State of California **Business, Transportation and Housing Agency**

Memorandum

To: ALL TRENCHING & SHORING MANUAL HOLDERS

Date: January 31, 2000

File: T&S Manual

From: DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
DIVISION OF STRUCTURE CONSTRUCTION

Subject: Trenching & Shoring Manual Revision No. 12

Attached is a recent revision to the Trenching and Shoring Manual.

Trenching and Shoring Memo 5, “Shoring Adjacent to Union Pacific Company Railroad Tracks”, has been included to describe the guidelines for the design and construction of shoring adjacent to the Union Pacific Railroad Company tracks.

Please remove and recycle “Index to Appendix H – Trenching and Shoring Memos" dated 12/96.

Please insert the “Index to Appendix H – Trenching and Shoring Memos" dated 01/00, “Trenching and Shoring Memo 5 (01/00)”, and the title sheet for Revision 12 (01/00).

Original signed by R.P. Sommariva

R.P. SOMMARIVA, Chief
Division of Structure Construction

Attachments

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

TRENCHING AND SHORING MANUAL

ISSUED BY
DIVISION OF STRUCTURE CONSTRUCTION

Copies of this manual may be obtained from:

Division of Structure Construction
Attention: Manual Coordinator
Phone (916) 227-8980
Fax (916) 227-8179

http://www.dot.ca.gov/hq/esc/construction/construc.htm

JANUARY 1990

Revision 12
01/00

STATE OF CALIFORNIA

DEPARTMENT OF TRANSPORTATION

TRENCHING
A N D
SHORING
MANUAL

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Revision 11
12/96

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INTRODUCTION

NOMENCLATURE

C = Cohesive intercept: The component of soil shear strength which is independent of the force pushing the particles together.

E = Modulus of elasticity (psi)

GW = Ground water surface

I = Moment of inertial (in^4)

K_a = Lateral earth pressure coefficient for the active pressure condition

K_o = Lateral earth pressure coefficient for the at-rest condition

K_p = Lateral earth pressure coefficient for the passive pressure condition

K_w = Equivalent fluid soil pressure *(pcf)*

N = Standard penetration resistance

N_c = Bearing capacity factor

N_0 = Stability number

Q = Level surcharge loading (pcf)

q_u = Unconfined compressive strength (psf)

S = Section modulus (in^3)

SF = Safety factor

S_U = Undrained shear strength

α = Alpha - Angle from vertical to center of surcharge strip
β = Beta - Angle of soil slope
γ = Gamma - Unit Weight of soil (pcf)
δ = Delta - Wall friction angle
ϵ = Epsilon - Linear strain
θ = Theta - Angle of repose
μ = Mu - Angle of tieback with horizontal
ρ = Rho - Degree of flexibility of an anchored bulkhead (Rowe's Moment Reduction theory)
σ = Sigma - Normal stress
Σ = Sigma - Sum
τ = Tau - Soil shear stress
υ = Upsilon - Poisson's ratio
ϕ = Phi - Angle of internal friction of soil
ψ = Psi - Failure wedge or slip angle
ω = Omega - Angle of the wall with respect to vertical

FHWA = Federal Highway Administration
Cal-OSHA = California Occupational Safety and Health Administration
Cal/OSHA = Cal-OSHA
AREA = American Railway Engineering Association

CALIFORNIA TRENCHING AND SHORING MANUAL

PREFACE

The California Department of Transportation Trenching & Shoring Manual was originally developed by the Office of Structure Construction in 1977. Its purpose was to provide technical guidance for Structure's field engineers analyzing designs of trenching & shoring systems used in the California Highway Construction program.

This 1990 updated manual edition continues to be devoted to the analysis of trench and excavation earth support (shoring) systems. Its main objectives are to inform the Engineer of California's legal requirements, and to provide updated technical guidance for analysis and review.

The Engineer should bear in mind that this manual is a book of reference and instruction to be used in respect to the administration and engineering of excavation shoring. In cases of conflict, the contract documents shall prevail.

Current concepts in soil mechanics or geotechnical engineering are summarized in order to better acquaint the reader with the practical considerations and accepted application of theoretical principles. Situations or conditions which may cause difficulty are noted. A section on earth tiebacks has been added to this 1990 edition.

The engineering objective of a shoring systems is to be both safe and practical. There are two major parts of the engineering effort. First is the classification of the soil to be supported, determination of strength, calculation of lateral loads, and distribution of lateral pressures. This is the soil mechanics or geotechnical engineering effort. Second is the structural design or analysis of members comprising the shoring system. The first part, the practical application of soil mechanics, is the more difficult. The behavior and interaction of soils with earth support systems is a complex and often controversial subject. "Experts", books, and papers do not always concur even on basic theory or assumptions. Consequently, there are no absolute answers or exact numerical solutions. A flexible, yet conservative approach, is justified. This manual presents a procedure that will be adequate for most situations.

A portion of the text is devoted to the legal requirements and the responsibilities of the various parties involved. Construction personnel must be aware of the various legal requirements. Special restrictions are noted for excavations or trenches adjacent to railroads. A discussion on manufactured products is included.

There are many texts and publications of value other than those listed in the list of references. Use them; however, be cautious with older material. There are other satisfactory methods of approaching the engineering problem. The subject is recognized as an engineering art. The need for good judgment cannot be over emphasized. Do not lose sight of the primary objective: a safe and practical means of doing the work.

There are two major reasons why the Department considers shoring and earth retaining systems a subject apart from other temporary works such as falsework. First, an accident in a trench or excavation is more likely to have a greater potential for the maximum penalty, that is, the death of a workman. Cave-ins or shoring failures can happen suddenly, with little or no warning and with little opportunity for workers to take evasive action. Second, earth support systems design involves the complex interaction of soil types plus engineering factors that at best are controversial and highly empirical.

Trenching or shoring is generally considered temporary work. Temporary work can mean 90 days for complicated structures, but it can also be understood to mean only several days for much trenching work. The term "temporary" can be adversely affected by weather, material delays, change order work, strikes and labor disputes, and even subcontractor insolvency.

In preparing this manual through the year 1990 it has been the editors goal to cover as completely as practical some temporary earth retaining structures or systems. This manual is the result of consolidating the Office of Structure Construction experience and continued research and study by the Engineering staff. The initial edition of 1977 was well received by both the Department and the construction industry, and was distributed nationwide and to many foreign countries.

It would be impossible to acknowledge each and every individual who contributed to the development of the manual. However, recognition is due to the major contributors as follows:

James Moese, Senior Bridge Design Engineer
Adlai Coldschmidt, Technical Consulting Senior Engineering Geologist
Gary Garofalo and John Vrymoed, Senior Materials & Research Engineers, Geotechnical Engineers
Daniel Speer, Associate Materials & Research Engineer, Geotechnical Engineer
Darrell Beddard, Senior Bridge Engineer

Resident Engineers, Office of Structure Construction (OSC):

Jeff Abercrombie	Thor Larsen	Rich Thompson
Glenn Carver	Loren Newell	Dolores Valls
Ross Chittendon	Rob Stott	Steve Yee

Janie Chlubna, Word Processing Technician, OSC
Denice Davis, Office Technician, OSC
Glenda Law, Office Assistant II, OSC

Cartoons by, and included as a dedication to, G. W. Thomson.

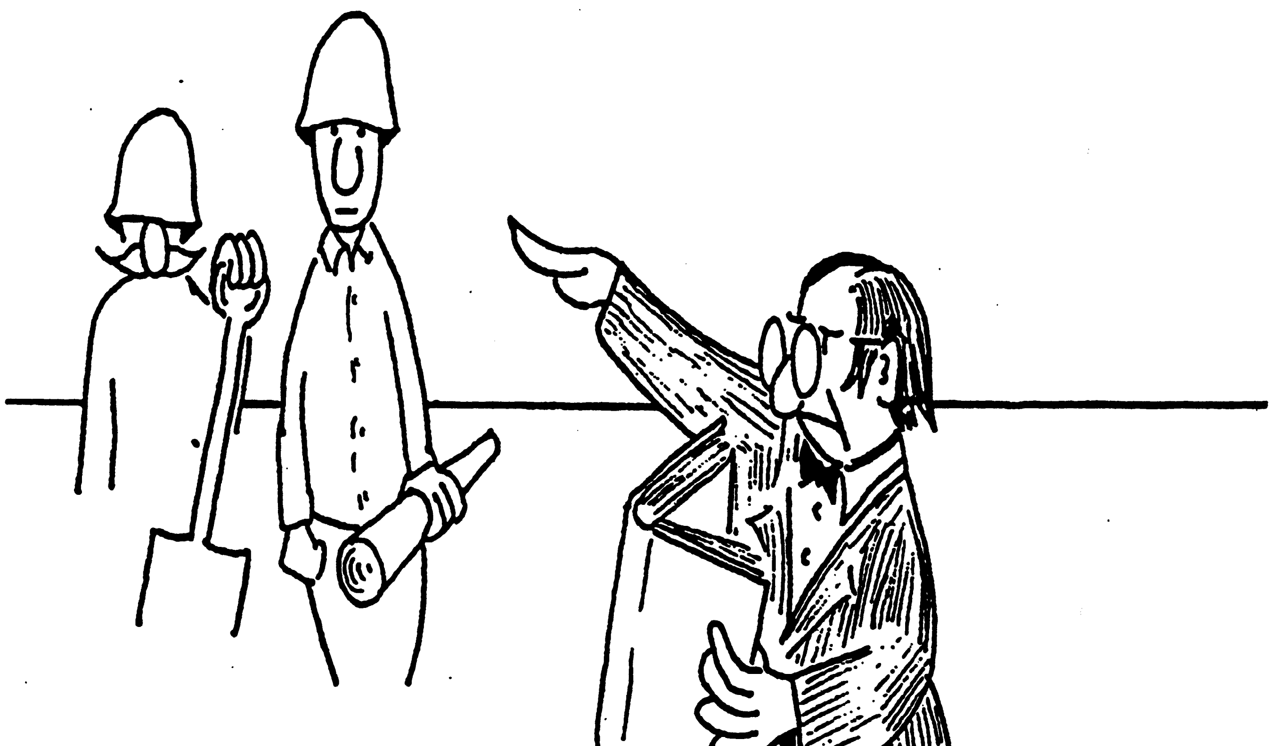
Editor in Chief: T. E. DeRosia
Editors: G. W. Thomson, Retired
J. J. Abercrombie and R. A. Stott

This manual is available to the public. Checks should be made payable to the DEPARTMENT OF TRANSPORTATION.

Copies of the manual can be obtained from:

Office of Structure Construction
Attention Manual Coordinator
Phone (916) 227-7777
Fax (916) 227-8179

LEGAL REQUIREMENTS



LEGAL REQUIREMENTS AND RESPONSIBILITIES

The State of California provides for the planning and design of permanent work to be prepared by the State (there are some exceptions with Design work prepared by a consultant), with the construction, including design of temporary work,to be performed by the Contractor.

This section of the manual deals with the responsibilities of the Contractor and the State as related to trench and excavation work during the construction phase. Under Department of Transportation specifications, the Contractor is responsible for performing the work in accordance with the contract. This responsibility includes compliance with all State and Federal Laws, and any other applicable county or municipal ordinances and regulations which in any manner affect the work.

Sections 5, 7, 8, 15, and 19 of the Standard Specifications contain references to the protection of workmen and public in trench and excavation operations. Of particular interest is Section 5-1.02A. "Trench Excavation Safety Plans":

> - Attention is directed to Section 7-l.OlE, Trench Safety." Excavation for any bench 5 feet or more in depth shall not begin until the Contractor has received approval from the Engineer, of the Contractor's detailed plan for worker protection from the hazards of caving ground during the excavation of such trench. Such plan shall be submitted at least 3 weeks before the Contractor intends to begin excavation for the trench and shall show the details of the design of shoring, bracing, sloping or other provisions to be made for worker protection during such excavation. No such plan shall allow the use of shoring, sloping or a protective system less effective than that required by the Construction Safety Orders of the Division of Occupational Safety and Health and if such plan varies from the shoring system standards established by the Construction Safety Orders, the plan shall be prepared and signed by an Engineer who is registered as a Civil or Structural Engineer in the State of California.

A Geotechnical Engineer shallprepare soils reports and supplemental geotechnical data.

Any excavation in which there is a potential hazard of cave-in or moving ground will require a protective earth retaining plant Section 5-1.02 of Standard Specifications provides that the Contractor furnish plans for temporary work to the Engineer. The Engineershall review and approvesuch plans before the start of any work. This same section 5-1.02 also provides for any earth retaining system for excavations which by definition, as given in the Construction Safety Orders, are not trenches.

Under Section 5-1.01 of the Standard Specifications the Engineer shall decide on questions which may arise as to acceptability of materials furnished and work performed. However, it is the Contractor's responsibility to properly evaluate the quality of. materials.

The State has the responsibility for administrating the contract. This means that interpretation of contract requirements, including acceptance of materials, is done by the State, not any other agency such as Cal/OSHA. Although the work must be performed in compliance with the Construction Safety Orders, there may be situations or conditions where they are not applicable or adequate. Under these circumstances the Engineer makes an interpretation and informs the Contractor accordingly of what is required.

The documents which apply to a contract are as follows:

- Department of Transportation Standard Specifications.
- Project Special Provisions.
- Contract and Standard Plans.
- California Occupational Safety and Health Standards. (Construction Safety Orders).
- California Labor Code (The Law).
- All existing and future laws, ordinances and regulations of other governmental bodies or agencies, such as railroads, having jurisdiction within the project.

LEGAL REQUIREMENTS

LABOR CODE

The California Labor Code is the document of enacted law to which all employers and employees must conform.

Division 5 'Safety in Employment' was enacted by Statute 1937 with changes in l973, 1977, and 1979. Sections 6300 to 6707 pertain to the subject of trenching and shoring.

Section 6300 establishes that the California Occupational Safety and Health Act of 1973 is enacted law. This authorizes the enforcement of effective standards for safety at work sites.

Section 6307 gives the Division of Occupational Safety and Health (DOSH) the power, jurisdiction, and supervision over every place of employment to enforce and administer laws (safety orders).

Section 6407, states that, "Every employer and every employee shall comply with occupational safety and health standards, with Section 25910 of the Health and Safety Code, and with all rules, regulations and orders pursuant to this division which are applicable to his own actions and conduct (Statute 1977 Ch. 62)".

Section 6705 establishes that for public work projects involving an estimated expenditure in excess of $25,000 for the excavation of any trench or trenches, five feet or more in depth, the Contractor must submit shoring plans to the awarding body.

Section 6706 pertains to the permit requirements for trench work.

The Labor Code is available on request from:

State of California
Department of General Services
Documents and Publications Section
P. O. Box 1015
North Highlands, CA 95660

DIVISION OF OCCUPATIONAL SAFETY AND HEALTH.

The Division of Occupational Safety and Health (DOSH) has the jurisdiction and power to enforce the legal standards for safety in every place of employment in the State. This includes work areas of governmental bodies as well as the private sector. DOSH enforces the Construction Safety Orders by means of inspections and investigations. citations are issued for violations and penalties may be assessed. In the event of an "imminent hazard", entry to the area in violation is prohibited.

DOSH may perform the following activities:

- Preparation of construction safety orders
- Policing of conformance with safety orders
- Investigation of accidents
- Compilation of Safety Statistics
- Conduct Safety Training
- Publication of Safety Order Changes
- Publication of Safety, Information (Training & Education Brochures)
- Consultation Service
- Assessment and review of citations

There are numerous geographical DOSH offices within the state. Consult Appendix A for a listing of DOSH offices.

Compliance with the Construction Safety Orders is not the same as conducting a "safety program" for employees. The objective of accident-free work is the same, but the means of implementation are quite different. The DOSH activity is essentially a policing operation in regard to ascertaining compliance with the Construction Safety Orders. An employer is required by DOSH to provide or conduct a safety program. In addition to the inspection for compliance with the Construction Safety Orders, a safety program includes education and training activities and taking positive actions in regard to conduct of the work.

DOSH will notperform engineering or inspection work for the Contractor or CalTrans.

The following paragraphs give highlights of the various portions of Title 8 of the Construction Safety Orders. Title 8 includes trench and excavation work.

The introduction to the Construction Safety Orders states that no employer shall occupy or maintain any place of employment that is not safe. This order is expanded in Section 1541 which directs that no work in or adjacent to an excavation will be performed until conditions have been examined and found to be safe by a competent person, and also that all excavation work shall have daily and other periodic inspections.

Section 1503 specifies that a permit be issued by DOSH prior to the start of any excavation work for any trench 5 feet or deeper in to which a person is required to descend. There are some exceptions, such as work performed by State forces on State R/W, and forces of utilities which are under the jurisdiction of the Public Utilities Commission. Railroads are included in the foregoing group.

It should be noted that a DOSH permit is not an approval of any shoring plan. To procure an excavation permit the Contractor makes application to DOSH. In this application the work, its location, and when it is to be done are described. DOSH may request that the Contractor furnishmore detailsforunusualwork, perhaps even a set of plans. These plans are not necessarily the detailed plans that are submitted to the Engineer for review and approval.

The objective of a DOSH Permit is to put DOSH on notice that potentially hazardous work is scheduled at a specific location. DOSH may then arrange to inspect the work.

Permits are issued by DOSH for various conditions. A single permit can cover work of a similar nature on different contracts, It can be for a specifictype of work within a DOSH regional area. In this case, the permit will have a time limit and the user is obligated to inform the appropriate DOSH office of his schedule for work covered by the permit. A copy of the permit is to be posted at the work site. It is the responsibility of the Engineer to ascertain that the Contractor has secured a proper permit before permitting any trenching or excavation work to begin.

Excavations are defined to include trenches. The Construction Safety Orders in Section 1540 of Article 6 define a trench as any excavation in which the depth generally exceeds the average width and the bottom width is not greater than 15 feet. Excavations which are more than 15 feet wide at the bottom, or shafts, tunnels, and mines are excavations by DOSH definition. However, this does not mean that an excavation permit and shoring plans are not required.' Excavations which do not fall into the trench category may require a permit because of their hazardous nature. Box culvert and bridge foundations are examples. Bridge abutments will present a trench condition at the time that back wall form panels are erected. The solution is to either provide a shoring system to retain the earth, or cut the slope back at an acceptable angle.

State Statutes

Section 137.6 of Article 3 in Chapter 1 of Division 1 of the Statutes, requires that the review and approval of Contractor's plans for temporary structures in connection with the construction of State Highways shall be done by a Registered Professional Engineer.

> "137.6. The design of, the drafting of specifications for, and the inspection and approval of state highway structures shall be by civil engineers licensed pursuant to the Professional Engineers Act (Chapter 7 (commencing with Section 6700), Division 3, Business and Professions Code)."
>
> "The approval of plans for, and the inspection and approval of, temporary structures erected by contractors in connection with the construction of state highway structures shall also be by such licensed civil engineers."

This means that the Engineer has the responsibility to see that appropriate plans are submitted and properly reviewed for work to be performed within State right of way.

FEDERAL HIGHWAY ADMINISTRATION (FWHA)

Section 6 of the Contract Special Provisions contains the Federal requirements for the project. These include provisions for safety and accident prevention. The Contractor is required to comply with all applicable Federal, state, and local laws governing safety, health, and sanitation. Conformancewith current DOSH standards will satisfy Federal Requirements, including FedOSHA.

RAILROAD RELATIONS AND REQUIREMENTS

Section 13 of the Contract Special Provisions for the project will contain the railroad agreement with the State. These provisions require that the Contractor shall cooperate with the railroad where work is over, under, or adjacent to tracks, or within railroad property, and that all rules and regulations of the railroad concerned shall be complied with. It also requires that the Contractor and subcontractors have approved Railroad Insurance and submit plans for all temporary works on railroad property to the railroad for review and approval.

The Department of Transportation has established an administrative procedure for handling shoring plans which involve railroads as follows:

- Contractor submits shoring plans to the Engineer (Project Resident Engineer). Railroads require that a plan be prepared even if proposed system is in accordance with DOSH Details. Shoring is required for excavations less than 5 foot in depth if specific-railroad criteria calls for it (railroads differ in requirements). The drawing must include a trench cross section and-a plan view giving minimum clearances relative to railroad track. Provisions for walkways if required, are to be submitted with the plans. Plans are to be prepared by a Professional Engineer with each sheet of the plans signed.

- Some railroads have their own specifications for shoring. The railroad specifications will be used in conjunction with DOT Policy and the DOSH Construction Safety Orders. The most restrictive of these will apply. The reader is referred to Appendix C of this manual for railroad requirements.

- The Resident Engineer reviews the plan: When satisfied, the Resident Engineer will forward the plan with the Contractor's and the Engineer's calculations to the Office of Structure Construction in Sacramento (OSC).

- In Sacramento, OSC will make a supplementary review. Then if the plans and calculations are satisfactory they will be forwarded to the railroad concerned.

- The railroad reviews and approves the shoring plans, and notifies OSC, Sacramento.

- OSC notifies the Resident Engineer of approval.

- The Resident Engineer approves the plans and notifies the Contractor.

Section 19-1.02, "Preservation of Property" of the Standard Specifications includes a provision stipulating that shoring plans be submitted at least 8 weeks before the Contractor intends to begin any excavation requiring shoring.

Note that the railroad deals directly with the Sacramento Office of OSC, not with the Engineer on the job site. Adequate time should be allowed for the review procedure. The railroad may take up to 6 weeks for review from the time that they receive the plans from Sacramento. The proper time to alert the Contractor to procedure and time needed is at the pre-job conference.

Normally the OSC Engineer on the job will handle the review and approval of shoring plans which involve railroads. When there is no Project Office of Structure Construction Representative or Resident Engineer, the District may request technical assistance from the Office of Structure Construction Area Construction Engineer, or from the Office of Structure Construction in Sacramento.

SHORING PLANS

Section 5-1.02A of the Standard Specifications requires that a Contractor submit a shoring plan for any trench 5 feet or deeper to the Engineer for his review and approval. Such plans are to be submitted in a timely manner as Specified in Section 5-1.02A of the Standard Specifications (or as required by the contract Special Provisions) before the Contractor intends to begin excavating. No work will begin until the shoring plans are approved by the Engineer.

If the Contractor elects to use the Details in the Construction Safety Orders, it is not required that a Professional Engineer prepare the plan. However, a shoring plan is still required. This plan can be a letter to the Engineer containing the information outlined in, "Shoring Plan Submittal," on page 2-2 of this manual.

The Details in the Construction Safety Orders consist of sloping, or tables of minimum member sizes for timber and aluminum hydraulic shoring with member spacings related to the three general types of soil, along with various restrictions on use of materials and construction methods. The Engineer is cautioned that conditions may be such that the Details will not directly apply - for example when a surcharge load exceeds the minimum construction surcharge of 72 psf. In such a case, an 'engineered' system is required. The proposed plan must provide a system at least as effective as the DOSH Details, and the plan must be prepared and signed by a California registered professional engineer. This plan would include the following items in addition to information listed for a Standard Detail plan:

- An engineering drawing showing sizes, spacing, connections, etc. of materials.
- Appropriate additional soils data.
- Supporting data such as design calculations or material tests.

