funds emergency repair, repair and preservation, safety improvements, and some operational improvements on the state highway system.

SHOPP Major Damage Program:

To expedite emergency projects so that normal operation of a transportation facility may be resumed, Caltrans has implemented the SHOPP Major Damage Program to allow exceptions to the formal advertising, bidding, and award requirements of the State Contract Act. GS involvement in emergency projects may occur in response to a major disaster or in response to a localized condition. The complexity of the District response will vary according to the magnitude of the emergency.

During or soon after a major disaster, the District Emergency Response Coordinator begins the initial disaster assessment to estimate the scope of damage, scope of repair work, cost of repair and ER funding eligibility. These first evaluations are usually based on rapid inspections and the resulting cost estimates are summarized by the Headquarters Major Damage Coordinator (Division of Maintenance) and/or by the Division of Local Assistance Emergency Response Coordinator and are used by the Governor to justify a State of Emergency declaration.

After the Division Administrator indicates there will be a positive natural disaster determination, the District Major Damage Coordinator organizes teams to conduct detailed damage assessments. Independent assessments may be conducted by a Damage Assessment Team comprised of knowledgeable Department staff.

Damage Assessment Form (DAF):

The DAF is the U.S. Department of Transportation Federal Highway Administration (FHWA) – California Division – Title 23 Damage Assessment Form. The DAF may be used by assessment teams to document site-specific repair estimates used to develop supporting material for programming purposes. The District Major Damage Coordinator keeps the original version of the DAF and copies are given to the FHWA, the Office of Federal Resources (OFR) Emergency Response Coordinator, and the Headquarters Major Damage Coordinator.

Maintenance Support Task M2:

Maintenance Support Task M2 (charging code M2) is used to respond to District Maintenance requests for non-emergency roadside work (i.e. slopes, drainage/vegetation) and rock scaling assessment. Maintenance Support Task M2 is not for use with storm damage or emergency related issues. Project I.D. 0000020717. When first contacted and prior to site review, it may not be clear if GS support will be emergency or non-emergency.

Maintenance Support Task M3:

Maintenance Support Task M3 (charging code M3) is used to respond to District Maintenance requests for preliminary geotechnical investigation for storm damage or other emergency situations (including scaling assessments) where development of a capital outlay Project ID will not occur or is not known. When a Capital Project ID has not been established in the district, Division or Engineering Services can use Project ID 0000000999 for up to 2 weeks. Time sheet corrections are submitted once the Capital Project ID is established.

Director’s Order:

A Director's Order is a formal document approving the use of the special authority granted to the Director by state law to set aside normal procedures for the advertising, bidding, and award of certain types of contracts when there is an emergency or other urgent situation. The purpose of a Director's Order is to contract for emergency work more quickly than through the normal contract approval process. The Director's Order speeds up reopening and reconstruction of damaged transportation facilities.

A District Director’s Order is issued by a District Director for project costs less than or equal to $281,000. A Caltrans Director’s Order is issued by the Caltrans Director for project costs greater than $281,000.

Emergency Opening:

An Emergency Opening consists of temporary or partial repairs made during or immediately after a disaster intended to: 1) allow essential traffic, 2) minimize the spread of damage, and/or 3) protect the remaining facilities. Emergency Opening work typically starts after a Director’s Order (SHOPP 130 program and Deputy Directive DD26-R2).

Repairs required to restore the facility to pre-disaster conditions are considered Permanent Restoration. However, Permanent Restoration work done under Emergency Opening procedures per FHWA policy, may be eligible as part of an Emergency Opening project.

Federal authorization is not required before beginning emergency opening work, including Preliminary Engineering, Right of Way, and Construction. However, once FHWA acknowledges that a Disaster declaration is justified, the District submits an Emergency Opening – Federal Funds Authorization Request (EO-FFAR) form to the OFR Area Engineer. The OFR Area Engineer will submit an authorization request (E-76) to FHWA for the Emergency Opening phases and its approval will be retroactive to the Disaster Declaration Date.

Permanent Restoration:

Permanent Restoration involves repairs needed after emergency openings to restore the highway to its pre-disaster condition. This phase of work usually requires the preparation of plans, specifications and estimates for design (SHOPP 131 program). The replacement of bridges, construction of retaining structures, highway relocations or the addition of significant protective measures are usually considered permanent restoration. Any additional features or changes in character from that of the pre-disaster facility are generally not eligible for ER funding unless justified by economy of construction, prevention of future recurring damage, or technical necessity.

Permanent restoration construction projects must be federally approved (E-76) for construction by the end of the second federal fiscal year following the year in which the disaster occurred. These projects have priority over all non-emergency SHOPP projects in the District.

Emergency Force Account (EFA):

Force account means a basis of payment for the direct performance of highway construction work with payment based on the actual cost of labor, equipment, and materials furnished and consideration for overhead and profit.

EFA contracts are used for emergencies requiring immediate action because of road closure or danger to public safety. An example of an EFA contract is the repair of a highway section washed away by a mudslide. EFA contracts do not require bids.

Caltrans interprets Force Account to mean a solicited time-and-materials contract pursuant to the Extra Work provisions of the Standard Specifications. Caltrans does not include work by its own employees in its definition of Force Account.