The Engineer will make a structural review of any plan which deviates from the DOSH Details.

In general practice, engineered drawings will be accompanied by the engineer's calculations. If railroads are involved, a minimum of three sets of calculations and seven sets of plans should be submitted. The railroads require a minimum of one set of calculations each from the designer and reviewer and four sets of shoring plans. One additional complete set of calculations and drawings will be needed for the OSC Sacramento Office.

TECHNICAL DATA

This manual contains a presentation of much of the technical engineering information which can be used by the Engineer in making a review of shoring plans.

The design or engineering analysis, of a shoring system is accomplished in the following sequence.

- The soil or earth that is to be retained and its engineering properties are determined.
- Soil properties are then used in geotechnical mechanics or procedures to determine the earth pressure force acting on the shoring system. An equivalent fluid, Kw, may be determined.
- The design lateral force is then distributed, in the form of a pressure diagram. The distribution, or shape, of the diagram is a function of type of shoring system and the soil interaction with the system.
- Lateral loads due to surcharges and from sources other than basic soil pressure (e.g., ground water) are determined and may be combined with the basic soil pressure diagram, Modified for practicability, the resulting lateral pressures become the design, lateral pressure diagram.
- The design lateral pressure diagram is applied to the system, and a structural analysis is made. Again, there is a range from simplified to refined or complex procedures that can be used.

Keep in mind a proper balance of engineering effort. If soils data is not detailed or is not available, it is not proper to use complex or sophisticated distribution theories. With good soils data it is satisfactory to first use simplified analysis procedures which is conservative; then if the system appears inadequate, use a more refined procedure.

The engineering analysis is a progressive procedure dependent upon complexity or sophistication. It is a function of size of project or how unusual or unique it is. A simplified analysis procedure can be used for the majority of trench and shoring projects. For complex systems, the Engineer may be presented with theories which are not discussed in this manual. The Engineer should be prepared to do some research. Do not reject a procedure just because it is not covered in this manual. Request any design information or copies of text material needed to analyze the calculations. The Geotechnical Engineering branch of the Transportation Materials and Research Laboratory is available for consultation for major problems. This manual presents very basic engineering procedures.

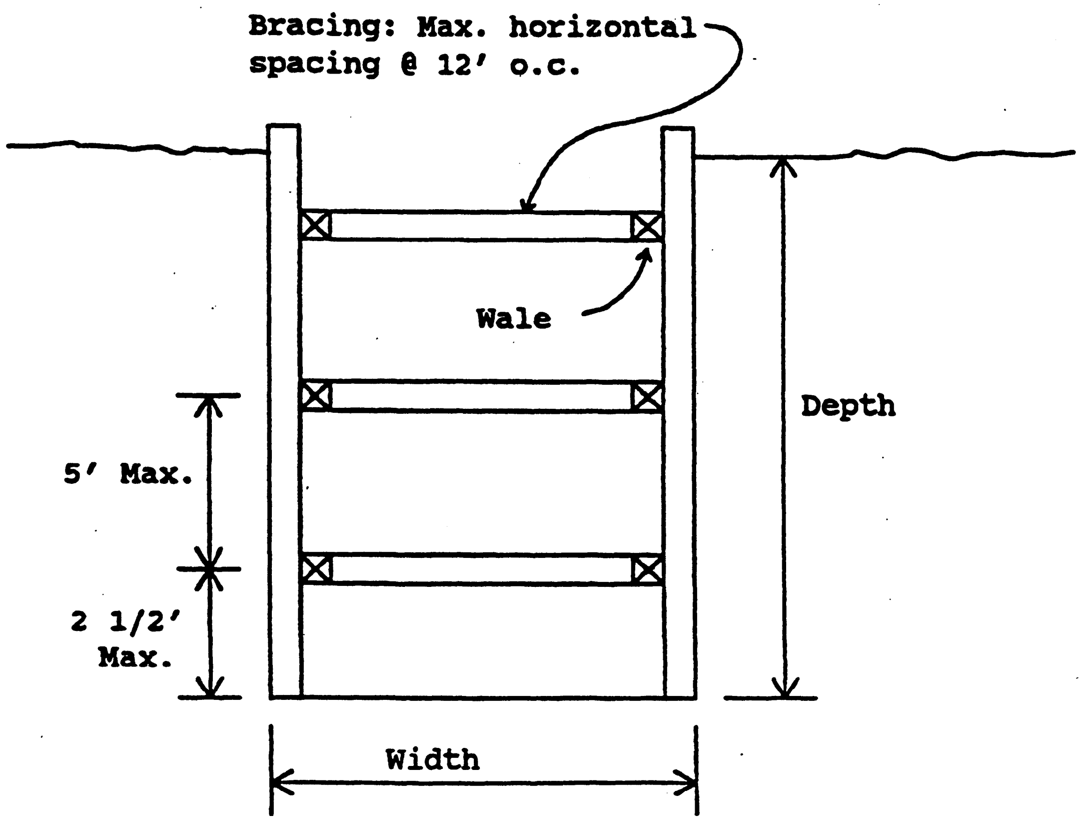
It is recognized that the construction phase is of equal importance. Construction activities include workmanship, inspection, and taking appropriate timely action with regard to changing conditions. The reader is referred to Chapter 12, on Construction Considerations, for more information.

When shoring plans are being reviewed, the following procedure is recommended. Perform an initial review of the shoring in conformance with the criteria in the trenching and shoring manual. If the review indicates discrepancies in the design, it will be necessary to review the criteria used by the designer. As with any set of plans or working drawings, if the submitted material is incomplete, it will be necessary to have the Contractor obtain a more thorough description of design procedures, assumptions, additional calculations, or copies of text confirming design computations. Note, however, there is no requirement that the design must be in conformance with the criteria outlined in this manual. In case of dispute, telephone Sacramento.

Many shoring designs completed by Consulting and/or Structural Engineering firms contain complete soils data with shoring recommendations. The Consulting or Structural Engineering firms may modify or conform to the soil data recommendations. Shoring designs by such firms will most generally be less conservative than if the design were in conformance with this manual; consequently the shoring may need to be reviewed in the manner in which it was designed.

D O S H

CONSTRUCTION SAFETY ORDERS



CALIFORNIA DIVISION OF OCCUPATIONAL SAFETY AND HEALTH (DOSH)

Cal/OSHA reports that more construction deaths occur in trenches than in any other form of construction work. This is not the whole story since a number of trench and excavation failures go unreported. It is evident from this that more attention needs to be paid to the planning, construction, monitoring, and supervisory aspects of excavations and trenching.

The California Occupational Safety and Health Program (Cal/OSHA), effective September 25, 1991, adopted the Federal OSHA safety regulations pertaining to protection of workmen in excavations. The information in this chapter and in Appendix A of this manual is current as of February, 1992. It will be the responsibility of the reader to determine up-to-date applicable requirements.

This chapter contains outlines of major portions of the adopted Safety Orders that pertain to safety in conjunction with excavations. Major considerations, or requirements of the Safety Orders, in numerical order of the Sections, are briefly outlined on the following pages. Following the brief outlining is a condensed outline of most of the Safety Orders pertaining to the subject of excavations. The text of most Cal/OSHA excavation requirements may be found in Appendix A of this manual. Appendix A text includes Construction Safety Order Sections 1504, 1539, 1540, 1541, 1541.1 (including appendices A - F), and Sections 1542-1547.

Excavations 20' Deep Or Less: Sections 1504 and Sections 1539 through 1547 of the Construction Safety Orders contain the excavation and shoring requirements of the California Division of Occupational Safety and Health. The Safety Orders provide for a variety of excavation plans for workman protection in excavations. For excavations less than 20 feet in depth the Contractor may use sloping or benching of the soil, tables for timber or aluminum hydraulic shoring, shields, or the shoring may be designed by a California registered professional engineer.

Excavations Over 20' Deep - Deviations: A California registered professional engineer's design will be required for excavations greater than 20 feet in depth, when deviations from the sloping criteria are to be used, when there will be deviations not covered in the Safety Orders from the timber or aluminum hydraulic shoring tables, when shields are, to be used in a manner not recommended or approved by the manufacturer, when surcharges must be accounted for or when alternate designs are to be used. The designing engineer may base his design-on manufacturer's information, on a variety of tables of charts, use of proprietary systems, on soils information furnished by a competent person, and in accordance with registered professional engineering practice.

Maintain Design Plan At The Jobsite: The Safety Orders provide that at least one copy of the tabulated data, manufacturer's data or engineer's design is to be maintained at the jobsite during construction of the protective system. The Safety Orders specify that the identity of the registered professional engineer approving tabulated or manufacturer's data be included in the information maintained at the jobsite. The registered professional engineer approving the data refers to the engineer responsible for the design of the protective system.

Registered Professional Engineer: For work in California the design registered professional engineer must be registered in California pursuant to Section 137.6 of the Statutes relating to the California Department of Transportation.

Competent Person: The Safety Orders in Section 1504 of Article 2 defines a competent person as, "One who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them."

Surcharges: The Safety Order Figures and Tables provide for a minimum surcharge equivalent to an additional soil height of 2 feet. The minimum surcharge may be considered to represent a 2 feet high soil embankment, small equipment, material storage, or other small loadings adjacent to the excavation. No provision is made for nearby traffic, adjacent structure loadings, or for dynamic loadings.

Shoring Plan Submittal: The Contractor may submit a shoring plan using Cal/OSHA sloping figures, or tabular data, in the form of a letter stating which portions of the Safety Orders are to apply to the plan. The letter should list location of the work, the limits of the work, the times the work is to start and be in progress, sequence, the applicable Cal/OSHA figures or tables, any other information which will pertain to the progress or complexity of the work, who will be in charge of the work, and who will be the designated competent person responsible for safety. If the Contractor elects to use the shoring details in the Safety Orders it is not necessary to have the shoring plan prepared by a professional engineer; and the reviewing engineer does not have to do a structural analysis. However, the reviewing engineer must ascertain that the Contractor does the work in accordance with the Safety Orders and that the site conditions are such that the shoring plan is appropriate for the soil conditions encountered.

SOME IMPORTANT DEFINITIONS

A lot of information about the requirements in the Safety Orders can be condensed by describing or citing primary sections. A few important definitions are included here, but the reader is directed to a more complete text of the Construction Safety Orders included in Appendix A of this manual.

From Section 1504, "Excavation, Trenches, Earthwork," *of* Article 2:

Geotechnical Specialist (GTS): "A person registered by the State as a Certified Engineering Geologist, or a Registered Civil Engineer trained in soil mechanics, or an engineering geologist or civil engineer with a minimum of 3 years applicable experience working under the direct supervision of either a Certified Engineering Geologist or Registered Civil Engineer".

From Section 1540, "Excavations," *of Article* 6

Accepted Engineering Practices: Those requirements which are compatible with standards of practice required by a registered professional engineer.

Excavation: All excavations made in the earth's surface. Any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal. Excavations are defined to include trenches.

Protective System: A method of protecting employees from cave-ins, from material that could fall or roll from an excavation face or into an excavation, or from collapse of adjacent structures. Protective systems include support systems, sloping and benching systems, shield systems, and other systems that provide the necessary protection.

Registered Professional Engineer: A person who is registered as a professional engineer in the state where the work is to be performed. However, a professional engineer, registered in any state is deemed to be a "registered professional engineer" within the meaning of this standard when approving designs for "manufactured protective systems" or "tabulated data" to be used in interstate commerce. (The interstate commerce provision affords relief for utilities when crossing State boundaries).

Shield (Shield System): A structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields may be either pre-manufactured or job-built.

Shield (Shield System: A structure such as a metal hydraulic, mechanical, or timber shoring system that supports the side of an excavation and which is designed to prevent cave-ins.

Sloping (Sloping System): A method of protecting workmen from cave-ins by sloping the sides of the excavation away from the excavation. The slope angle varies with the soil type, surcharges, and weather.

Tabulated Data: Tables and charts approved by 6 registered professional engineer and used to design and construct a protective system.

Trench: A narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is not greater than 15 feet. If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 feet or less, (measured at the bottom of the excavation), the excavation is also considered to be a trench.

SOME IMPORTANT REQUIREMENTS

A few important considerations from the General Requirements section of the Construction Safety Orders are listed here for easy reference. A complete text of Section 1541 referred to below is included at Appendix A of this manual.

From Section 1541, "General Requirements," *of Article 6:*

Underground utilities must be located prior to excavation. The Contractor should notify Underground Alert or other appropriate Regional Notification Centers a minimum of 2 working days prior to start of work. Excavation in the vicinity of underground utilities must be undertaken in a careful manner while supporting and protecting the utilities.

Egress provisions which may include ladders, ramps, stairways, or other means shall be provided for excavations over 4 feet in depth so that no more than 25 feet of lateral travel will be needed to exit trench excavations.

Adequate protection from hazardous atmospheres must be provided.

Employees shall be protected from the hazards of accumulating water, from loose or falling debris, or from potentially unstable adjacent structures.

Daily inspections, inspections after rain storms and as otherwise required for hazardous conditions, are to be made by a competent person. Inspections must be conducted prior to the start of work and as needed throughout the shift. The competent person will need to check for potential cave-ins, indications of failure of the protective system, and for hazardous atmospheres. When the competent person finds a hazardous situation he shall have the endangered employees removed from the area until the necessary. precautions have been made to insure their safety.

Adequate barrier physical protection is to be provided at all remotely located excavations. All wells, pits, shafts, etc., shall be barricaded or covered. Upon completion of exploration and other similar operations, temporary shafts etc., shall be backfilled.

PROTECTIVE SYSTEM SELECTION

From Section 1541.1, "Requirements For Protective Systems"

Section 1541.1 of Article 6 of the Safety Orders covers almost all of the requirements that must be considered in selecting or reviewing a particular type of shoring system. The text of this section contains general information and considerations about the various selections which may be made for shoring systems. This section describes the various shoring systems which can be used with and without the services of a registered professional engineer. Additional information about the various shoring systems may be found in Appendix A through Appendix F of Section 1541.1 (See Appendix A of this manual).

The Contractor will use this portion of the Safety Orders to select a particular type of shoring system best suited to fit the soil conditions and the jobsite situation. The services of a registered professional engineer will not be required for a number of the shoring options available to the Contractor.

An overview of the major portions of Section 1541.1 is outlined below. The complete text of Construction Safety Order Section 1541.1 is included in Appendix A of this manual.

The design of a protective system for workmen in an excavation may be selected from one of the possible options listed below:

Stable rock - No shoring needed.

Excavation less than 5 feet deep - No shoring needed.

Sloping or benching:

- Slope 1 l/2 : 1 as for Type C soil.
  Steeper slopes may be used for short term (1 day).
- Slope using Table B-l or Figure B-l of Appendix B.
  Slopes dependent on soil type - see Appendix A.
- Per tables or charts identified by a California registered professional engineer.
- Design by a California registered professional engineer.

Design of support systems, shield, or other systems:

- Design in accordance with Appendix A, or C - F.
  Appendix A - Soil classification.
  Appendix C - Timber shoring tables.
  Appendix D - Hydraulic shoring tables.
  Appendix E - Alternatives to timber shoring.
  Appendix F - Flow chart guides to system selection.

- Design using Manufacturer's data (shields for example)
  Data includes specifications, limitations, and/or other tabulated data (Tables or Charts).

- Design using other tabulated data (Tables or Charts),
  Identified by a California registered professional engineer approving the data. [Approving engineer implies the California professional engineer designing or submitting the shoring plan.]

- Design by a registered professional engineer.
  Identified by a California registered professional engineer approving the plan. [Approving engineer implies the California professional engineer designing or submitting the shoring plan.]

Shoring system designs (including manufacturer's data) other than those selected directly from tables in Appendix A - F will need to be posted at the jobsite during construction of the protective system.

Damaged materials or equipment will need to be reevaluated for use by a competent person or by a registered professional engineer before being put back into use.

Individual members of support systems may not be subjected to loads exceeding those which they were designed to withstand.

Excavation of material to a level no greater than 2 feet below the bottom of the members of a support system shall be permitted, but only if the system is designed to resist the forces calculated for the full depth of the excavation, and no loss of soil is possible.

Shields systems are not be subjected to loads exceeding those which the system was designed to withstand.

SOIL CLASSIFICATION

APPENDIX A TO SECTION 1541.1

Appendix A to Section 1541.1 of Article 6 (See Appendix A of this manual) contains the soil classification information which may be used for the proper selection of a shoring system. This Section describes when this soils classification information may be used, defines soil, defines the soil types (A, B, or C), covers the basis of soil classification, who can classify soil and how it may be done with visual or manual tests, and field testing methods to determine soil type.

A competent person, or a testing lab, may make determinations by at least one visual and at least one manual test to classify rock or soil for the proper selection of, or for the design of, a shoring system. Classification of the soil is necessary to determine the effective active soil pressures that the shoring system may be subjected to. The tables for the selection of sloping, timber shoring, or aluminum hydraulic shoring, are based on one of three types of soil (A, B, or C).

The three soil types in the Safety Orders are described below:

Type A: Cohesive soil with unconfined compressive strength of 1.5 tsf or greater.

Examples of this soil type are: clay, silty clay, sandy clay, clay loam, silty clay loam, sandy clay loam, cemented soils like caliche or hardpan.

No soil-is Type A if:

- The soil is fissured.
- Vibratory or dynamic loads will be present.
- The soil has been previously disturbed.
- Sloped (4 : 1 or greater) layers dip into the excavation.
- Other factors preclude Type A classification.

Type B: Cohesive soil with unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf <u>or:</u>

Granular cohesionless soils including: angular gravel, silt, silty loam, sandy loam, or maybe silty clay loam and sandy clay loam, *or:*

Previously disturbed soils not classified as Type C *or:*

Soil that meets the requirements of type A but is fissured or subject to vibration, *or:*

Dry rock that is not stable, <u>or</u>:

Type B soil that has sloped (4 : 1 or less.) layers that dip towards the excavation.

Type C: Cohesive soil with unconfined compressive strength of 0.5 tsf or less <u>or</u>:

Granular soil including gravel, sand, and loamy sand, <u>or</u>:

Submerged soil, or from which water is freely seeping, <u>*or*</u>:

Submerged rock that is not stable, <u>or</u>:

Material sloped towards the excavation 4 : l or steeper in a layered system.

Tables in the Safety Orders for timber or for aluminum hydraulic shoring consider the effective lateral pressures for a depth H due to the three different soil type's as follows:

Type A: $P_A = 25H + 72$psf (2 Ft. Surcharge)
Type B: $P_A = 45H + 72$psf (2 Ft. Surcharge)
Type C: $P_A = 80H + 72$psf (2 Ft. Surcharge)

Manual testing of soils includes tests for plasticity, tests for dry strength, thumb penetration, and the use of a pocket penetrometer or hand operated shearvane. Samples of soil can be dried to determine relative cohesive content. A few of these tests may be used to determine compressive strength, the other tests may be used to determine relative cohesive properties of the soil. The test procedures are outlined in the complete text of Appendix A to Section 1541.1 (See Appendix A of this manual). Note that expansive clays are not mentioned and may need special consideration.

SLOPING OR BENCHING SYSTEMS

APPENDIX B TO SECTION 1541.1

Appendix B of Section 1541.1 of Article 6 (See Appendix A of this manual) contains all of the sloping or benching options in picture form for excavations less than 20 feet in depth allowed by the Safety Orders. Alternate configurations may be designed by a registered professional engineer.

Slopes may be laid back in conformance with the figures in Appendix B to-Section 1541.1 providing there is no sign of distress and surcharge loads will not be a factor. Signs of distress include:

caving-in-of the soil, development of fissures, subsidence, bulging or heaving at the bottom of the excavation, or spalling or raveling at the face of the excavation.

When there is a sign of distress the slope is to be laid back at least l/2 horizontal to 1 vertical less than the maximum allowable. slope.

Allowable slopes shall be reduced as determined by a competent person when surcharge loads, other than from structures, will be present.

When surcharge loads from structures are present underpinning or bracing will be required, or the structure must be on stable rock, or a registered professional engineer must determine that the excavation work will not pose a hazard to employees.

Table B-1 of Appendix B to Section 1541.1 lists the following maximum slopes for the various soil types:

Stable Rock:	**Vertical**
Type A:	**3/4 : 1**
Type B:	**1 : 1**
Type C:	**1 1/2 : 1**

Exceptions:

Type A soil: 1/2 : 1 Slope permitted for up to 12 feet in depth for short term duration (24 hours or less).

Excavations over 20 feet in depth shall be designed by a registered professional engineer.

<u>Type A Soil Slopping and/or Benching Options:*</u>

3/4 : 1 slopes.
l/2 : 1 slopes (Short term and 12 feet or less).
3/4 : 1 slope with 4 foot high single bench at bottom.
3/4 : 1 slope replaced with 4' high aid 5' high benches.
3/4 : 1 slope above 3 l/2 foot high bench (8, max. depth).
1 : 1 slope above 3 l/2 foot high bench (12, max. depth).
3/4 : 1 slope above supported or shielded system.

A single or lower bench may be in front of the slope line, but all higher benches must be behind the slope line.

* See diagrams of Appendix B to Section 1541.1 in Appendix A.

Type B Soil Sloping and/or Benching Actions:*

1 : 1 slopes.
1 : 1 slopes above 4' high single bench.
1 : 1 slopes replaced with 4' high benches (Cohesive soil).
1 : 1 slopes above supported or shielded system.

A single or lower bench may be in front of the slope line, but all higher benches must be behind the slope line.

*** See diagrams of Appendix B to Section 1541.1 in Appendix A.**

Type C Soil Sloping and/or Benching Options:*

1 l/2 : 1 slopes.
1 1/2 : 1 slopes above supported or shielded system.

*** See diagrams of Appendix B to Section 1541.1 in Appendix A.**

Layered Soils Sloping:*

Type B over Type A: Slope Type B 1 : 1 and Type A 3/4 : 1.
Type C over Type A: Slope Type C 1 l/2:1 and Type A 3/4 : 1.
Type C over Type B: Slope Type C 1 l/2 : 1 and Type B 1 : 1.
Type A over Type B: Slope both 1 : 1.
Type A over Type C: Slope both 1 l/2 : 1.
Type B over Type C: Slope both 1 1/2 : 1.

*** See diagrams of Appendix B to Section 1541.1 in Appendix A.**

TIMBER SHORING FOR TRENCHES

APPENDIX C TO SECTION 1541.1

Appendix C to Section 1541.1 (See Appendix A of this manual) contains information and tables that the Contractor may utilize to shore trenches less than 20 feet deep with rough or finish timbers in any of the three types of soil. Tables C-1.1 through C-l.3 may be used for minimum rough (actual) size timbers having a minimum f_b of 850 psi, and Tables C-2.1 through C-2.3 are for finished (S4S) timbers having a minimum f_b of 1500 psi. There is one table for each soil type for each of the timber grading sizes,

Summaries of notes which are meant to accompany the tables are listed below:

When conditions are saturated use tight sheeting (tight sheeting refers to 3 " rough tongue and groove timbers, steel sheet piling or similar to resist imposed lateral loads including water). Close spacing refers to placing planks side-by-side as close as possible.

All spacings indicated are center to center.

Wales are to be installed with greatest dimension horizontal.

If the vertical distance from the center of the lowest crossbrace to the bottom of the trench is to exceed 2.5 feet uprights are to be firmly imbedded, or a mudsill is to be used. A mudsill is a waler placed at the bottom of the trench.

Maximum distance from lower brace to bottom of trench:
36 inches for imbedded sheeting.
42 inches when mudsills are used.

Trench jacks may be used in place of, or in combination with timber struts.

Upper crossbrace (strut) vertical spacing from top of excavation is not to exceed one-half tabulated vertical crossbrace spacing.

When any of the following conditions will exist the tables will not be adequate:

When loads imposed by structures of stored materials adjacent to the trench will exceed the load from a 2 foot surcharge. Adjacent means within a horizontal distance equal to the depth of the trench.

When vertical loads on the center of crossbraces exceed 200 pounds.

When surcharge loads from equipment weighing over 20,000 pounds are present.

When only the lower portion of a trench is shored and the remaining portion is slopped or benched unless:

The sloping portion is sloped less than 3 : 1, or
The shoring is selected for full depth excavation.

Appendix C to Section 1541.1 (See Appendix A of this manual) contains Tables C-1.1 through C-1.3, Tables C-2.1 through C-2.3, and 4 example problems demonstrating selection of shoring from the tables.

ALUMINUM HYDRAULIC SHORING FOR TRENCHES

APPENDIX D TO SECTION 1541.1

Appendix D to Section 1541.1 (See Appendix A of this manual) contains, typical installation diagrams, tables, and information for the use of aluminum hydraulic shoring in trenches less than 20 feet deep. Tables D-1.1 and D-1.2 are for vertical shores in Type A and B soils. Tables D-1.3 and D-1.4 are for horizontal waler systems in Type B and Type C soils. Type B soils may require sheeting, whereas Type C soils always require sheeting.

The tables consider two cylinder sizes with minimum safe working capacities as follows: 2 inch inside diameter with 18,000 pounds axial compressive load at maximum extension, or 3 inch inside diameter with 30,000 axial compressive load at extensions as recommended by the product manufacturer.

When any of the following conditions exist the tabular data will not be valid:

> When vertical loads exceeding 100 pounds will be imposed on the center of hydraulic cylinders.
>
> When surcharge loads are present from equipment weighing in excess of 20,000 pounds.
>
> When only the lower portion of the trench is shored and the upper portion is sloped or benched steeper than 3 : 1; unless the shoring is selected for a trench full depth from the upper hinge point to the bottom of the trench.

Footnotes for the aluminum hydraulic shoring will be found in Section (g) of Appendix D to Section 1541.1 immediately preceding the Figures (See Appendix A of this manual).

Minimum thickness plywood of 1 l/8" (or 3/4" thick 14 ply Finply) may be used in conjunction with aluminum hydraulic shoring to prevent raveling, but may not be used as structural members.

Alternate designs and designs for excavations over 20 feet deep may be submitted by a California registered professional engineer.

SHIELD SYSTEMS

APPENDIX E TO SECTION 1541.1

Appendix E to Section 1541.1 (See Appendix A of this manual) contains a few diagrams of manufactured trench shields.

The reviewing engineer should be aware that manufacturers will normally furnish engineering data to a supplier, who in turn will furnish the data to the Contractor. A Contractor may submit a sales brochure as a shoring plan for approval. A brochure is not a plan; it generally will represent the manufacturer's data (the strength or capacity of the product). A shoring plan for specific use of the shield must be prepared. The engineer can determine forces, including surcharges, that are to be resisted, and then make comparisons with manufacturer's data, or with the submitting engineer's computations which define the capacity of the shoring system.

A number of the trench shoring and shield manufacturers/suppliers belong to the TRENCH AND SHIELDING ASSOCIATION. The Association has published a manual covering product use and safety with respect to trench and shoring work. Member listing and other information may be obtained from :

TRENCH SHORING AND SHIELDING ASSOCIATION
25 North Broadway
Tarrytown, N. Y. 10591 Phone (914) 332-0040

Some members of the Trench Shoring and Shielding Association are listed below:

AIRSHORE INTERNATIONAL CORPORATION, 16211-84 Avenue, Surrey, B.C. Canada V3S 2P3.
ALLIED (TREN-SHORE), 5800 Harper Road, Solon, OH, 44139. Phone (216) 248-2600. Offices in Escondido, Pittsburgh, and L.A.
EFFICIENCY PRODUCTION,INC., P.O. Box 24126, Lansing, MI. 48909
GRISWOLD MACHINE & ENGINEERING, INC. (GME) Highway M-60, Union City, MI, 49094, Phone l-800-248-2054.
SAFE-T-SHORE, 3102 S. Roosevelt, Tempe AZ. 85282.
SPEED SHORE CORPORATION P.O. Box 262591, Houston TX 77207. Phone (713) 943-0750.
PLANK PLATES, 8979 Elk Grove-Florin Road, Elk Grove, CA 95624. Phone (916) 686-5151 or (916) 486-1307.
TRENCH PLATE RENTAL CO. Phone: Sacramento l-800-548-0688, Northern CA l-800-321-5500, Southern CA l-800-8231-4478.

MANUFACTURED PRODUCTS

Manufactured trench shoring and worker protection products include screw jacks, hydraulic shores, screw or hydraulic operated frames, work shields and other devices used to shore a trench and/or protect workmen.

If the Contractor's shoring or worker protection plan includes a manufactured product, the Engineer should not hesitate to request from the Contractor the manufacturer's recommendations if they are needed to verify the safe load capacity of the product.

The maximum loading which may be applied to a manufactured product shall not exceed the capacities as given by the manufacturer. These are usually shown in a catalog or brochure published by the manufacturer, or in the form of a letter from the manufacturer pertaining to the use of his product for specific job conditions. This statement may be shown on a working drawing or included in a letter. To be acceptable it must be signed by the manufacturer; not the Contractor. When professional engineering data accompany manufactured products that data may be used with minimum supplemental review.

Be aware that some manufacturers catalogs do not always present enough engineering data; they are sales brochures. Also, make sure of the conditions which apply to the data that is presented. An example of this is often encountered in 'capacity ratings' for shoring products. It may be necessary to search further to ascertain that such were established for the minimum equivalent lateral earth fluid pressure loads permitted by the Safety Orders. Request that the Contractor furnish additional engineering data (from the manufacturer if possible).

The maximum allowable safe working load as recommended by the manufacturer will be based on the use of new or undamaged used material. If the product or its components are not in good condition it must be determined if the product can function as intended, or if the safe working loads should be reduced. It is the responsibility of the Contractor to furnish proof of loading capacity.

In the case of manufactured products which cannot be found in any catalog, and the manufacturer is unknown or unable to recommend a safe working load, the Engineer should require a load test to establish the safe load capacity of the product as it is to be used.

A load test, if possible, should be conducted to failure or to near failure to determine the maximum capacity of the product. The safe working load may then be assumed to be one-half of the ultimate test loading. We accept a minimum value of 2 for a safety factor by rationalizing that there is greater quality control for a manufactured product relative to other shoring materials (such as timber). Load tests witnessed by the Engineer should be documented in the project records and a copy submitted to Sacramento with the approved shoring plans.

Materials must be properly identified when calculations are to be made. made. This is very important when analyzing aluminum members as there are many different alloys.

ALTERNATE DESIGN CONSIDERATIONS

A minimum live load surcharge of 72 pounds per square foot lateral pressure shall be included in all shoring designs. Any additional surcharge loads such as from equipment, buildings, etc., should also be included in the shoring design. Refer to allowable working stresses in Chapter 12. Alternate allowable stresses may be used provided that is can be satisfactorily shown that these values conform to acceptable engineering practice.

INFORMATION ABOUT TEXT FORMATTING IN THE CONSTRUCTION SAFETY ORDERS

In the Construction Safety Orders all subtopics are usually indented the same amount only on the first line of type. The subjects and subheadings format generally conforms to the following example:

Article No. Major Heading

Section Number. Heading.

(a) Lower case letter used for first subtopic.
(1) Number used for subtopic to lower case letter.
(A) Upper case letter used for subtopic to number.
1. Number used-for subtopic to uppercase letter.

Another Heading.

SUMMARY OF EXCAVATION SECTIONS IN THE CONSTRUCTION SAFETY ORDERS

Most of the subjects and first subtopics of the Cal/OSHA Construction Safety Orders related to trenching and shoring are outlined on the following pages for easy reference (Additional information about the subject may be included in brackets]. A complete text of the same Construction Safety Orders is included in Appendix A of this manual.

SUMMARY OF EXCAVATION SECTIONS IN THE CONSTRUCTION SAFETY ORDERS

Article 2. Definitions

1504. Definitions.

Competent Person: One who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them.

Excavations, Trenches, Earthwork.

(A) Bank.
(B) Exploration Shaft.
(C) Geotechnical Specialist (GTS),
(D) Hard Compact (as it applies to Section 1542).
(E) Lagging.
(F) Running Soil (as it applies to Section 1542).
(G) Shaft.

Article 6. Excavations

1539. Permits.

For regulations relating to Permits for excavations and trenches refer to Article 2, Section 341.

1540. Excavations.

(a) Scope and application. [Open excavations, trenches]
(b) Definitions applicable to this article.
Accepted engineering practices.
Aluminum hydraulic shoring.
Bell-bottom pier hole.
Benching (Benching system).
Cave-in.
Crossbraces
Excavation.
Face or sides.
Failure.
Hazardous atmosphere.
Kickout.
Protective system.
Ramp.

Registered professional engineer.
Sheeting.
Shield (Shield system).
Shoring (Shoring-system).
Sides.
Sloping (Sloping system).
Stable rock.
Structural ramp.
support system.
Tabulated data.
Trench (Trench excavation).
Trench box.
Trench shield.
Uprights
Wales.

1541. General Requirements.

(a) Surface encumbrances. [To be removed or supported]
(b) Underground installations. [Notify Underground Alert]
(c) Access and egress. [Ramps and ladders]
(d) Exposure to vehicular traffic. [Reflectorized apparel]
(e) Exposure to falling loads.
(f) Warning system for-mobile equipment. [For operators]
(g) Hazardous atmospheres.
(h) Protection from water hazards.
(i) Stability of adjacent structures.
(j) Protection from loose rock or soil.
(k) Inspections. [By a competent person]
(1) Fall protection. [Walkways, bridges, barricades, covers]

1541.1 Requirements for Protective Systems.

(a) Protection of employees in excavations.
(b) Design of sloping and benching systems.
(c) Design of support, shield, and other protective systems.
(d) Materials and equipment. [Protection and approval of]
(e) Installation and removal of supports.
(f) Sloping and benching systems.
(g) Shield systems.

Appendix A to Section 1541.1 [See Appendix A]

<u>SOIL CLASSIFICATION</u>:

(a) Scope and application.
(b) Definitions. [of soil properties]
[Soil classification system]
[Soil types - A, B, or C]
(c) Requirements. [classifying the soil]
(d) Visual and manual tests. [Acceptable methods]

Appendix B to Section 1541.1 **[See Appendix A].**

SLOPING AND BENCHING:

(a) Scope and application. [Specifications]
(b) Definitions. [Actual and allowable slopes - distress]
(c) Requirements. [Classification,surcharges,configurations]
Table B-l Allowable slopes
Figure B-l Slope configuration diagrams

Appendix C to Section 1541.1 **[See Appendix A]**

TIMBER SHORING FOR TRENCHES:

(a) Scope. [Timber can be used in lieu of other systems]
(b) Soil Classification. [Per Appendix A of Section 1541.11]
(c) Presentation of Information. [Allows use of 2 Tables]
(d) Basis and limitations of the data.
[Restrictions for surcharges - slopes - bracing loads]
(e) Use of Tables: [Minimum sizes - maximum spacings]
(f) Examples to Illustrate the use of Tables.
Tables C-1.1 through C-1.3 are for rough lumber
Tables C-2.1 through C-2.3 are for nominal lumber

Appendix D to Section 1541.1 **[See Appendix A]**

ALUMINUM HYDRAULIC SHORING FOR TRENCHES:

(a) Scope. [Use this or manufacturers data]
(b) Soil Classification. [Per Appendix A of Section 1541.11
(c) Presentation of information. [Explanation of Tables]
Tables D-l.1 and D-l.2 are for Type A and B soils
Tables D-l.3 and D-l.4 are for Type B and C soils
(d) Basis and limitations of the data.
(f) Examples to illustrate 'use of the Tables.
Figures 1 - 4 illustrate typical installations
Tables D-1.1 through D-l.4

Appendix E to Section 1541.1 **[See Appendix A]**

ALTERNATIVES TO TIMBER SHORING:

Figure 1. Aluminum Hydraulic Shoring
Figure 2. Pneumatic/hydraulic Shoring
Figure 3. Trench Jacks (Screw jacks)
Figure 4. Trench Shields

Appendix F to Section 1541.1 [See Appendix A]

SELECTION OF PROTECTIVE SYSTEMS: [Flow Charts]

1542. Shafts.

(a) General.
(b) Small Shafts in Hard Compact soil.
(c) Shafts in Other Than Hard Compact Soil.
(d) Bell Excavations.
(e) Exploration Shafts.

1543. Cofferdams.

(a) Overtopping.
(b) Warning signs.
(c) Rapid exit provisions.
(d) Protection from navigable shipping.

1544. Earthwork and Excavating.

Text deleted.

1545. Overburden.

Text deleted.

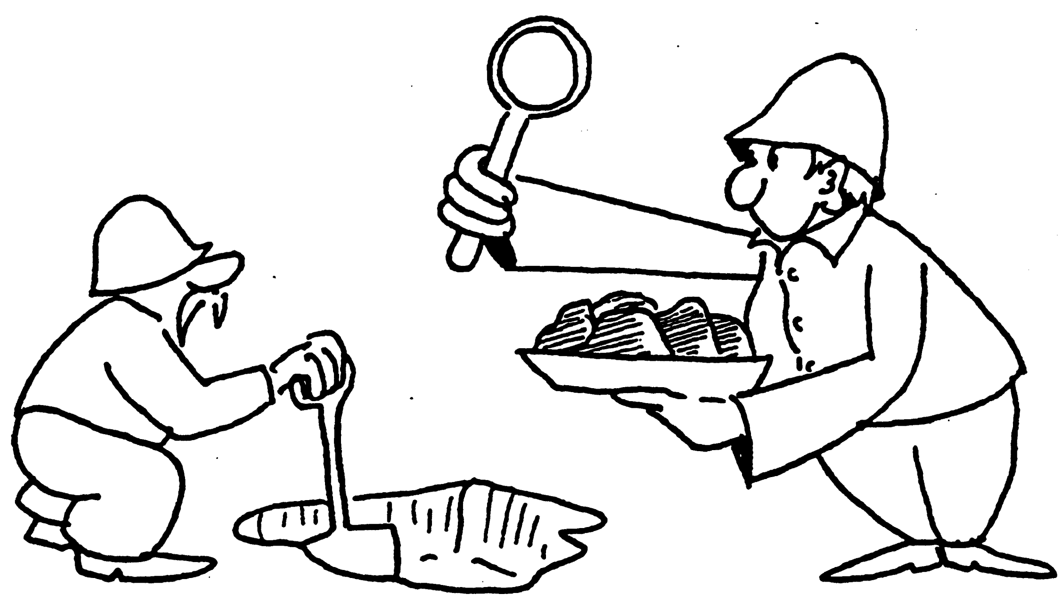
1546. Face Inspection and Control.

Text deleted.

1547. Protection of Workers at the Face.

Text deleted.

SOIL CLASSIFICATION AND PROPERTIES



SOIL CLASSIFICATION AND PROPERTIES

To engineer an adequate earth retaining system for a trench or excavation it is first necessary to identify the material (soil) in which the excavation is to be made. The basic load upon an earth retaining system is caused by the resulting lateral pressure of the retained soil. Soils can be quite different. It is these differences that result in variations in the lateral earth loads or pressures.

In general, there are two types of soils; cohesionless soils (sand and gravel) and cohesive soils (clays). Silts, depending on plasticity, may or may not be considered cohesionless. Natural soils areusually between these two extremes. Unusual soil types such as organic peat and permafrost conditions are not addressed in this manual.

A soil classification defines what soil is comprised of (silty sand for example). Various classification systems have been established. The Department of Transportation prefers the use of the ASTM Unified Classification System. This system was initially developed by the U.S. Army Corps of Engineers and the U.S. Bureau of Reclamation in 1952. The classification system is discussed in detail in the California Department of Transportation Materials Manual Volume VI 1973. Geotechnical Engineers and Contractors will not always use the same classification system and word description. If enough of the soil properties are known, a soil can be described-or classified properly. in the Unified Soil Classification System.

Classification is only the first part of a soil description. There are various characteristics or properties that must be known in order to predict the effect of a soil on an earth retaining system. These are the Engineering Properties of the soil. Standard means of measuring and determining these properties have been developed.

Soils Investigation usually consists of obtaining representative soil samples, performing tests, and summarizing the data. Additional pertinent information such as ground-water conditions, recommendations for an equivalent fluids oil pressure (Kw) and shape of pressure distribution diagrams may also be included.

Soil shear strength is of primary concern in trenching and shoring work. One of the fundamental relationships governing soil shear strength was first recognized by Coulomb and can be expressed as follows:

$$s = c' + \sigma' \tan\phi'$$
$$= c' + (\sigma - u)\tan\phi'$$

where
- s = soil shear strenght
- c' = effective cohesion intercept
- σ' = effective normal stress (stress causing soil particles to press together)
- ϕ' = effective angle of internal friction
- σ = total normal stress
- u = pore water pressure

In general, this relationship between shear strength and normal stress is not linear for large stress ranges. Therefore the shear strength parameters c' and σ' should be defined for narrow stress ranges.

Depending on soil permeability and the degree of saturation, the presence of water will tend to prevent a soil from changing volume when it is loaded. Without volume change, there can be no change in effective normal stress and therefore no change in soil shear strength. When a soil is saturated, a change in loading will produce a change in pore water pressure which is in excess of thehydrostatic pore water pressure. Excess pore water pressure will not necessarily be positive, and may, depending on soil stress history and loading conditions, be negative.

If a soil has high permeability, such as a coarse sand, the excess pore water pressure will dissipate almost instantly and there will be an instantaneous change in shear strength.

If a soil has low permeability, a clay for example, then excess pore water pressure will dissipate very slowly and no change in shear strength will be observed for quite sometime. Because the strength of saturated impervious fine grained soils changes slowly with externally applied pressures, their strength can sometimes be expressed as:

$$s = s_u$$

where s_u is the undrained shear strength.

These fine-grained types of soil, because their shear strength is initially indifferent to confining pressures, are often said to derive their strength through "cohesion" and are many times referred to as "cohesive soils."

In cohesive soils, excess pore pressure will reach zero over a period of time as the soil either consolidates or swells (depending respectively on whether the soil has been loaded or unloaded.) Trenching and shoring work often creates situations where soil loading is reduced - an excavation for example. A fine-grained soil in this situation will tend to expand and has the potential to lose shear strength over time.

Soil permeability, drainage and loading conditions, and degree of saturation greatly effect the pore pressures generated when soils are loaded, which in turn significantly affect Soil shear strength. Therefore, these factors require careful consideration when planning or evaluating a soil testing program to estimate shear strength parameters for use in analysis.

A few helpful TABLES and FIGURES are included in this section listing various soil properties and relationships.

An essential value for the determination of some of the soil relationships described in TABLE 7 is the specificgravity (G) of the soil. The specific gravity (G) of a soil may be satisfactorily estimated in accordance with the following:

Soil Type	Specific Gravity (G)
Sands and Gravels	2.65 - 2.68
Inorganic Silt	2.62 - 2.68
Organic Clay	2.58 - 2.65
Inorganic Clay	2.68 - 2.75

VOLUME AND WEIGHT RELATIONSHIPS

Property			Saturated sample (W_s, W_w, G, are known)	Unsaturated sample (W_s, W_w, G, V are known)	Supplementary formulas relating measured and computed factors			
Volume components	V_s	volume of solids	$\frac{W_s}{G\gamma_w}$		$V-(V_a+V_w)$	$V(1-n)$	$\frac{V}{(1+e)}$	V_v/e
Volume components	V_w	volume of water	$W_w/\gamma_w{}^*$		V_v-V_a	SV_v	$\frac{SVe}{(1+e)}$	SV_se
Volume components	V_a	volume of air or gas	zero	$V-(V_s+V_w)$	V_v-V_w	$(1-S)V_v$	$\frac{(1-S)Ve}{(1+e)}$	$(1-S)V_se$
Volume components	V_v	volume of voids	$W_w/\gamma_w{}^*$	$V-\frac{W_s}{G\gamma_w}$	$V-V_s$	$\frac{V_sn}{1-n}$	$\frac{Ve}{(1+e)}$	V_se
Volume components	V	total volume of sample	V_s+V_w	measured	$V_s+V_a+V_w$	$\frac{V_s}{1-n}$	$V_s(1+e)$	$\frac{V_v(1+e)}{e}$
Volume components	n	porosity	V_v/V		$1-V_s/V$	$1-\frac{W_s}{GV\gamma_w}$	$\frac{e}{1+e}$	
Volume components	e	void ratio	V_v/V_s		V/V_s-1	$\frac{GV\gamma_w}{W_s}-1$	$\frac{W_wG}{W_sS}$	$\frac{n}{1-n}$ $\frac{wG}{S}$
Weights for specific sample	W_s	weight of solids	measured		$\frac{W_T}{(1+w)}$	$GV\gamma_w(1-n)$	$\frac{W_wG}{eS}$	
Weights for specific sample	W_w	weight of water	measured		wW_s	$S\gamma_wV_v$	$\frac{eW_sS}{G}$	
Weights for specific sample	W_t	total weight of sample	W_s+W_w		$W_s(1+w)$			
Weights for sample of unit volume	γ_D	dry unit weight	$\frac{W_s}{V_s+V_w}$	W_s/V	$\frac{W_T}{V(1+w)}$	$\frac{G\gamma_w}{(1+e)}$	$\frac{G\gamma_w}{1+wG/S}$	
Weights for sample of unit volume	γ_T	wet unit weight	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+W_w}{V}$	W_T/V	$\frac{(G+Se)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w/S+1/G}$	
Weights for sample of unit volume	γ_{SAT}	saturated unit weight	$\frac{W_s+W_w}{V_s+V_w}$	$\frac{W_s+V_v\gamma_w}{V}$	$W_s/V+\left(\frac{e}{1+e}\right)\gamma_w$	$\frac{(G+e)\gamma_w}{(1+e)}$	$\frac{(1+w)\gamma_w}{w+1/G}$	
Weights for sample of unit volume	γ_{SUB}	submerged (buoyant) unit weight	$\gamma_{SAT}-\gamma_w{}^*$		$W_s/V-\left(\frac{1}{1+e}\right)\gamma_w{}^*$	$\left(\frac{G+e}{1+e}-1\right)\gamma_w{}^*$	$\left(\frac{1-1/G}{w+1/G}\right)\gamma_w{}^*$	
Combined relations	w	moisture content	W_w/W_s		W_t/W_s-1	$\frac{Se}{G}$	$S\left[\frac{\gamma_w{}^*}{\gamma_D}-\frac{1}{G}\right]$	
Combined relations	S	degree of saturation	1.00	V_w/V_v	$\frac{W_w}{V_v\gamma_w{}^*}$	$\frac{wG}{e}$	$\frac{w}{\left[\frac{\gamma_w{}^*}{\gamma_D}-\frac{1}{G}\right]}$	
Combined relations	G	specific gravity	$\frac{W_s}{V_s\gamma_w}$		$\frac{Se}{w}$			

Notes.—γ_w is unit weight of water, which equals 62.4 pcf for fresh water and 64 pcf for sea water (1.00 and 1.025 gm/cc). (Where noted with * the actual unit weight of water surrounding the soil is used.) In other cases use 62.4 pcf.
Values of (w) and (s) are used as decimal numbers.