Caltrans Force Account contracts are not advertised. Any contractor that has the necessary equipment and expertise to perform the emergency work may be hired; there is no pre-qualification process. Payment for labor, materials and equipment used are paid for pursuant to Chapter 9 of the Caltrans Standard Specifications. Plans are not required

(rough plans, if possible, are recommended). Work proceeds at the direction of the (RE). The RE may prescribe means and methods to the Contractor.

Emergency Limited Bid (ELB):

An ELB contract is a modified type of force account contract that includes a competitive bidding element. The ELB method of contracting generally is used for permanent restoration emergency work and occasionally for emergency opening work. These contracts are used for emergency work, even when the facility is stable. ELB contracts require bids from at least three contractors. Contractors compete based on markup rates of the prime contractor's labor, equipment rental, and materials.

Caltrans Districts obtain the required approvals for emergency work and select contractors to perform emergency work under EFA and ELB contracts. Contractor selection is based on several factors such as, proper licensing and registration, ability and expertise, proximity to the site, willingness to mobilize quickly, or by lowest bid. Caltrans maintains a list of potential contractors available for emergency work. Contractors may register on the Caltrans Contractor Registry.

Project Initiation Report (PIR)/Project Initiation Document (PID):

A PIR or PID is an engineering document or technical report that documents the scope, cost, and schedule of a project. The PIR is based on the project scoping effort. The PIR is a record of the purpose-and-need for the project, and the approach that will be taken to meet or reduce transportation deficiencies. It is a record of the existing information, initial assumptions, identified risks, and constraints that drove the development of the project work plan. A PIR is used to obtain approval for inclusion of a project into a programming document or to get conceptual approval of a project-funded-by-others.

Emergency Checklists

The GP should review relevant sections of the Geotechnical Manual when responding to emergencies involving specific highway features (e.g. review the Rockfall portion of the manual when responding to a rockfall emergency). The following checklists are provided to assist in the gathering of information needed for the ER effort:

Preliminary Information:

- Name, title, and contact information of requestor
- District-County-Route-Postmile
- EA or ID if available
- Name of Site/Event
- Names, titles, and contact information of involved office and field staff
- Current closures and travel restrictions

- Available/potential detours
- Nature of emergency
- Site or event history
- As-built plans
- Aerial photos

Recommended field equipment and provisions:

- Personal Protection Equipment
- Base map
- Field notebook
- Aerial photos
- Clinometer
- Range Finder
- Pocket measuring tape
- 100 to 200 ft measuring tape
- Brunton or Clar Stratum Compass
- Camera
- Geological rock hammer/pick
- Sample bags and shovel
- Survey marking tape (multiple colors)
- Wooden stakes/lath
- Handheld GPS
- Binoculars
- Flashlight
- Water and Food
- Gas Meter
- Climbing Gear

Landslide/Slope failure:

- Potential detours/alternate routes
- Geologic map
- Right-of-way plan/map
- Base map
- Material types, estimated strengths
- Slide type
- Estimated thickness
- Create landslide map: slide boundary, scarps, cracks, slope angles, erosion, ponds, seeps and springs
- Create cross section(s)
- An assessment of the likely slide behavior, mechanism, and influencing factors
- Preliminary field evaluation of mitigation alternatives and risk.
- Scope and Cost Estimates

Washout:

- Height, length, and volume of embankment
- Material type, estimated strength
- Slope angle
- Potential detours
- Cause (plugged culvert, overtopped dike, etc.)
- Culvert size, type
- Potential material sources

Rock Fall:

- Slope Assessment Form
- Right of Way plans/maps
- Rock type
- Type of failure, e.g., Slide/Topple/Wedge
- Discontinuity type, orientation, and spacing

- Range of block sizes
- Rock fall volume
- Rock weathering
- Slope angle
- Fall height
- Surface water
- Seeps
- Impact points
- Runout

Sinkhole:

- As-built drainage plans
- As-built utility plans
- Sinkhole drawing with dimensions
- Culvert size, condition, perforations, cavities
- Pavement structural section
- Pavement undermining
- Storm data
- Alternative path to reroute drainage temporarily or permanently
- Cause

Burn Area:

- Right of Way plan/map
- Presence of debris flow geomorphology
- Culvert size and condition below burned watershed
- Inlet basin size and condition
- Bridge channel condition below burned watershed
- Size and slope of watershed
- Burned Area Emergency Response (BAER) Report

Questions the GP should anticipate during emergencies:

- What is the effect on the roadway? On private property?
- How big was the event? What caused it? Did anyone witness the event? Were there injuries?
- Were any utilities damaged?
- Can we open the road? What needs to be done to open the road?
- Is it safe?
- Is it done moving? How fast will it progress?
- What are some immediate management strategies?
- Do we need a barrier (K-rail, temporary rock fence, etc.)?
- Do we need a spotter?
- Do we need a light plant?
- Do we need signs?
- Do we need closures windows? (times, such as night, the section would be closed to traffic or perhaps limited to only local traffic)
- What is the recommended repair? How much will it cost? How long will it take?