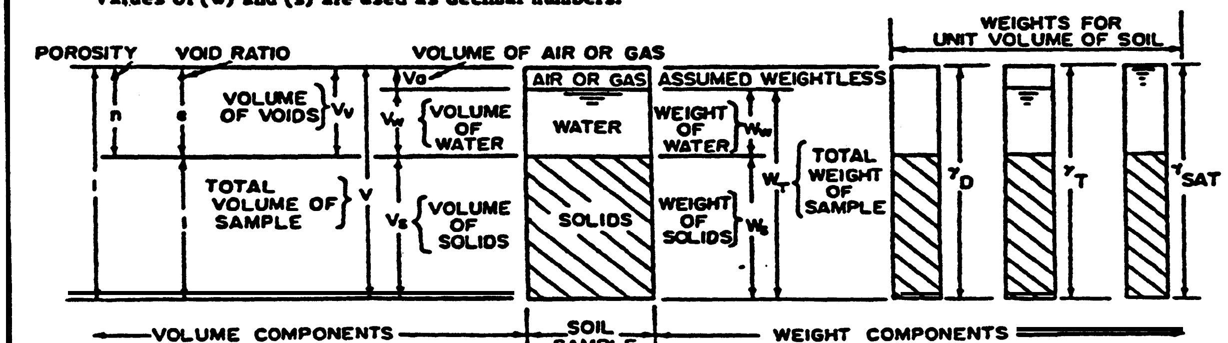


TABLE 7

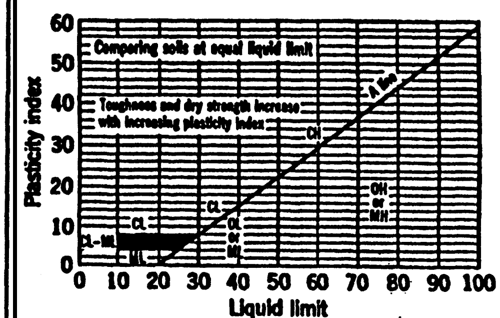
UNIFIED SOIL CLASSIFICATION CHART

Major divisions			Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)	Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria	
Coarse-grained soils: More than half of material is *larger* than No. 200 sieve size[b] (The No. 200 sieve size is about the smallest particle visible to naked eye)	Gravels: More than half of coarse fraction is larger than No. 7 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 7 sieve size)	Clean gravels (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes	*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing	*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*	
		Gravels with fines (Appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)	*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics	Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7	
	Sands: More than half of coarse fraction is smaller than No. 7 sieve size	Clean sands (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes	*SW*	Well graded sands, gravelly sands, little or no fines	Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% nonplastic fines with low dry strength; well compacted and moist in place; alluvial sand; *(SM)*	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			Predominantly one size or a range of sizes with some intermediate sizes missing	*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*	
		Sands with fines (Appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)	*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)	*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7	

Use grain size curve in identifying the fractions as given under field identification.

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:
Less than 5% *GW, GP, SW, SP*
More than 12% *GM, GC, SM, SC*
5% to 12% *Borderline* cases requiring use of dual symbols

Major divisions		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)	Group Symbols[a]	Typical Names	Information Required for Describing Soils
Fine-grained soils: More than half of material is *smaller* than No. 200 sieve size (The No. 200 sieve size is about the smallest particle visible to naked eye)	Identification Procedures on Fraction Smaller than No. 40 Sieve Size						
	Silts and clays, liquid limit less than 50	None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses
		Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
		Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions
	Silts and clays, liquid limit greater than 50	Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	
		High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays	Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; *(ML)*
		Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity	
Highly Organic Soils		Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils	



Plasticity chart
for laboratory classification of fine grained soils

TABLE 8

SOIL PROPERTY NOMENCLATURE

γ_{dry}	= Dry Unit Weight (pcf)
γ_{sat}	= Saturated Unit Weight (pcf)
γ_{sub}	= Submerged Unit Weight (pcf)
γ_w	= Unit Weight of Water (pcf)
γ_{sat}	$= \gamma_{sub} + \gamma_w$
γ_{sub}	$\approx 0.6\gamma_{dry} \approx \gamma_d(1 - 1/G)$ [Assuming G = 2.65]
γ_{sat}	$\approx 0.6\gamma_{dry} + \gamma_w$
q_u	= Unconfined compressive strength (psf)
s_u	= Undrained Shear Strength = $q_u/2$ (psf) for the unconfined compression test.
c	= Cohesion intercept: the component of soil shear strength which is independent of the force normal to the shear plane.
ϕ	= Angle of Internal Friction (degree): represents the frictional component of soil shear strength, dependent on the forces pushing the particles together.
θ	= Angle of Repose (degrees) (The angle at which a soil will stand unsupported).
w_p	= PLASTIC LIMIT: This is the moisture content at which the soil mass ceases to be plastic; it will become brittle or crumbly without any further reduction in moisture content.
w_L	= LIQUID LIMIT: The water content at which the soil has such a small shear strength that it flows to close a grove of standard width when jarred in a specified manner.

TABLE 9

Test Borings permit classification and consistency determinations of underlying soils. Soils usually change at various levels or depths. Sometimes it is necessary to identify soils below the bottom of the proposed excavation. Soils samples for testing may be extracted and the presence and the elevation of ground water determined.

The Standard Penetration Test (SPT) is a means of retrieving, from the bottom of a bore hole, a disturbed sample of soil for visual classification or index testing. The number of hammer blows used to drive the sampler provides an indication of thedensity of a granular soil or the consistency of a fine-grained soil. Empirical relationships can then be used which estimated soil friction angle (ϕ)from density of granular soils and unconfined compressive strength (q_u) from consistency of fine-grained soils.

Standard test procedures have been developed to obtain the properties from field samples. Some of these tests will be performed in the field and the remainder in a soils laboratory. For detailed information on soils investigation procedures and tests, see the Department of Transportation Materials Manual Volume VI, 1973.

There are several sources of soils information usually available to the Contractor and the Engineer. When structures are included in a contract there will be a report prepared by the Transportation Materials and Research Laboratory Engineering Geology Branch. The "Log of Test Borings", which is included as part of the contract plans is from this report. The soil is classified and its density or hardness at various elevations is determined. Moisture content and ground water conditions may also be given. If the borings are within reasonable distance of the proposed trench work they may serve as a guide to review or confirmthe soils data submitted with the shoring or excavation plans. A sample of a Log of Test Borings is shown in FIGURE 5.

Soils investigations may also be made by District Materials and Headquarters Laboratories. Numerous properties can be developed beyond those normally shown on the Log of Test Borings. This additional soils information is a part of the project materials -report. An example portion of a reporting form is shown in FIGURE 6.

Material reports are available on request. The Contractor should be informed of the availability of soils information at the pre-job conference. Observation of adjacent work in similar material by both the engineer and the Contractor during such operations as pile driving and excavations will often supply useful soils information.

The Contractor may elect, or find it necessary, to have a soils investigation performed. In this case the soils information or report will be furnished to the Engineer as a part of the supporting data accompanying the shoring plans.

It is recommended that exploratory borings be made and soil properties determined for unusual conditions such as a very deep excavation, work adjacent to buildings, and known areas of potentially unstable ground. This is especially critical when ground water level is close to the surface. Materials that require special consideration include existing and former tidal flats, estuaries, marshes, alluvial flats, andground reclaimed by fill.

Soil test results need to be used with caution. Soil test reports from many soils laboratories or similar sources will include safety factors incorporated in the reported results. Other soil test data may not include safety factor considerations. The soil properties shown in FIGURE 6 for example, are direct result values which do not contain appropriate factors of safety.

Factors which the engineer will consider when assigning strength parameters to a soil include:

- the method with which soil shear strengthwas determined.
- the variability of subsurface-profile.
- the number and distribution of shear strength tests.

Of these factors only the first can be addressed here, the others must be dealt with on a site specific basis.

Many ways of evaluating soil shear strength have been developed. Not all methods are equally precise, therefore one needs to consider the source of the shear strength data when making an evaluation of a proposed trenching or shoring system. The following is a guide for judging the reliability of soil shear strength parameters:

Test Method	Course-grained soil	Fine-grained soil
Triaxial compression test	very good*	very good
Unconfined compression test	mot applicable	very good
Direct shear test	good*	fair
Vane shear test	not applicable	good
Cone penetration test (CPT)	fair	fair
Pocket penetrometer	not applicable	fair
Standard penetration test (SPT)	fair	poor

*Recovery of undisturbed samples can be difficult.

Additional information concerning soils testing may be found in Appendix B.

Unstable ground conditions may also be encountered in areas underlain by soils developed in-situ from weathering of rock and in deeply weathered and sheared rock. Adversely oriented bedding, fracturing and jointing planes of shear zones which are inclined towards the excavation should be investigated.

Instability of excavation walls can also occur in certain geologic formations such as clays and shales that are subject to cracking and spalling upon exposure to the atmosphere and to swelling when saturated with water. Excavating in such materials requires protection of the excavation walls to help retain natural moisture content and thus prevent cracking, spalling, and eventual wall instability. If cracking does occur, water should be prevented from seeping into the cracks.

A term that comes up frequently in trenching work is "running ground." It is referred to in the Construction Safety Orders and is criteria for more restrictive requirements for a shoring system. A running ground is defined as a soil that cannot stand by itself even for a short term, and is the dynamic state of actual failure or cave-in. Running soil will have little shear strength and will flow with virtually no angle of response in an unsupported condition. A mud under pressure which flows is an example of running ground. For running ground conditions, the full dry weight or the saturated unit weight of the material has to be resisted. The angle of internal friction (ϕ)and the cohesive value, are both zero. The shoring system wall in contact with the material must be solid. Running soil is the most adverse soil conditions that can be encountered. The soils investigation should state if a soil is known to be running.

Quick sand is a type of running soil. It occurs in cohesionless soil when the force of the upward flow of water is sufficient to make the soil bouyant and there by prevent grain interlocking. The soil grains are suspended in the water. *A* quick condition can be developed by adverse water flow. It may best be stabilized when the trench is dewatered. Quick conditions can occur in silt as we11 as in sand.

Major Divisions			Group Symbols	Typical Names
Coarse-grained Soils	Gravels	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands, gravelly sands, little or no fines
		Sands with fines (Appreciable amount of fines)	SM	Silty Sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures

Major Divisions		Group Symbols	Typical Names
Fine-grained soils	Silts and Clays (Liquid limit less than 50)	ML	Inorganic silts and very find sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays (Liquid limit greater than 50)	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
	Highly Organic Soils	Pt	Peat and other highly organic soils

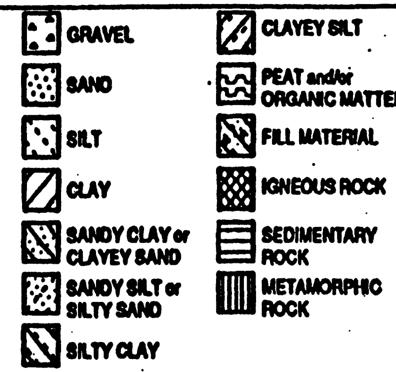
UNIFIED SOIL CLASSIFICATION SYSTEM

CONSISTENCY CLASSIFICATION FOR SOILS

According to the Standard Penetration Test

Penetration Index (Blows / Ft)	Granular	Cohesive
0-4	Very loose	Very soft
5-9	Loose	Soft
10-19	Slightly compact	Stiff
20-34	Compact	Very stiff
35-69	Dense	Hard
>70	Very dense	Very hard

LEGEND OF EARTH MATERIALS



NOTE: Classification of earth material as shown on this sheet is based upon field inspection and is not to be construed to imply mechanical analysis unless gradations and Unified Soil Classification are shown.

Blows per foot may be converted to approximate ϕ angle or approximate cohesion (C)

LEGEND OF BORING OPERATIONS

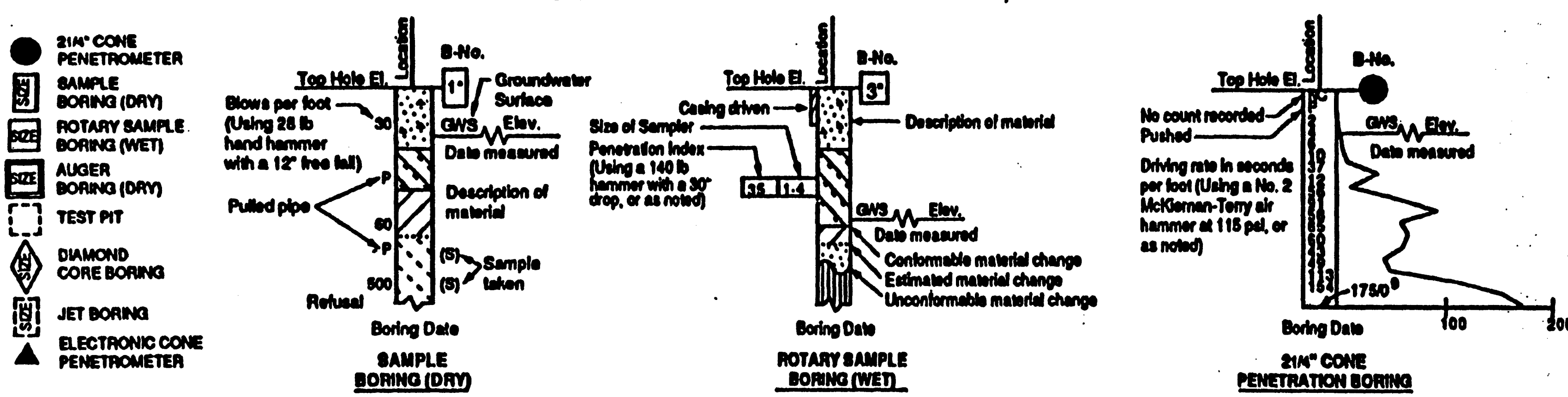
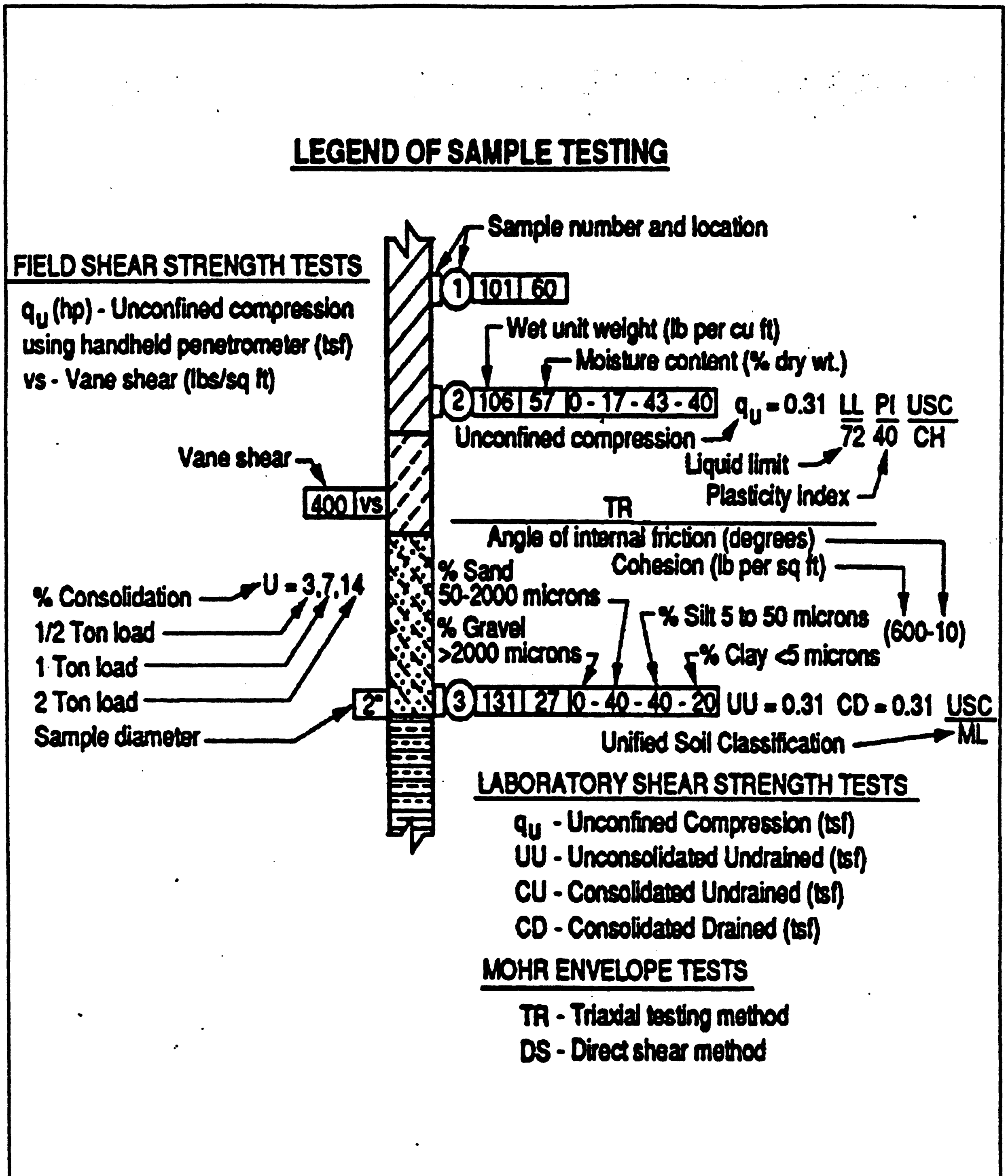


FIGURE 5



Example Boring Legend
As Used On State Plans

FIGURE 6

UNIFIED SOIL CLASSIFICATION SYSTEM

Inches	U.S.Standard Sieve No.	Particle Size
Over 8"		BOULDER
8" to 3"		COBBLE
3" to 3/4"		GRAVEL Coarse
3/4" to	4	GRAVEL Fine
	4 TO 10	SAND Coarse
	10 TO 40	SAND Medium
	40 TO 200	SAND Fine
	Under 200	SILT or CLAY

1 micron = 0.001 mm

TABLE 10

USEFUL CONVERSIONS	
Pa = Pascal = N/m^2	1 Ft^3 = 0.028 m^3
N= Newton	1 Ft^2 = 0.093 m^2
1 ksf = 47.88 kPa = 47.88 kN/m^2	1 in = 0.025 m
1 Lb/Ft^2 = 0.048 kN/m^2 = 47.9 N/m^2	1 in^2 = 645.16 mm^2
1 Lb/in^2 = 6.89 kN/m^2	1 kip = 4.448 kN
1 Ton/Ft^2 = 2 ksf = 95.76 kPa	1 kg = 9.807 N

SOIL CLASSIFICATION AND PROPERTIES

The Transportation Materials and Research Laboratory Engineering Geology Branch has prepared a summary of "simplified typical soil values" For average trench conditions the Engineer will find the data very useful to establish basic properties or evaluate data presented by the Contractor. The following table lists approximate values.

SIMPLIFIED TYPICAL SOIL VALUES

Soil Classification	ϕ Friction Angle of the Soil	Density or Consistency	γ Soil Unit Weight (pcf)	K_a	$Kw=K_a\gamma$ Equiv. Fluid Wt. (pcf)
Gravel, Gravel-	41	Dense	130	.21	27
Sand Mixture,	34	Compact	120	.28	34
Coarse sand	29	Loose	90	.35	32
Medium Sand	36	Dense	117	.26	30
	31	Compact	110	.32	35
	27	Loose	90	.38	34
Fine Sand	31	Dense	117	.32	37
	27	Compact	100	.38	38
	25	Loose	85	.41	34
Fine Sand Silty,	29	Dense	117	.35	41
Sandy Silt	27	Compact	100	.38	38
	25	Loose	85	.41	34
Silt	27	Dense	120	.38	45
	25	Compact	110	.41	45
	23	Loose	85	.44	37

TABLE 11

For active pressure conditions use a unit weight value of γ = 115 psf minimum when insufficient soils data is known.

A rough correlation between the standard penetration index value (N) and the angle of internal friction (ϕ) in a granular can be made as shown by TABLE 12. Also, the penetration index can be related to the cohesive value (C) in a cohesive soil as shown in TABLE 13. The standard penetration index is converted to q_u (unconfinedcompressive strength) which in turn is equated to "C" by the formula, $C = q_u/2$.

Please note that these conversion tables are approximate. They can be used by characterizing the soil as being either predominately granular or cohesive. If possible, the conversion of the penetration index (N Value) should be checked by performing laboratory or in-site tests.

GRANULAR SOILS

COMPACTNESS	VERY LOOSE		LOOSE		MEDIUM		DENSE		VERY DENSE
Relative Density, D_d		15%		35%		65%		85%	
Standard Penetration Resistance, N = Blows/ft*		4		10		30		50	
Angle of Internal Friction, ϕ		28		30		36		41	
Unit Weight (PCF)									
Moist	100		95-125		110-130		110-140		130+
Submerged	60		55-65		60-70		65-85		75+

VERY LOOSE: A reinforcing rod can be pushed into soil several feet.
DENSE: Difficult to drive a 2x4 stake with a sledge hammer.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

$$\text{Relative Density, } D_d = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

e = existing void ratio of mass being considered.
e_{max} = void ratio of same mass in its loosest state.
e_{min} = void ratio of same mass in its most compact state.

TABLE 12

COHESIVE SOILS

CONSISTENCY	VERY SOFT		SOFT		MEDIUM		STIFF		VERY STIFF		HARD
q_u = unconfined comp. strength (PSF)		500		1000		2000		4000		8000	
Standard Penetration Resistance, N = Blows/Ft *		2		4		8		16		32	
Unit Weight (PCF) Saturated		100-120			110-130			120-140			130+

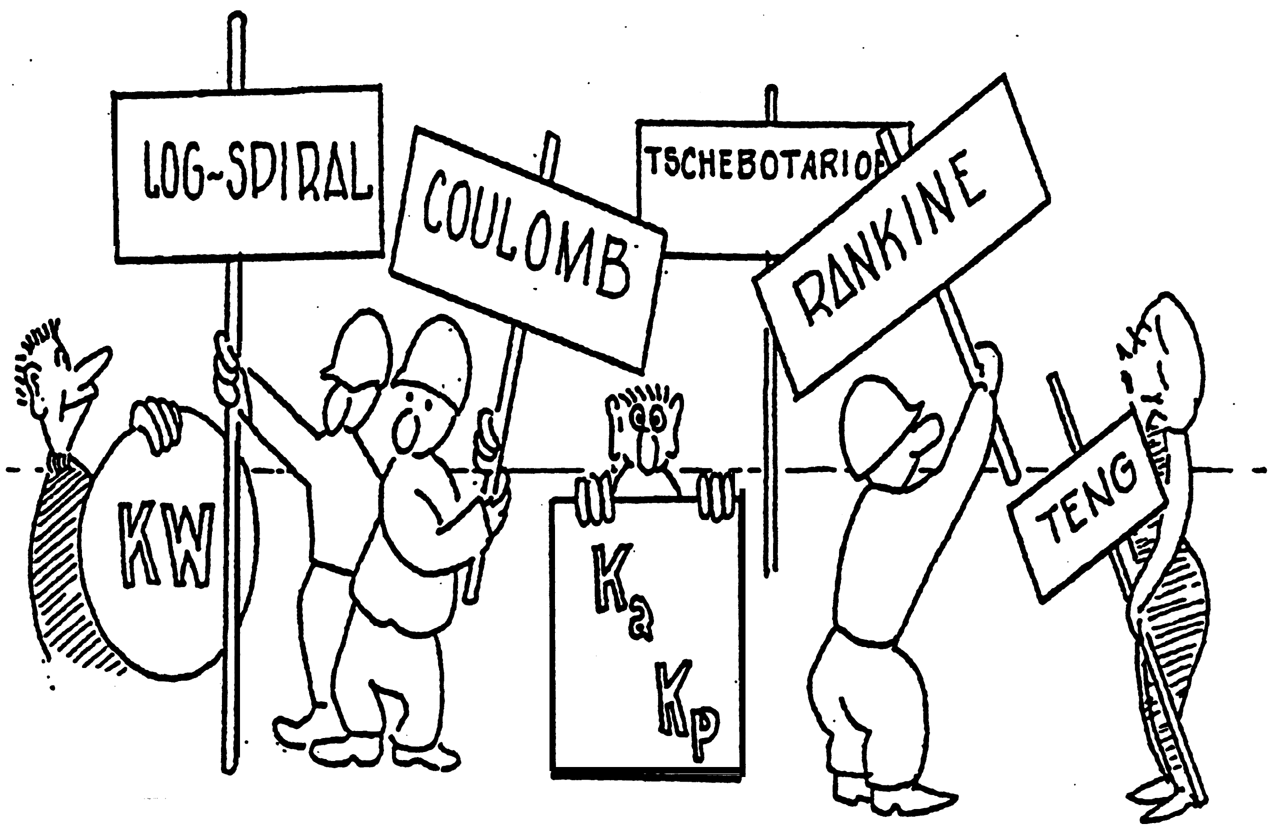
VERY SOFT: Exudes from between fingers when squeezed in hand.
SOFT: Molded by light finger pressure.
MEDIUM: Molded by strong finger pressure.
STIFF: Indent by thumb.
VERY STIFF: Indent by thumb nail.
HARD: Difficult to indent by thumb nail.

* N = Blows/Ft as measured by the standard penetration test (See Appendix B).

To be used only as a rough guide.

TABLE 13

EARTH PRESSURE THEORY AND APPLICATION

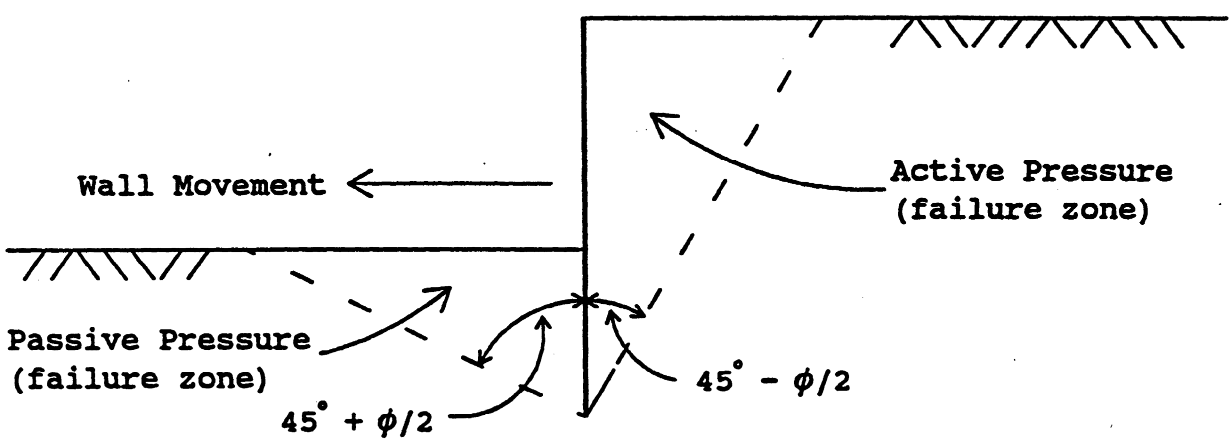


EARTH PRESSURE THEORY AND APPLICATION

Earth pressure is the lateral force exerted by the soil on a shoring system. It is dependent on the soil structure and the interaction or movement with the retaining system. Due to many variables, shoring problems can be highly indeterminate. Therefore, it is essential that good engineering judgment be used.

ACTIVE AND PASSIVE EARTH PRESSURES

Active and passive earth pressures are the two stages of stress in soils which are of particular interest in the design or analysis of shoring systems. Active pressure is the condition in which the earth exerts a force on a retaining system and the members tend to move toward the excavation. Passive pressure is a condition in which the retaining system exerts a force on the soil. Since soils have a greater passive resistance, the earth pressures are not the same for active and passive conditions. When a state of oil failure has been reached, active and passive failure zones, approximated by straight planes, will develop as shown in the following figure (level surfaces depicted).



The well known earth pressure theories of Rankine and Coulomb provide expressions for the active and passive pressure for a soil mass at a state of failure.

COEFFICIENT OF EARTH PRESSURE

The coefficient of earth pressure (K) is the term used to express the ratio of the lateral earth pressure to the vertical earth pressure or unit weight of the soil. For a true fluid the ratio would be l. The vertical pressure is determined by using a fluid weight equal to the unit weight of the soil: $P_v = KP_v$ The basic formulas for horizontal earth pressures are as follows:

$P_H = KP_V = K\gamma H$ = Lateral earth pressure

If a soil has a cohesive value the formula becomes:

$$P_H = K\gamma H \pm 2C[K]^{1/2}$$

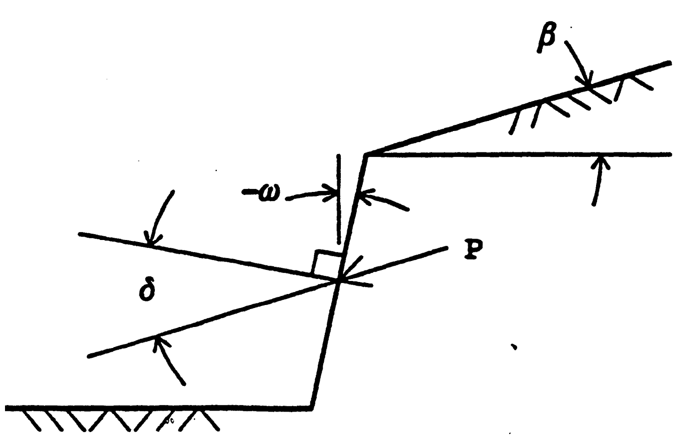
There are three ranges of earth pressure coefficients to be considered:

Ka = Coefficient of Active earth pressure (0.17 to 1.0)
K_p = Coefficient of Passive earth pressure (1.0 to 10.0)
K_0 = Coefficient of earth pressure for soils at rest or in place (0.4 to 0.6 for drained soils).

The next step is to determine the value of the earth pressure coefficient (K). This is accomplished by utilizing the known soil properties and the accepted theories, formulas, graphs and procedures that are available.

Refer to the Table of Simplified Typical Soil Values TABLE 11, which lists active coefficient (K_a) and equivalent fluid (Kw) directly.

Earth pressure coefficients may also be calculated by acceptable soil mechanics formulas. Two of the more well known authors are Rankine and Coulomb.



ϕ = Angle of internal friction

β = Angle of the backfill slope

δ = Angle of wall friction

ω = Angle of the wall with respect to vertical (negative value shown)

P = Active lateral earth pressure

THE RANKINE THEORY

The Rankine theory assumes that there is no wall friction ($\delta = 0$) the ground and failure surfaces are straight planes, and that the resultant force acts parallel to the backfill slope. The coefficients according to Rankine's theory are given by the following expressions:

$$K_a = \cos\beta \left[\frac{\cos\beta - [\cos^2\beta - \cos^2\phi]^{1/2}}{\cos\beta + [\cos^2\beta - \cos^2\phi]^{1/2}} \right]$$

$$K_p = \cos\beta \left[\frac{\cos\beta + [\cos^2\beta - \cos^2\phi]^{1/2}}{\cos\beta - [\cos^2\beta - \cos^2\phi]^{1/2}} \right]$$

If the embankment is level ($\beta = 0$) the equations are simplified as follows:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \tan^2(45^\circ - \phi/2)$$

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi} = \tan^2(45^\circ + \phi/2)$$

The Rankine formula for passive pressure can only be used correctly when the embankment slope angle β equals zero or is negative. If a large wall friction value can develop, the Rankine Theory is not correct and will give less conservative results. Rankine's theory is not intended to be used for determining earth pressures directly against a wall (friction angled does not appear in equations above). The theory is intended to be used for determining earth pressures on a vertical plane within a mass of soil.

THE COULOMB THEORY

The Coulomb theory provides a method of analysis that gives the resultant horizontal force on a retaining system for any slope of wall, wall friction, and slope of backfill provided $\beta \leq \phi$. This theory is based on the assumption that soil shear resistance develops along the wall and failure plane. The following coefficient is for a resultant pressure acting at angle δ.

$$K_a = \frac{\cos^2(\phi - \omega)}{\{\cos^2\omega\}\{\cos(\delta + \omega)\}\left[1 + \sqrt{\dfrac{\{\sin(\phi + \delta)\}\{\sin(\phi - \beta)\}}{\{\cos(\delta + \omega)\}\{\cos(\beta - \omega)\}}}\right]^2}$$

The passive K_p value for sloping embankment is not listed since this value can be drastically overestimated.

The following coefficients are for a horizontal resultant pressure and a vertical wall:

$$K_a = \frac{\cos^2 \phi}{\cos \delta \left[1 + \sqrt{\dfrac{\{\sin(\phi + \delta)\}\{\sin(\phi - \beta)\}}{(\cos \delta)(\cos \beta)}}\right]^2}$$

$$K_p = \frac{\cos^2 \phi}{\cos \delta \left[1 - \sqrt{\dfrac{\{\sin(\phi + \delta)\}\{\sin(\phi + \beta)\}}{(\cos \delta)(\cos \beta)}}\right]^2}$$

Wall friction angle (δ) varies from 0° to 22°, but is always less than the internal angle of friction of the soil (ϕ). It is accepted practice to assume a value of $\delta = 1/3(\phi)$ to $2/3(\phi)$. See TABLE 14 for some typical friction values.

If the shoring system is vertical and the backfill slope and wall friction angles are zero (ω, β and $\delta = 0$), Coulomb's equation will be the same as Rankine's for a level ground condition. Coulomb's pressure distribution has been shown to be essentially correct for the lateral movements of sheeting of braced cuts which closely correspond to the conditions of rotation of a wall around its top.

Since wall friction requires a curved surface of sliding to satisfy equilibrium, the Coulomb formula will give only approximate results as it assumes planar failure surfaces. The accuracy for Coulomb will diminish with increased depth. For passive pressures the Coulomb formula can also give inaccurate results when there is a large back slope or wall friction angle. These conditions should be investigated and an increased factor of safety considered.

LOG-SPIRAL THEORY

A Log-spiral theory was developed because of the unrealistic values of earth pressures that are obtained by theories which assume a straight line failure plane. The difference between the Log-Spiral curved failure surface and the straight line failure plane can be large and on the unsafe side for Coulomb passive pressures (especially when wall friction exceeds $\phi/3$). The following figure shows a comparison of the Coulomb and Log-Spiral failure surfaces:

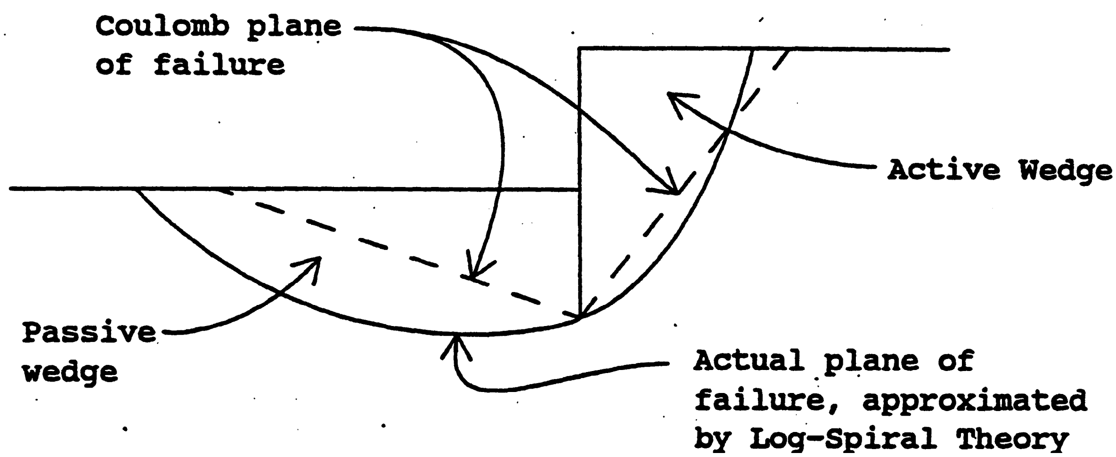


FIGURE 7

The coefficient of earth pressure values for the log-spiral failure surf ace can be obtained from FIGURE 8. K_a may be read directly from the curves using the lower portion of FIGURE 8 whereas K_p must be multipliedby a reduction factor (R) located at the top of the figure.

Rankine is conservative relative to other methods. Except for the passive condition when δ is greater than approximately $\phi/3$, Coulomb is conservative relative to Log-Spiral. These methods developed as refinements to one another; each in its turn accounting for more variables and thereby requiring increasing levels of analytic complexity.

EARTH PRESSURE COEFFICIENT WHEN AT-REST

The at-rest earth pressure coefficient (Ko) is applicable for, determining the active pressure in clays for strutted systems. Because of the cohesive property of clay there will be no lateral pressure exerted in the at-rest condition up to some height at the time the excavation is made. However, with time, creep and swelling of the clay will occur and a lateral pressure will develop. This coefficient takes the characteristics of clay into account and will always give a positive lateral pressure. This method is called the Neutral Earth Pressure Method and is covered in the text by Gregory Tschebotarioff.

$$K_o = \frac{v}{1 - v}$$

v = The Poisson's Ratio. It is determined by a Laboratory test (Maximum value = 0.5)

An alternate solution for K_o is to use Jaky's equation:

$$K_o = 1 - \sin \phi'$$

Where ϕ' is the effective angle of internal friction and not the total stress value. For most short term shoring situations the internal friction angle ϕ may be substituted for ϕ'.

In general, for a level ground situation, values of K_o will be greater than K_a. If movement of a retaining system is severely restricted (approaching a fixed condition) the active failure wedge cannot fully develop and consideration should be given to using K_o in lieu of K_a. For very deep excavations the horizontal movement that can occur is usually less than that needed to develop active failure condition, therfore K_o values should be used. It is noted that for deadman anchorages, K_o could be used to calculate the passive resistance.

WALL FRICTION (δ)

Wall friction angle (δ) varies from 0° to 22°, but is always less than the internal angle of friction of the soil (ϕ). It is accepted practice to assume a value of $\delta = 1/3(\phi)$ to $2/3(\phi)$. For systems subject to dynamic loading (adjacent railroads, pile driving operations, etc.) use $\delta = 0$. It is important to note that as wall friction increases, lateral pressures decrease, but the vertical load on the shoring system increase. Vertical load components must be considered in shoring design. TABLE 14 lists friction of select soil types acting against various structural materials.

ULTIMATE FRICTION FACTORS AND ADHESION FOR DISSIMILAR MATERIALS

INTERFACE MATERIALS	FRICTION ANGLE, δ DEGREES
Steel sheet piles against the following soils:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheet piling aginst the following soils:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	17 to 22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14
Mass concrete on the following materials:	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, coarse sand	29 to 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29
Clean fine sand, silty or clayey fine to medium sand	19 to 24
Fine sandy silt, nonplastic silt	17 to 19
Very stiff and hard residual or preconsolidated clay	22 to 26
Medium stiff and stiff clay and silty clay	17 to 19
(Masonry on foundation materials has same friction factors.)	
Various structural materials:	
Masonry on masonry, ingneous and metamorphic rocks:	
Dressed soft rock on dressed soft rock	35
Dressed hard rock on dressed soft rock	33
Dressed hard rock on dressed hard rock	29
Masonry on wood (cross grain)	26
Steel on steel at sheet pile interlocks	17

INTERFACE MATERIALS (COHESION)	ADHESION C_a (PSF)
Very soft cohesive soil (0 - 250 psf)	0 - 250
Soft cohesive soil (250 - 500 psf)	250 - 500
Medium stiff cohesive soil (500 - 1000 psf)	500 - 750
Stiff cohesive soil (1000 - 2000 psf)	750 - 950
Very stiff cohesive soil (2000 - 4000 psf)	950 - 1,300

TABLE 14 - WALL FRICTION AND ADHESION

POISSON'S RATIO

Soil Type	Typical Value for Poisson's Ratio *	K_o
Clay, saturated	0.40 - 0.50	0.67 - 1.00
Clay, unsaturated	0.10 - 0.30	0.11 - 0.42
Sandy Clay	0.20 - 0.30	0.25 - 0.42
Silt	0.30 - 0.35	0.42 - 0.54
Sand		
Dense	0.20 - 0.40	0.25 - 0.67
Coarse		
(void ratio 0.4 - 0.7)	0.15	0.18
Fine-grained		
(void ratio 0.4 - 0.7)	0.25	0.33
Rock	0.10 - 0.40	0.11 - 0.67

TABLE 15

MOVEMENT OF WALL NECESSARY TO PRODUCE ACTIVE PRESSURES

Soil Type	Wall Yield
Cohesionless, dense	0.001 H
Cohesionless, loose	0.001 - 0.002 H
Clay, firm	0.010 - 0.020 H
Clay, soft	0.020 - 0.050 H

* New Zealand Department of Public Works Retaining Wall Manual

TABLE 16

For sands, Terzaghi & Peck have indicated one could expect that a movement of 0.5% times the height of the support system would be needed to obtain a complete active condition. For a 20' deep excavation the movement needed at the top of the excavation would amount to (0.005) (20) = 0.1 foot of movement to develop a fully active condition.

Example:

Given: $\phi = 26°$, $\beta = -10°$, $\delta = \pm 7.5°$
$\beta/\phi = -0.4$, $\delta/\phi = -0.3$ (Passive)

To determine K_a use the lower portion of **FIGURE 8**.

1) Locate the curve that coresponds to a $\beta/\phi = -0.4$.

2) Find the vertical line for $\phi = 26°$.

3) Follow that line up to where it intersects with the curve for $\beta/\phi = -0.4$.

4) Proceed to the left of the graph and read K_a directly.

For this example K_a is approximately 0.34. The resultant force ($P = \gamma H K_a$) acts downward at an angle δ to the horizontal.

To determine K_p use the upper portion of **FIGURE 8**.

1) Locate the curve that corresponds to a $\beta/\phi = -0.4$.

2) Find the vertical line for $\phi = 26°$.

3) Follow that line up to where it intersects with the curve for $\beta/\phi = -0.4$.

4) Proceed to the left of the graph and read a value of approximately 3.4.

5) Next apply the reduction factor from the table at the top. NOTE: the R is not a safety factor.
For $\delta/\phi = -0.3$ and $\phi = 26°$, R = 0.694 by interpolation.

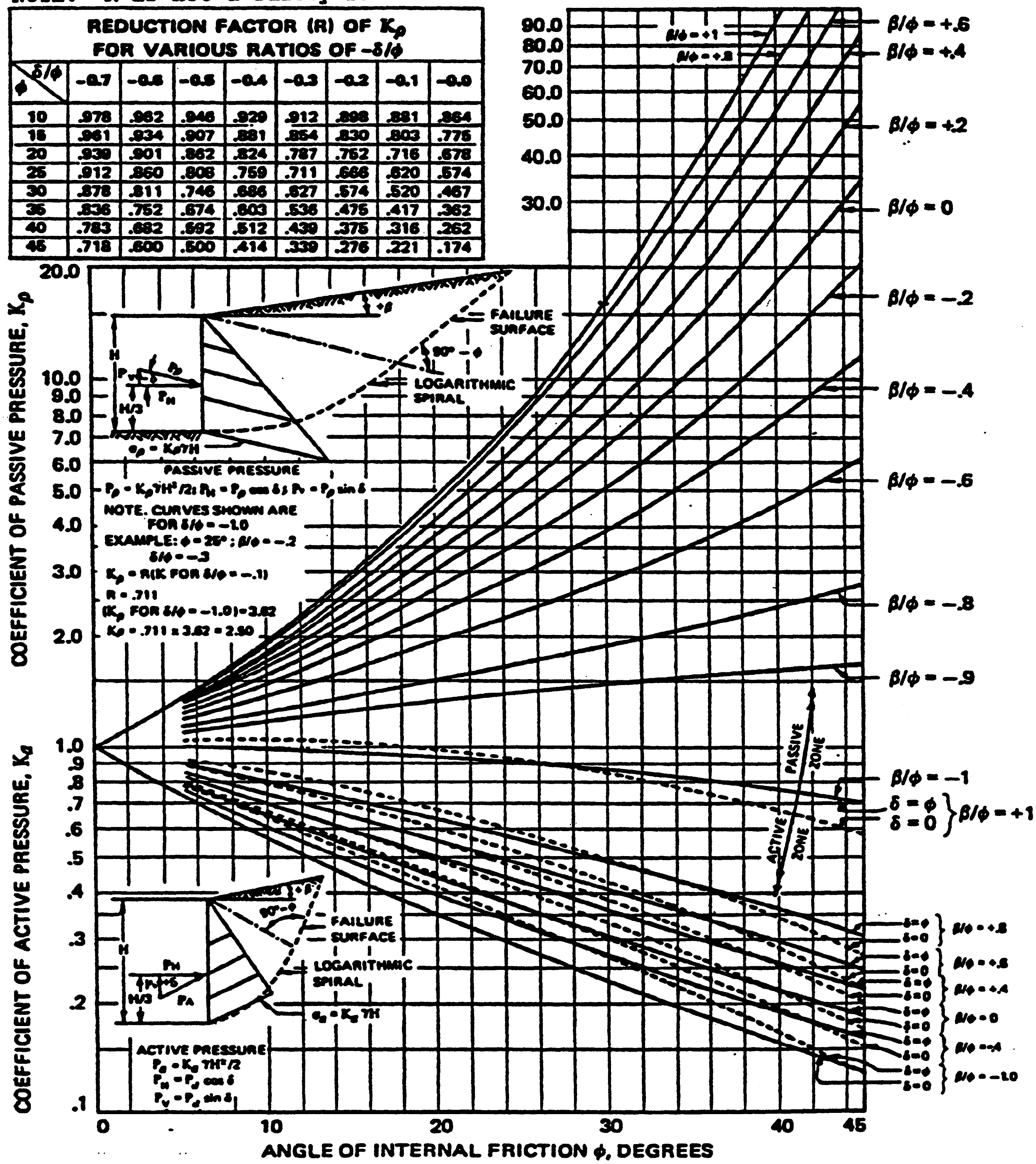
For this example then, K_p is approximately (3.4)(0.694) = 2.36. The resultant force ($P = \gamma H K_p$) acts upward at an angle δ to the horizontal.

LOG - SPIRAL FAILURE SURFACE

NOTE: R is not a safety factor.

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	-0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174



Active and passive coefficients with wall friction (sloping backfill)

FROM USS STEEL SHEET PILING DESIGN MANUAL

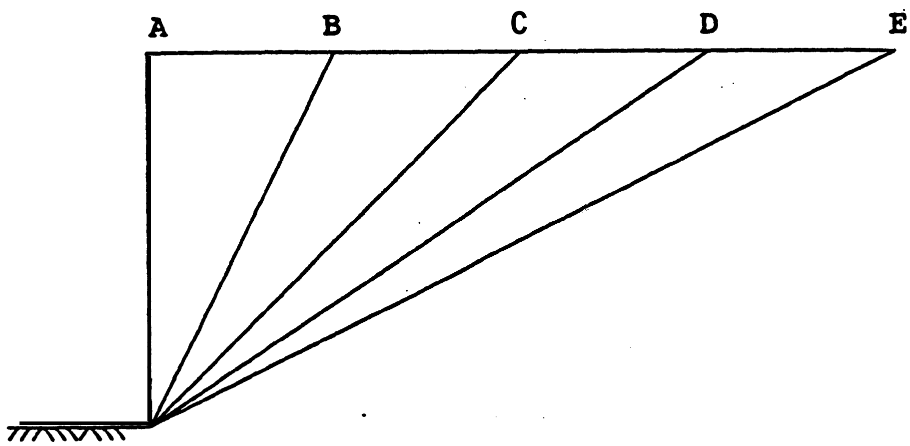
FIGURE 8

APPROXIMATE ANGLES OF REPOSE FOR SOILS

Soils will stand at some natural slope, the angle of repose, unless acted upon by some external force, or unless it is subjected to an internal change of composition, such as a change in water content. FIGURE 9 depicts some repose angles for various materials.

A slope of 1:l corresponds to Type B soil per CAL/OSHA.
A slope of 1.5:l corresponds to Type C soil per CAL/OSHA.

A running soil by Cal/OSHA defintion would have a repose angle of less than 2:l.



A. Solid Rock, Shale or Cemented Sand and Gravels ...... (90°)

B. Compacted Angular Gravels .... (1/2:1)

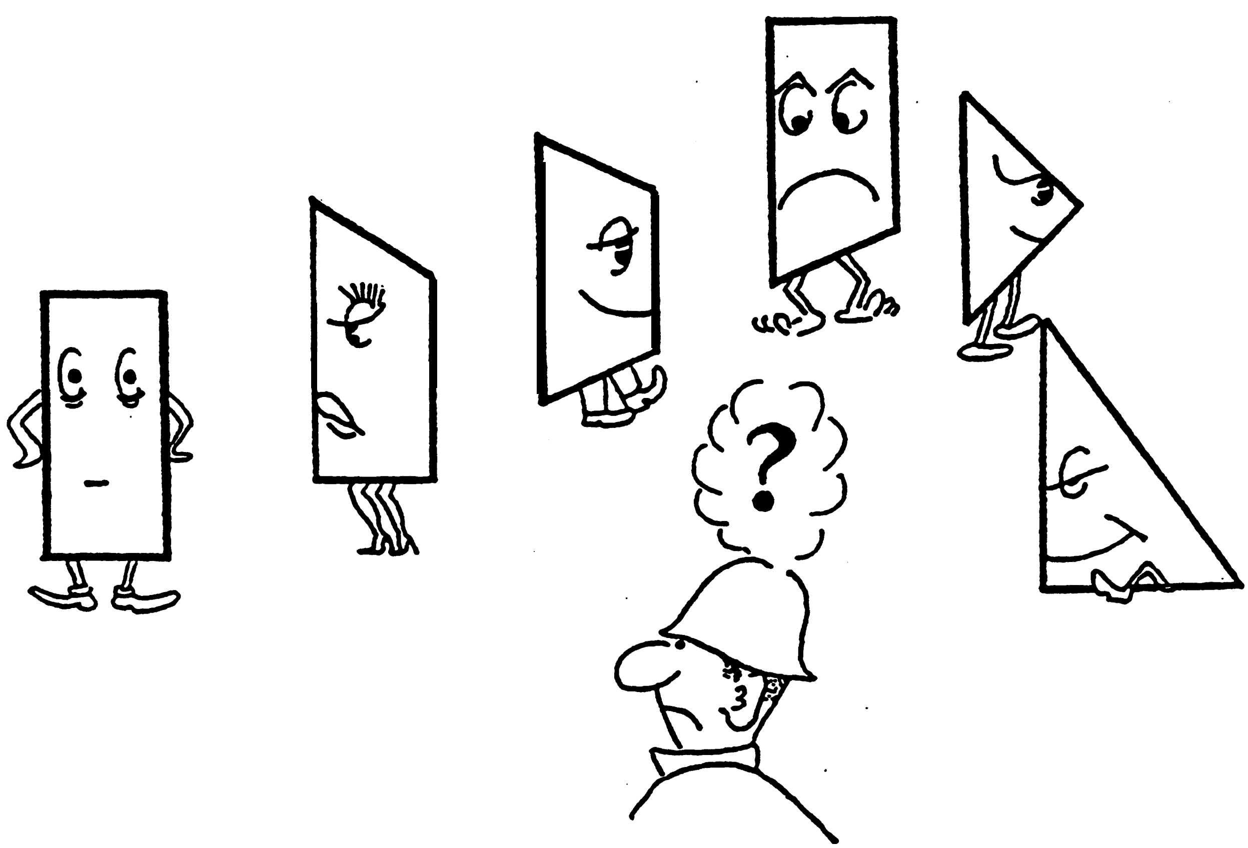
C. Recommended Slope for Average Soils .... (1:1)

D. Compacted Angular Sand .... (1.5:1)

E. Well Rounded Loose Sand .... (2:1)

FIGURE 9

DISTRIBUTION OF LATERAL SOIL PRESSURE

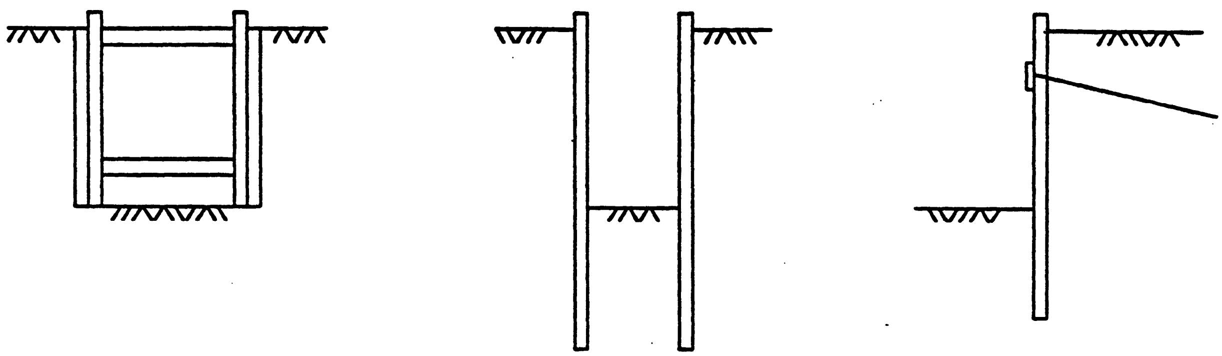


DISTRIBUTION OF LATERAL SOIL PRESSURE

DEVELOPMENT OF LATERAL EARTH PRESSURE DISTRIBUTION

The resultant total earth pressure is determined by application of soils mechanics formulas and procedures. This pressure is represented by a lateral earth pressure diagram. External loadings which affect total lateral pressures must be considered. External loads consist of surcharges and hydrostatic pressure. The design pressure diagram will be a summation of the basic soil pressures, surcharges, and hydrostatic pressure.

The type of shoring has to be identified. The shape of the soil pressure distribution diagram depends upon the type of soil to be encountered and the amount of shoring movement that can be permitted. A shoring system can be restrained fixed, or flexible.



Open Strutted Trench RESTRAINED

Cantilevered Sheet Piling or Soldier Pile System FLEXIBLE

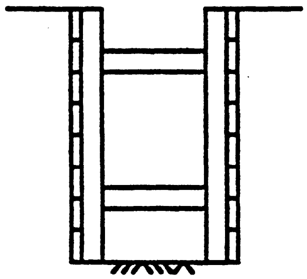
Tieback System FLEXIBLE or RESTRAINED depending upon movement that is permitted.

FIGURE 10

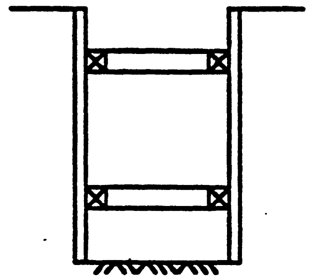
The sequence of work may alter the shape of the pressure diagram during the various construction phases. For example, a tieback sheet pile wall converts from a cantilever to a flexible restrained systemwhen the tiebacks are stressed.

A true fixed system is unusual in shoring work. No movement of the earth retained can occur in a fixed system. An example of a fixed system would be a concrete or concrete slurry wall with tiebacks locked off at a value in excess of design load which causes the wall to exert pressure on the contained soil. This complex type of shoring has been used for excavations for large buildings adjacent to existing structures.

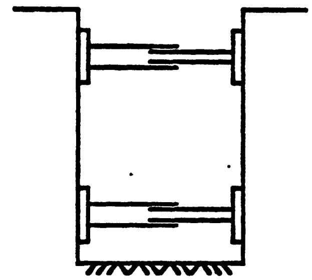
TYPICAL SHORING SYSTEMS



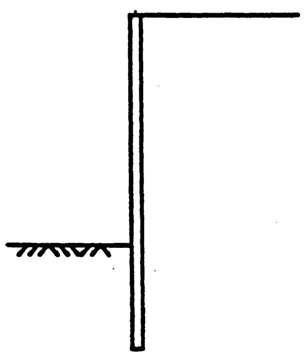
STRUTTED VERTICAL MEMBERS WITH HORIZONTAL LAGGING



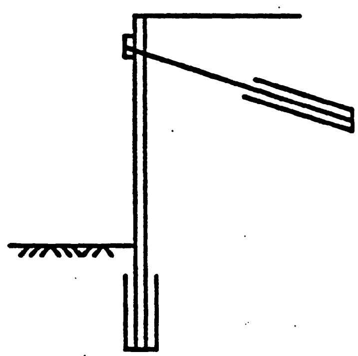
STRUTTED HORIZONTAL WALES WITH VERTICAL SHEETING



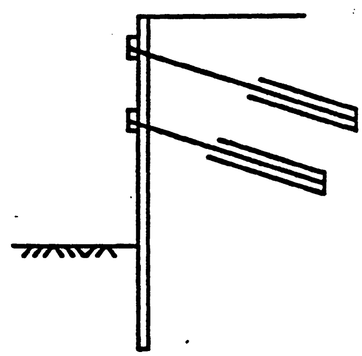
ISOLATED STRUTS (MANUFACTURED SHORING SYSTEMS)



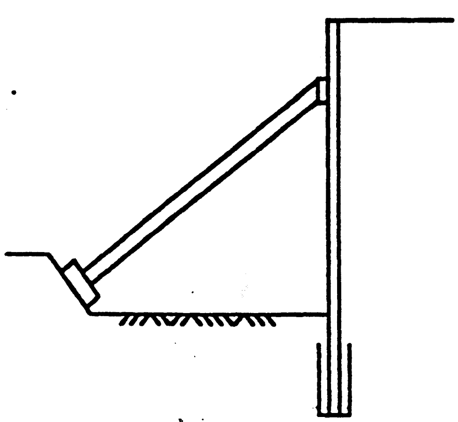
CANTILEVER DRIVEN SOLDIER PILES OR SHEET PILES



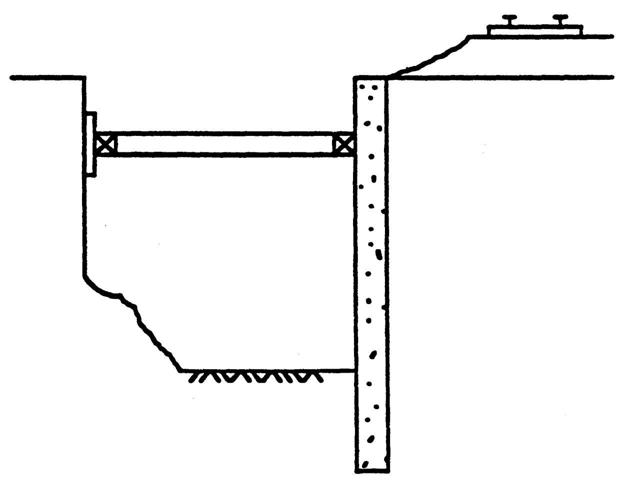
SOLDIER PILE WITH TIEBACK



SHEET PILE MULTIPLE TIEBACKS



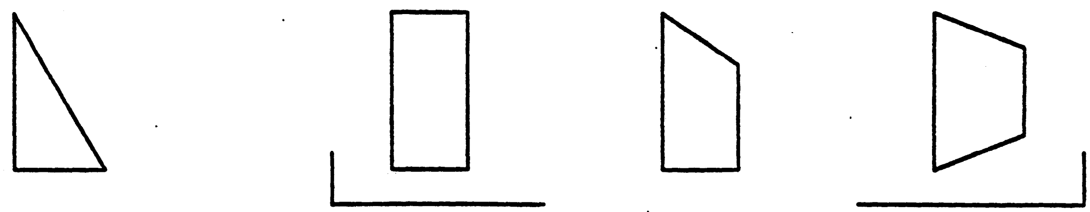
STRUTTED SOLDIER PILE



STRUTTED CONCRETE WALL

DISTRIBUTION OF LATERAL SOIL PRESSURE

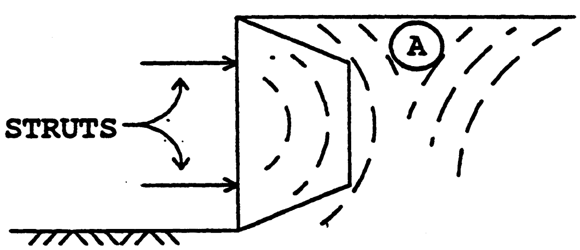
The accepted earth pressure diagram for a cantilever wall is a triangle. The triangle represents the distribution of the equivalent fluid pressure. Triangular pressure diagrams are used only for flexible type shoring systems. For restrained systems it has been determined by research and actual tests that the shape of the earth pressure diagram will approximate a trapezoid.



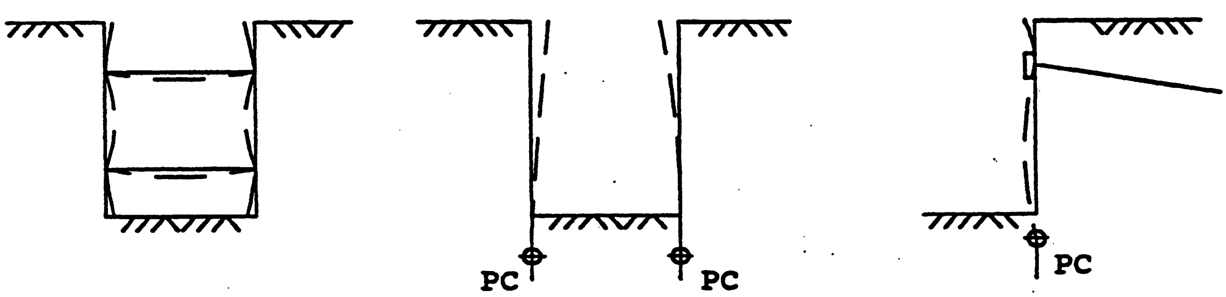
FLEXIBLE RESTRAINED

Tests have shown that the maximum width of the pressure diagram for cohesionless soils (a trapezoid) should be 0.8HKw (Kw = γK_a). For cohesive soils different values and formulas apply.

In a restrained system the upper or top struts will often carry the greatest load. Strutted or anchored walls, built from the top down, have high upper strut loads due to deflection which occurs during excavation below a strut. Shoring deflection causes load transfer up to non-yielding strutted zones. The soil within the zone adjacent to the upper struts crowds together or interlocks tighter, exerting a greater pressure on the upper struts. The concept is illustrated as follows:



In the arching zone at A, wedging occurs and the material in the zone is compressed.



RESTRAINED FLEXIBLE APPROACHES RESTRAINED

The previous diagrams show the mode of failure or movement for various types of systems. Note that for a flexible system the arching will not take place and the active earth pressure is properly taken as an equivalent fluid with a triangular pressure distribution. The location of the apparent point of fixity, PC, (sometimes referred to as the point of contraflexure) will be affected by the type of soil and configuration of the system.

Lateral soil pressure is normally thought of as increasing uniformally with depth. The first pressure diagram conceptualized is a triangle. However, a triangle pressure diagram configuration is generally used only for flexible support systems. A variety of other pressure diagrams are more appropriate for other than flexible conditions and the selected choice will depend upon soil type and the designed system.

Geotechnical authors differ on theories for shape of the lateral pressure distribution. It may be necessary to compare different methods to confirm that the method used by the designer adequately fits the conditions, and will provide for changing conditions as the work progresses.

Occasionally, the submitted design will be based on an equivalent Fluid pressure parameter (Kw) and may or may not include information about internal soil friction angles (ϕ) or the unit weight (γ) of the soil. Comparisons can be made between selected pressure diagrams based on total lateral pressure as well as the use of Kw in lieu of known soil parameters.

Alternate soil pressure diagrams may be related to the common trapezoidal diagram with pressure coefficients 'normalized' so that total lateral pressures are equal. Examples of 'normalized' pressure diagram conversions start on page 5-6.

Cohesive soil pressure diagrams are not used unless the soil contains more than 50% clay, regardless of the type of shoring system to be used.

Some of the more commonly accepted soil pressure diagrams are included in this chapter following the 'normalized' pressure diagrams. Little is done in this manual relative to sheet piling systems. The subject of sheet piling systems is more than adequately covered in the USS Steel Sheet Piling Design Manual.

DISTRIBUTION OF LATERAL SOIL PRESSURE

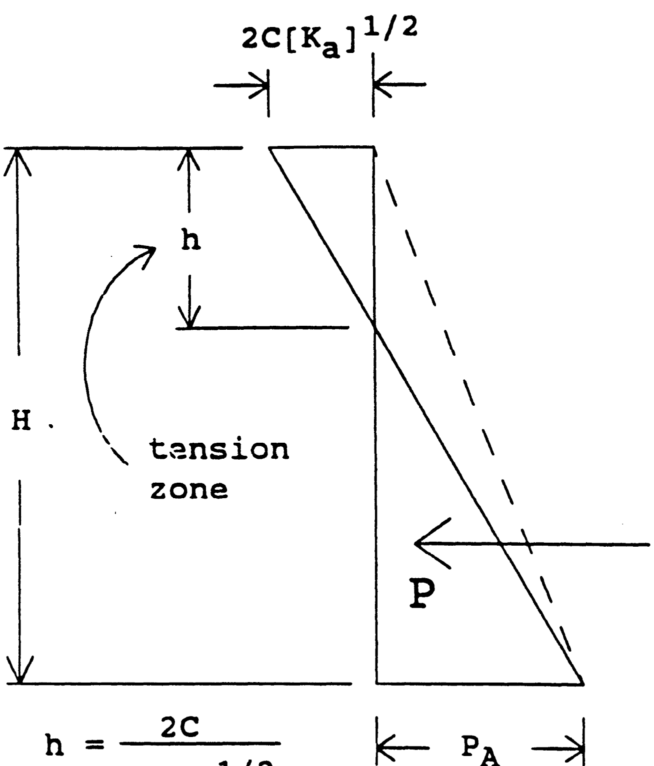
GENERAL EQUATIONS FOR ACTIVE AND PASSIVE CONDITIONS

ACTIVE LATERAL EARTH PRESSURE

Active lateral earth pressure is that horizontal force P_A exerted by the soil upon the shoring system. $P_A = (f)P_V$. P_V is normally assumed to be the weight of the overburden and (f) is some constant function.

General Equation:

$$P_A = \gamma H \tan^2(45° - \phi/2) - 2C\tan(45° - \phi/2)$$
$$= \gamma H K_a - 2C[K_a]^{1/2}$$



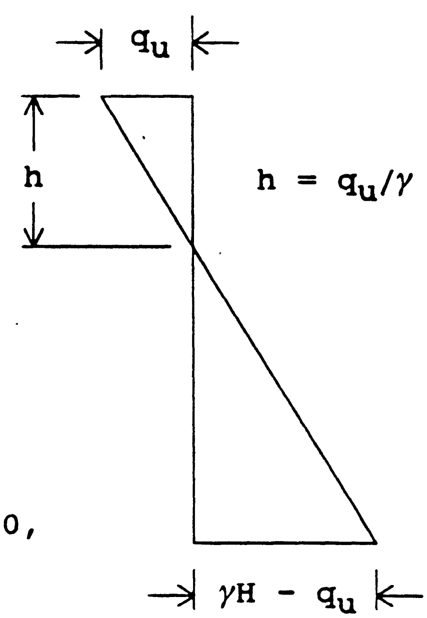
$$h = \frac{2C}{\gamma[K_a]^{1/2}}$$

(Equation does not consider effects of friction or sloping embankment). The tension zone is neglected by conservatively setting h=0 (dashed line).

The horizontal pressure at any plane at depth H is:

$$P_A = (f)P_V + \text{Constant}$$
$$P_A = (f)P_V + (f)C$$
$$P = \frac{P_A(H)}{2}$$

NOTE: For sheet piling Teng uses:



Special Conditions:

Granular Cohesionless Soils When C = 0,

$$K_a = \tan^2(45° - \phi/2) = \frac{1 - \sin\phi}{1 + \sin\phi} < 1$$

Frictionless Cohesive Soils When $\phi = 0$,

$K_a = 1$: $P_A = \gamma H - 2C = \gamma H - q_u$

For Strutted Walls and Cofferdams when $\phi = 0$,

$K_a = 1$: $P_A = \gamma H - 4C = \gamma H - 2q_u$

PASSIVE LATERAL EARTH PRESSURE

Passive lateral earth pressure is that horizontal force P_P that the shoring system exerts on the soil. This horizontal pressure tends to induce vertical expansion of the soil. $P_P = (f)P_V$

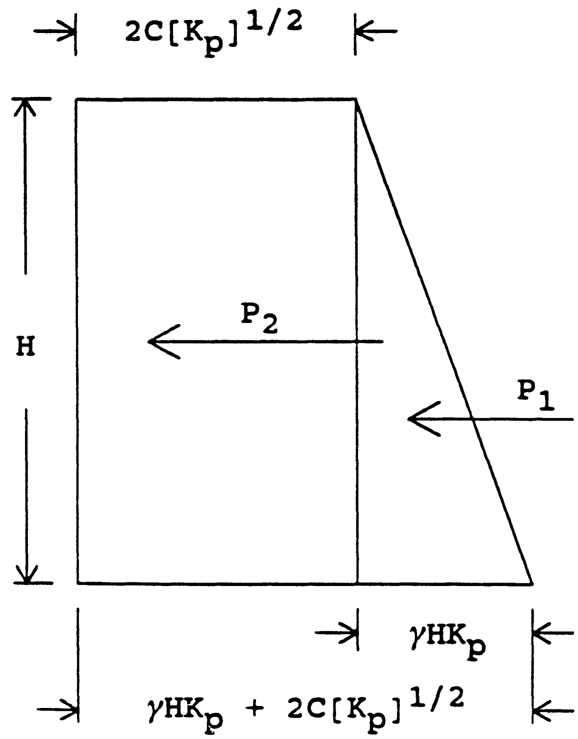
General Equation:

$$P_p = \gamma H \tan^2(45° + \phi/2) + 2C\tan(45° + \phi/2)$$

$$P_p = \gamma H K_p + 2C[K_p]^{1/2}$$

$$P_1 = 1/2(\gamma H^2 K_p)$$

$$P2 = 2CH[K_p]^{1/2}$$



Special Conditions:

Granular Cohesionless Soils when C = 0

$$K_p = \tan^2(45° + \phi/2) \geq 1$$

For the special condition where there is no sloping surcharge and the wall friction angle is assumed as zero:

$$K_p = 1/K_a \geq 1$$

Frictionless Cohesive Soils
When $\phi = 0$, $K_p = 1$

$$P_p = \gamma H + 2C = \gamma H + q_u$$

K_w COEFFICIENT

Kw, the equivalent fluid soil pressure is a very useful concept. By definition: $Kw = K_a\gamma$.

K_a can be determined if Kw and γ are furnished or known.

$$K_a = Kw/\gamma$$

Soil unit weights vary from 85 pcf to about 130 pcf maximum (See **TABLE 11**), the latter occurring in dense gravels. A good average value for a cohesionless soil is 115 pcf.

K_a can be determined by various means: It may be part of the soils report, calculated by either the Rankine or Coulomb method, be determined by the Log-Spiral concept, etc.

If K_p (passive coefficient) is not given in the soils report, and cannot be adequately estimated by using **FIGURE 8**, the following estimation may be used for approximately level surfaces:

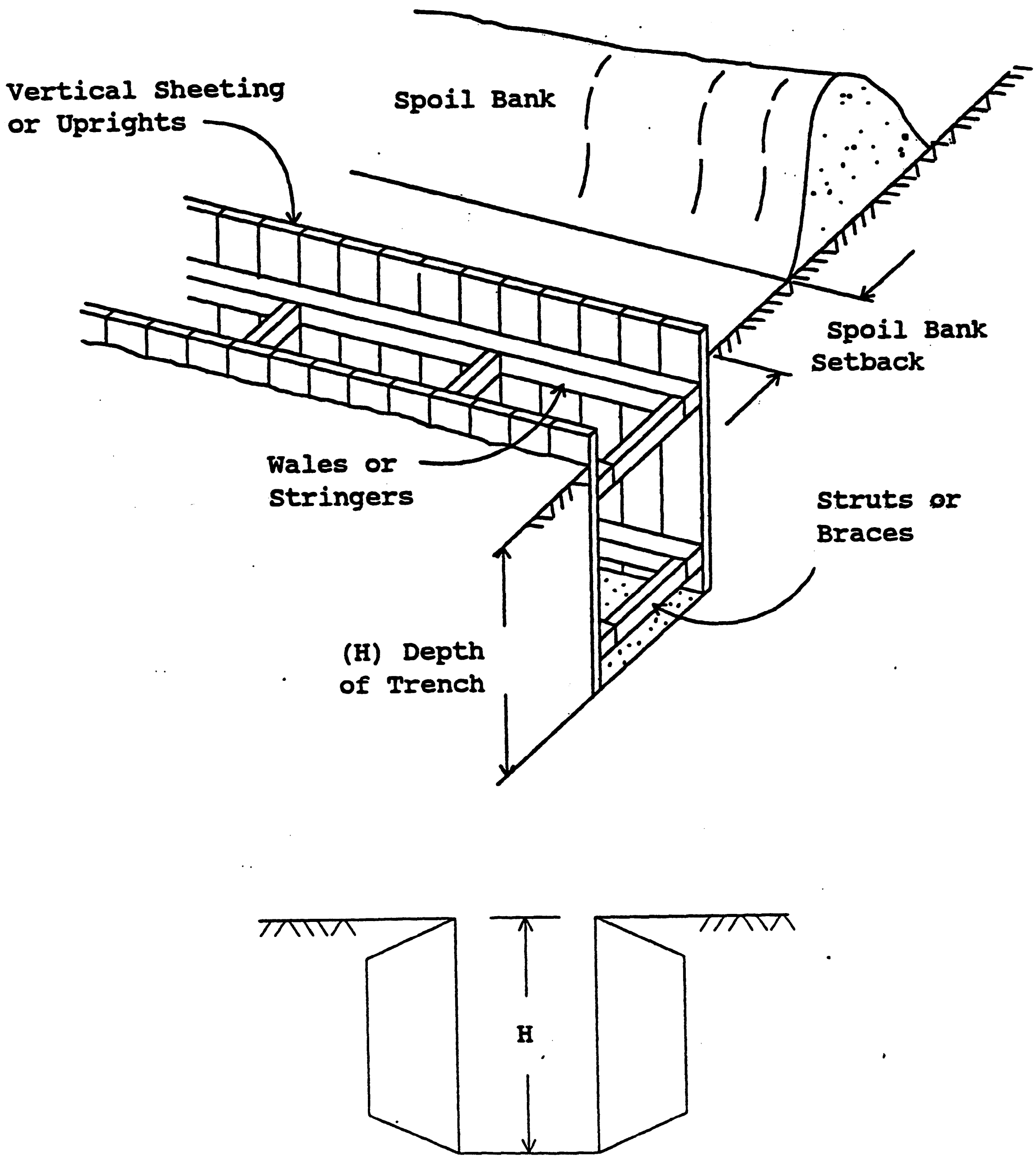
$$K_p = 1/K_a \quad (2.5 < K_p < 5.5)$$

Kw values permit the plotting of lateral pressures (pressure diagrams) so that pressure areas can be determined.

K_a (Active coefficient) and K_p (Passive coefficient) are needed for sheet piling and soldier pile system calculations.

For approximately level surfaces when the cohesion value C = O, K_a can be assumed to be equal to $\tan^2(45^\circ - \phi/2) = \dfrac{1 - \sin\phi}{1 + \sin\phi}$; and K_o may be assumed as $1 - \sin\phi$.

OPEN-STRUTTED TRENCH



Earth Pressure

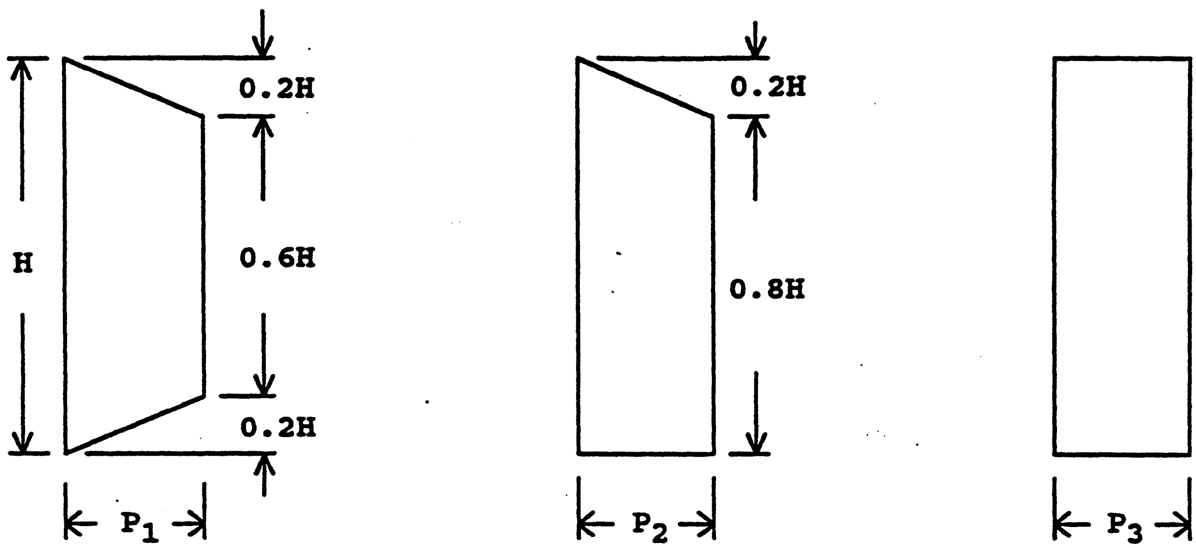
FIGURE 11

DISTRIBUTION OF LATERAL SOIL PRESSURE

EXAMPLE OF EQUIVALENT LATERAL PRESSURE DISTRIBUTION

Basic soil diagrams for cohesionless & equivalent cohesion soils

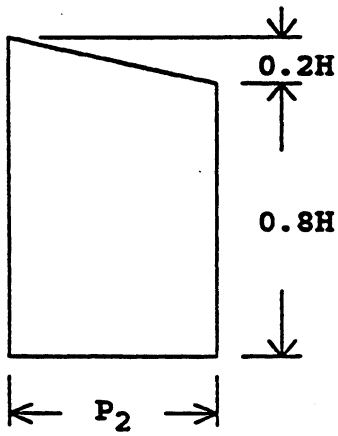
STRUTTED (RESTRAINED SYSTEM)



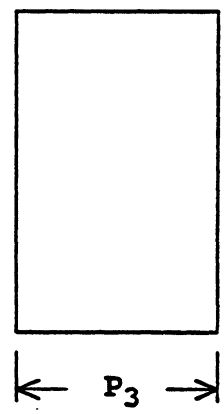
$P_1 = P_A = 0.8KwH$	$P_2 = P_A = 0.71KwH$	$P_3 = P_A = 0.64KwH$
Use for H > 10' (medium to stiff equivalent cohesive)	**Use for H > 10' (Sheet pile and soldier pile systems, soft to medium equivalent cohesive)**	**Approximation: May be used for H ≤ 10' (All soils)**
Total Lateral Force: $P = 0.8P_1H$	**Total Lateral Force:** $P = 0.9P_2H$	**Total Lateral Force:** $P = 1.0P_3H$

Total lateral pressure for the 3 diagrams shown above are the same. Equivalent P_A values were computed as shown on the next sheet, so that the resultant total pressures equate to the standard trapezoidal diagram (above left).

Normalizing the Kw to the standard trapezoidal loading diagram for braced excavations in cohesionless soil permit direct comparisons of the various earth diagrams and support systems.



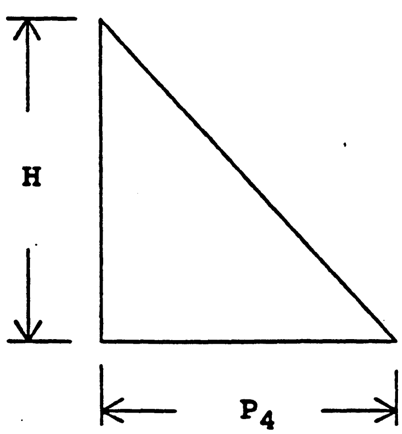
$P = 0.9P_2H = 0.8P_1H$
$P_2 = (0.8/0.9)P_1$
$P_2 = (0.8/0.9)0.8KwH$
$P_2 = 0.71KwH$



$P = P_3H = 0.8P_1H$
$P_3 = 0.8P_1$
$P_3 = (0.8/1.0)0.8KwH$
$P_3 = 0.64KwH$

FLEXIBLE SYSTEMS

Cantilever sheet pile or soldier pile systems, as well as tieback systems with one tier of ties, are considered flexible systems.



$P = 0.5P_4H$
$P_4 = 1.0KwH$

If it is desired to compare a triangular shape to the trapezoidal diagram use:

$P = 0.5P_4H = 0.8P_1H$
$P_4 = (0.8/0.5)P_1$
$P_4 = (0.8/0.5)0.8\ KwH$
$P_4 = 1.28KwH$

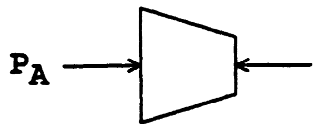
Note that the total force of the various areas are equal. If total lateral force is calculated by other means (Rankine, Coulomb, Log Spiral, etc.), solve for P_A (and equivalent Kw, if needed).

BRACED OPEN CUTS IN SAND

Active pressure for Cohesionless Soil

P_A = $0.8K_a\gamma H\cos\delta = 0.8KwH\cos\delta$.

P_A = The maximum ordinate of the earth pressure diagram (psf).



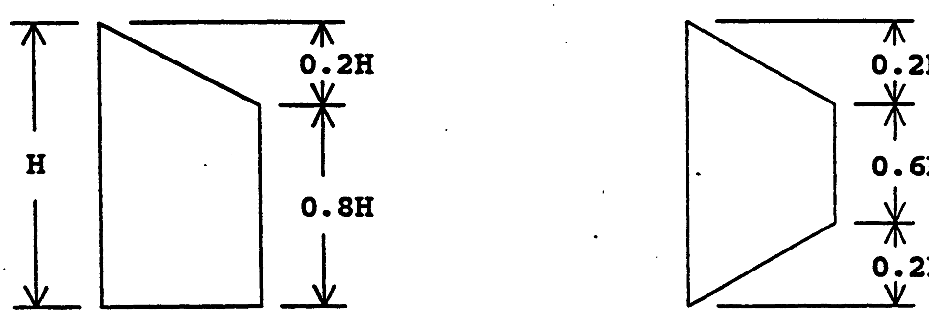
K_a = The active earth pressure coefficient. Determined by calculation (Rankine, Coulomb, Log-Spiral) or tabular value.

γ = Unit weight of the soil.

δ = Wall friction angle.

Terzaghi & Peck:

Loose Sands — Dense Sands

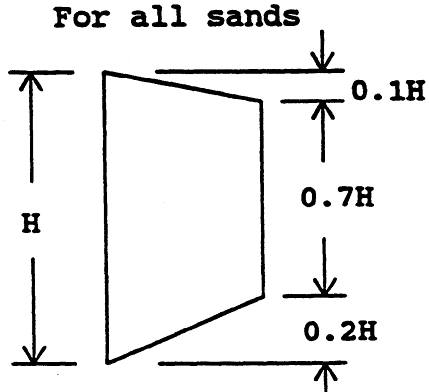


$P_A = 0.8KwH\cos\delta$ — $P_A = 0.8KwH\cos\delta$

FIGURE 12

Tschebotarioff:

For all sands



$P_A = 0.8KwH\cos\delta$

FIGURE 13

Normalize calculation:

Total force

$(0.1)(H)(P_A)/2$	$= 0.05\ P_AH$
$(0.2)(H)(P_A)/2$	$= 0.10\ P_AH$
$P_A(H - 0.1H - 0.2H)$	$= \underline{0.70}\ P_AH$
P	$= 0.85\ P_AH$

To convert from standard trapezoidal loading:

$P = (0.8KwH)(0.8H) = 0.64KwH^2$

Equate total forces

$0.85P_A = 0.64KwH^2$

$P_A = (0.64/0.85)KwH = 0.75KwH$

BRACED OPEN CUTS IN CLAY

Determination of the lateral pressures for cohesive soils is more. difficult than for cohesionless materials and in some cases comparative calculations have to be made.

The general equation for active lateral pressure for clay is as follows:

$$P_A = \gamma H K_a - 2C[K_a]^{1/2}$$

For temporary works limited to strutted trench shoring systems, the equation has been symbolically modified on the basis of field tests to the following:

$$P_A = \gamma H K_a - 4C[K_a]^{1/2}$$

Pure clay soils have no angle of internal friction (ϕ).

K_a = 1.00 and the formula reduces to:

$$P_A = \gamma H - 4C$$

It is a conservative approach to consider no angle of internal friction when investigating or designing shoring systems in Clays. However, if soils investigations indicate otherwise, K_a may be modified accordingly, For a soil to be classified as a clay, it must contain at least 50% pure clay, If it does not meet this criteria, then an appropriate cohesionless soil pressure formula and pressure diagram should be used.

Another characteristic of clays is that properties, such as cohesion and moisture content will change appreciably when the clay is exposed for extended time periods. The cohesive strength will decrease and the material will approach a cohesionless soil condition. A time period longer than one month would be considered an extended period for trench or other shoring work.

It is possible to get negative values in the basic clay formulas, Initially clays can stand unsupported to some depth. This depth is called the critical depth within the critical depth limit an active lateral pressure may or may not exist depending on other conditions, such as groundwater. For design or analysis of earth pressure systems it is not acceptable to use negative pressure. Other formulas have been developed which will always give a positive pressure value at any depth. The controlling design pressure is then determined by making comparative calculations. Negativevalues are not to be used.

The shape of the earth pressure diagram for clays varies with different authors.

Following are some theories:

Terzaghi & Peck

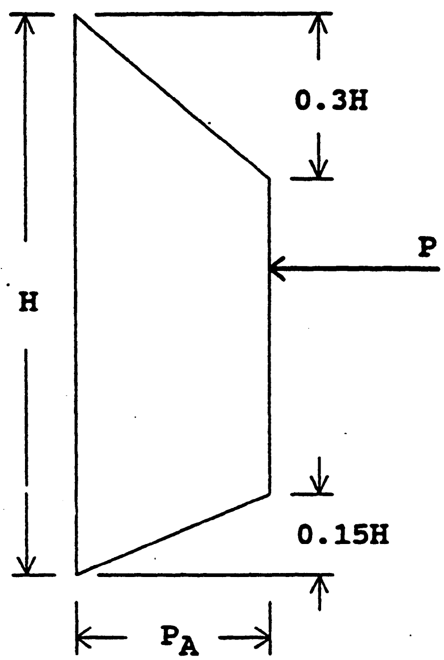


FIGURE 14

$P_A = \gamma H - 4C$
(Do not use negative value)

OR

$P_A = \gamma kH$
Use k = 0.375 for soft clays
(k = 0.300 for stiff clays)

Use whichever controls

Total Lateral Force:

$1/2(0.3H)(P_A)$	$= 0.15\ P_AH$
$1/2(0.15H)(P_A)$	$= 0.075\ P_AH$
$P_A(H-0.3H-0.15H)$	$= 0.55\ P_AH$
P	$= 0.775\ P_AH$

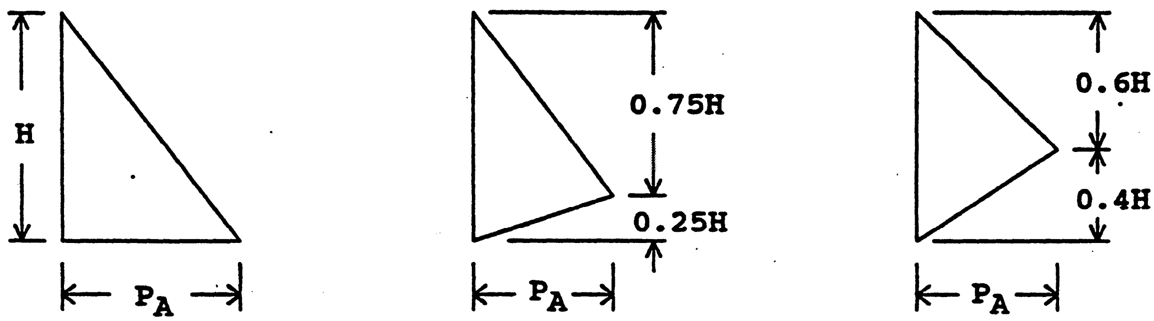
If it is desired to convert from the trapezoidal loading, use the following:

Normalize (0.8/0.775)(0.8) = 0.83

	Soft Clays	Stiff Clays
P_A	$= 0.83KwH = 0.375\gamma H$	$= 0.83KwH = 0.30\gamma H$
Kw	$= (0.375/0.83)\gamma$	$= (0.30/0.83)\gamma$
Kw	$= 0.452\gamma$	$= 0.361\gamma$

<u>Tschebotarioff</u>

This author takes advantage of the ability of stiff clays to stand initially without support; a temporary condition.



P_A	$= 0.50\gamma H$	$= 0.375\gamma H$ to $0.50\gamma H$	$= 0.30\gamma H$ to $0.5\gamma H$
	Soft Plastic Clays (Approaching true liquid)	**Medium Clays**	**Stiff Clays** (Temporary condition)

FIGURE 15

If it is desired to convert from the trapezoidal loading, use the following:

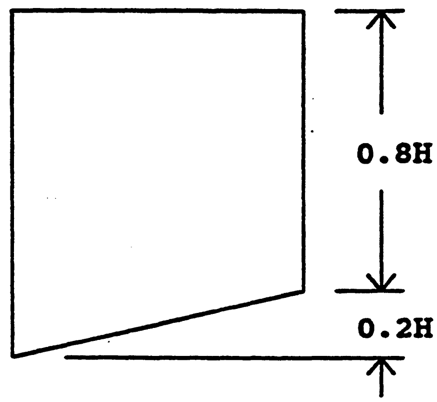
$$\text{Total Force:}\quad P = 1/2P_AH$$

$$\text{Normalize:}\quad (0.8/0.5)0.8 = 1.28$$

P_A	$= 0.50\gamma H$	$= 0.375\gamma H$	$= 0.30\gamma H$
P_A	$= 1.28KwH$	$= 1.28KwH$	$= 1.28KwH$
Kw	$= (0.5/1.28)\gamma$	$= (0.375/1.28)\gamma$	$= (0.30/1.28)\gamma$
Kw	$= 0.39\gamma$	$= 0.29\gamma$	$= 0.23\gamma$

FHWA-RD-75-128 LATERAL SUPPORT SYSTEMS AND UNDERPINNING

Proposed pressure diagram for internally braced shoring-dense cohesive sands, stiff sandy days.



$P_A = 0.15\gamma H$ to $0.25\gamma H$

FOR UPPER 1/3H DOMINATED BY COHESIONLESS SOIL

FIGURE 16

If it is desired to convert from the trapezoidal loading, use the. following:

Total force: $P = P_A(0.8H + H)/2 = 0.9\ P_A H$

Normalize: $(0.8/0.9)0.8 = 0.71$

	For $P_A = 0.15\gamma H$:	For $P_A = 0.25\gamma H$:
P_A	$= 0.71KwH$	$= 0.71KwH$
Kw	$= (0.15/0.71)\gamma$	$= (0.25/0.71)\gamma$
Kw	$= 0.21\gamma$	$= 0.35\gamma$

STABILITY NUMBER METHOD

Another means of determining the lateral pressure P_A and the shape of the pressure diagrams by the Stability Number Method. This method will always give positive values and is acceptable to any depth. Another advantage of the Stability Number Method is that it provides an indicator of when the problem of bottom heave should be investigated. Heave is possible when the Stability Number (N_o) is greater than 6.

$N_o = \gamma H/C$ (but not greater than 20)

$P_A = C/150\ (7\ N_o^2 + 10\ N_o)$

$A = 0.3\ (1-N_o/20)\ H$, but $\leq 0.15\ H$

$B = 1.1\ (1-N_o/20)\ H$, but $\leq 0.55\ H$

P_H = Total Pressure

Do not use negative pressure values. The formulas above are generally accepted for P_A, A, and B.

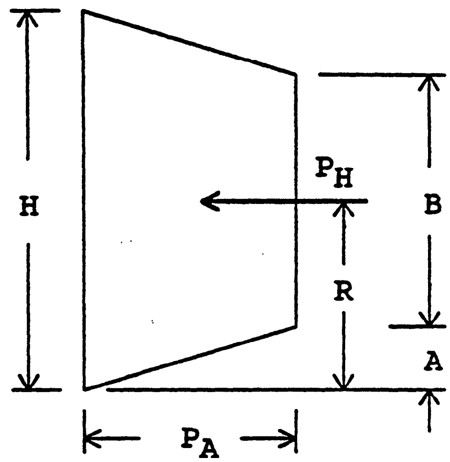


FIGURE 17

Ref: See USS Steel Sheet Piling Design Manual, P. 58.

Modifications bv another source follows:

NAVDOCKS DM-7 (U.S. Navy Engineer Corps)			
	$2 < N_o < 5$	$5 < N_o < 10$	$10 < N_o < 20$
P_H	$0.78 P_A H$	$0.78 P_A H$	$(2.1 - 0.55 N_o) P_A H$
P_A	$\gamma H - 1.5(1+N_o)C$	$\gamma H - 4C$	$\gamma H - (8-0.4 N_o)C$
A	0.15H	0.15H	$(0.3 - 0.015 N_o)H$
B	0.55H	0.55H	$(1.1 - 0.055\ N_o)H$
R	0.46H	0.46H	0.38H

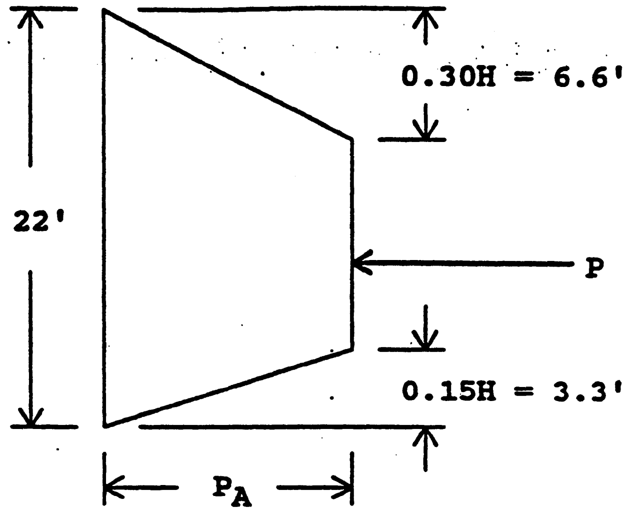
TABLE 17

SAMPLE PROBLEM No. 1 - Strutted Trench (Restrained System)

Known Properties:

SOFT CLAY
$H = 22'$
$\gamma = 120$ pcf
$q_u = 800$ psf
$C = q_u/2 = 400$ psf

Use pressure distribution per Terzaghi & Peck for cohesive soil.



General Equation
$P_A = \gamma H - 4C = (120)(22) - (4)(400) = 1040$ psf ⟵ CONTROLS

Solution by Terzaghi & Peck.
$P_A = 0.35\gamma H = (0.35)(120)(22) = 924$ psf

Solution by Tschebotarioff.
$P_A = 0.5\gamma H = (0.5)(120)(22) = 1320$ psf

SAMPLE PROBLEM No. 2 - Strutted Trench (Restrained System)

Known Properties:

SOFT CLAY
$H = 11'$ $q_u = 800$ psf
$\gamma = 120$ pcf $C = q_u/2 = 400$ psf

General Equation
$P_A = \gamma H - 4C = (120)(11) - (4)(400) = -280$ psf

The answer is negative, indicating that H of 11' is less than the critical height of this clay. The clay will stand unsupported for a short time, but is subject to change because of the effect of weather on exposed surface, creep in the clay, loss of cohesion, dynamic load effects, etc. For this reason negative pressure values will not be used.

Solution by Tschebotarioff.
$P_A = 0.5\gamma H = (0.5)(120)(11) = 660$ psf ⟵ CONTROLS

SAMPLE PROBLEM No. 3 - Strutted Trench (Restrained System)

Known Properties:

CLAY $H = 16'$ $q_u = 0.5$ tsf $\gamma = 110$ pcf

$\therefore$ $C = q_u/2 = 500$ psf $N_o = \gamma H/C = (110)(16)/500 = 3.52$

Solution by Stability Number Method.

$$P_A = (C/150)(7N_o^2 + 10N_o) = (500/150)\{(7)(3.52)^2 + (10)(3.52)\}$$
$$= 406 \text{ psf}$$

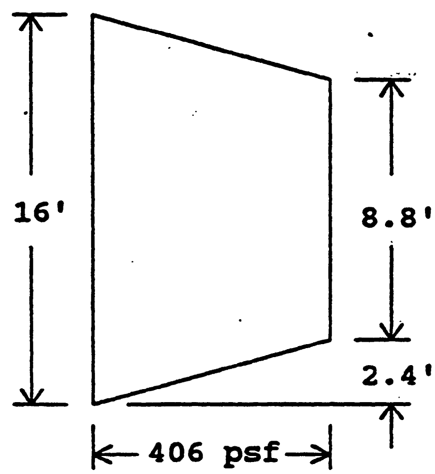
Dimensions of Pressure Diagram:

$$B = (1.1)(1 - N_o/20)$$
$$= (1.1)(1 - 3.52/20)(H)$$
$$= 0.906H > 0.55H$$

$\therefore$ Use .55 H

$$A = (0.3)(1 - N_o/20)$$
$$= (0.3)(1 - 3.52/20)(H)$$
$$= 0.247H > 0.15 H$$

$\therefore$ Use .15 H



Note that the value of P_A would be considerably higher if $0.3\gamma H$ had been used. (0.3)(110)(16) = 528 psf, and the pressure diagram would be drawn differently. This illustrates the point that different answers may be obtained by using an alternate acceptable analysis.

When comparing Sample Problems 1 and 2 to Problem 3 it is noted that the highest calculated lateral earth pressure was used in the former problems but not in the latter: the reader should be made aware that the degree of accuracy is often more-dependent on proper estimates of soil strength parameters than on the method used for calculation of lateral earth pressure.

CRITICAL HEIGHT OF CLAY

Definition:

1. Maximum height at which material will stand without support.*
   This will generally be a short term condition.

2. The depth of potential tension cracks in cohesive material.**

   From the general equation for active pressure:

   $P = K_a \gamma h - 2C[K_a]^{1/2}$

   $P = K_a - 2C/\gamma h[K_a]^{1/2}$

Assuming unchanging conditions, and the material is unsupported laterally, then P = 0.

$H_c = h = 2C/\gamma[K_a]^{1/2}$ Where H_c equals the critical height of clay.

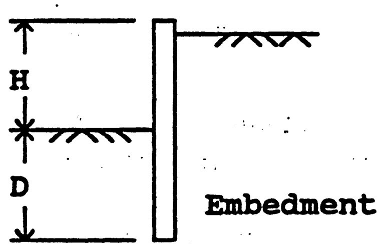
If $\phi = 0$, then $K = 1$, and $H_c = 2C/\gamma$

If $C = 0$, then $H_c = 0$

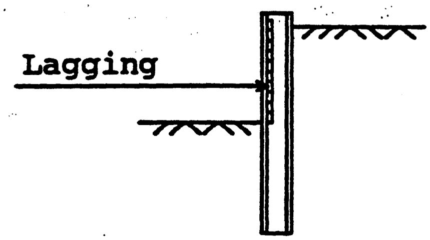
* This does not mean excavations will not require shoring. Changing conditions alter characteristics of clay. Clayey soils may crack but remain standing.

** Note that cracks can fill with water causing additional lateral pressures.

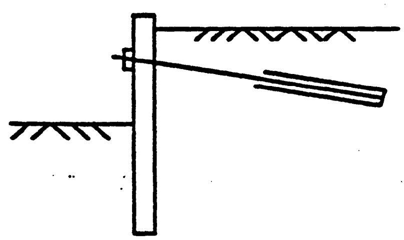
FLEXIBLE OR YIELDING SYSTEMS



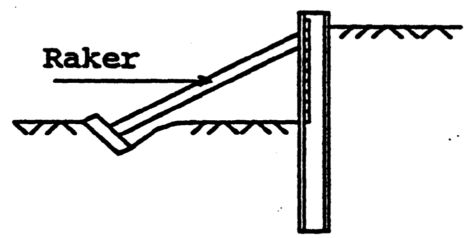
Cantilevered Steel
Sheet Pile



Soldier Pile
System



Single Tier
Tieback System



Soldier Pile
with Raker

Flexible Systems have a different distribution of earth pressure as compared to a restrained system. The material will approach an equivalent fluid and the correct diagram for active lateral pressure will be a triangle. Passive pressures now have to be considered for the portion of the system embedded in the ground. Steel-sheet piles, or soldier piles, are installed in to the ground a sufficient distance below the bottom of the excavation to utilize passive pressures.

Walls designed as pure cantilevers undergo large lateral deflections. Walls may be subject to scour and erosion. Member stresses and movement increase quite rapidly with. height causing required pen&ration depths to become quite high relative to the height of the wall. Cantilevered sheet pile walls for shoring systems are therefore usually restricted to moderate heights of less than 15' However, very heavy sheet pile sections are now available (see TABLE 19 'Sheet Pile Sections' in Chapter 8).

The examples in the USS Steel Sheet Piling Design Manual are recommended for cantilever and braced cantilever systems. A few general considerations are included at the end of this section.

Following is a general procedure which OSC recommends for determining an acceptable pressure distribution to use for structural analysis of shoring systems.

DISTRIBUTION OF LATERAL SOIL PRESSURE

SHORING: GENERAL PROCEDURE

1. Classify the soil.
   At one extreme would be a large or complicated project for which there is a complete geotechnical soils report which will give all pertinent parameters, description of soil, ground water conditions, andrecommendations for temporary shoring loading. The other extreme is often encountered for relatively small projects such as trenches for pipes along streets or highways - often there is no soils data included with the shoringplans. The reviewing engineer will have to confirm that the soil at the location conforms to Cal/OSHA Type A, B, or C soil (or an equivalentminimum fluid pressure value). This is done by site inspection, test pits, review of other data such as log of test borings for contract or contracts within same area, etc. The less information furnished, the more conservative the review of the shoring plans must be.

2. Determine an equivalent Kw if necessary.

3. Select pressure distribution (pressure diagram).
   This is a function of the type of system: whether flexible, restrained, or in between (see FIGURE 10). Develop the basic soil pressure diagram.

4. Calculate the effect of surcharges.
   The Boussinesq strip loading formula may be the most useful.
   Equivalent surcharge loading for soil slopes above the top of the excavation is a specialized case.
   Railroad loading surcharges require special treatment.

5. Sketch pressure diagrams.
   Compute basic soil pressures. Combine all surcharge loads (including ground water effect if applicable). Simplify the combined diagrams for analysis or design.

6. Apply diagrams to the shoring system and make structural review.
   For normal short duration loading (less than threemonths) an overstress of 1.33 is permitted (except for struts and tiebacks), Overstress allowance should not be used for the following: Cal/OSHA TABLES, high risk areas, over stressing, shoring subject to vibratory loads such as adjacent pile driving, and railroad loading surcharges. Allowable stresses for shoring are included in Chapter 12.

SETTLEMENT AND DEFLECTION

Wall deflections, and soil settlement behind temporary shoring walls are dependent on both wall stiffness and soil strength. Wall stiffness is a function of EI/L^4, and the soil strength can be related to the undrained shear strength.

E = Elastic modulus of the wall.
I = Moment of inertia/foot of wall.
L = Vertical distance between support points.

Ground surface settlement will most often be a maximum directly behind shoring walls. With granular soils, settlement can be expected at a distance '(from the face of the wall) equaling two times the depth of l excavation. For clay soils this distance can be as much as three times the excavation depth. Vertical wall displacements as well as wall deflections' contribute to the amount of settlement.

Maximum lateral displacements for temporary suppport walls can be as much as 0.2% of the wall height for granular soils, and 0.35% of the wall height for cohesive soils.

Horizontal movement of soils under buildings, roads, or other structural components generally cause more damage than vertical displacements.

Tiebackwalls usually experience the samedeformations as internally braced walls in dense cohesive sands or very stiff clays. If deformation of the wall is deemed critical, K_0 should be used for design in lieu of K_a. If settlement will be detrimental, the vertical components of tiebacks should be considered. If wall deflections are considered to be a problem, special consideration will be required for design.

Lagging in soldier pile walls have a tendency to absorb more load as time progresses. Load transfer with time will be more pronounced in cohesive soils. Subsidence may occur behind the wall if poor construction control results in voids behind the lagging. Voids behind the lagging should be backpacked so lagging is effectively tight to the soil.

Construction practices will also have a significant effect on net soil movements. Be aware that large settlement behind shoring could be an indication of bottom heave.

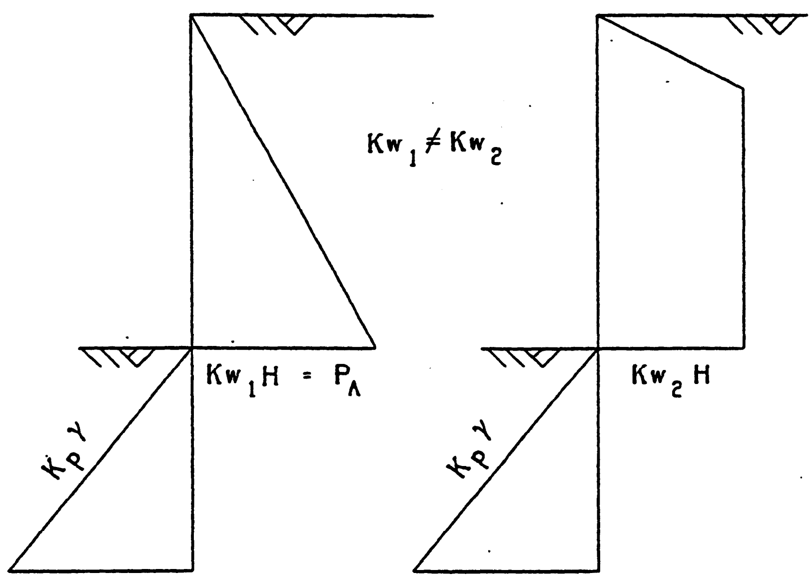
DISTRIBUTION OF LATERAL SOIL PRESSURES

PRESSURE DIAGRAMS IN USE

Selection of the proper pressure diagrams to use for shoring normally rests with the designer. Consultants use a variety of soil pressure diagrams, sometimes depending on recommendations made by professional geotechnical sources.

A common recommendation is that the soil pressure diagram for cantilever members should be a triangle. For single tie back or strut conditions the recommendation may include triangular soil pressure-diagram for the vertical members only (especially for sheet pile type walls), whereas either the same triangular loading diagram or a separate trapezoidal pressure diagram will be recommended for loading the wales, tieback members, or for struts.

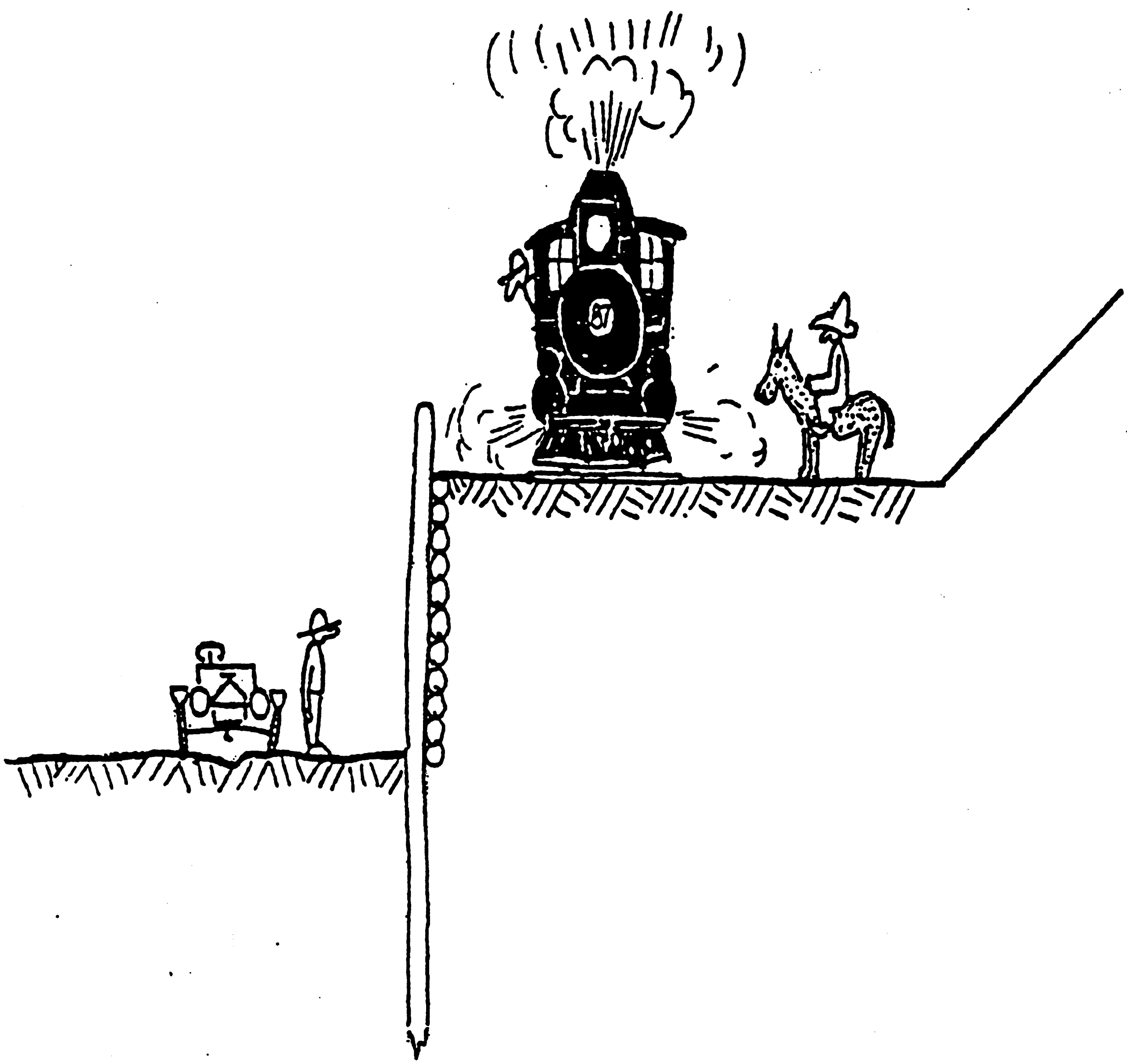
Trapezoidal soil pressure diagrams are generally shown with the active lateral pressures shown as KwH, where Kw equals $K_a\gamma$ pounds per cubic foot. Active values for Kw in common usage vary between 20 to 40 pounds per cubic foot. The selection of Kw values depends on soil characteristics, site conditions, anticipated shoring configuration, and local experience.



Passive lateral pressures may be shown in the form of $K_p\gamma$ pounds per square foot per foot of depth.

When soil pressure diagrams and lateral pressure value recommendations come from geotechnical sources those values may be used for review of the shoring. Verification of the soil characteristics, if furnished, should be made with the log of test borings closest to the site of the planned work.

SURCHARGES

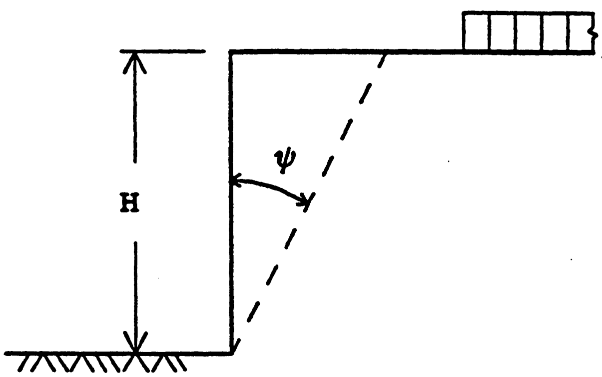


SURCHARGES

SURCHARGE LOADS

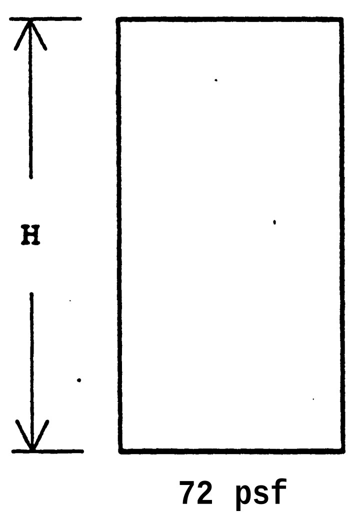
A surcharge load is any load which is imposed upon the surface of the soil close enough to the excavation to cause a lateral pressure to act on the system in addition to the basic earth pressure. Groundwater will also cause an additional pressure, but it is not a surcharge load.

Examples of surcharge loads are spoil embankments adjacent to the trench, streets or highways, construction machinery or material stockpiles, adjacent buildings or structures, and railroads.



A soil surcharge load 4' or less in height will not need to be considered if the load is positioned to the right of the assumed* failure plane as shown. With higher, irregular, or sloping embankments it will be necessary to consider all loads acting on wedges used in the Trial Wedge analysis.

MINIMUM CONSTRUCTION SURCHARGE



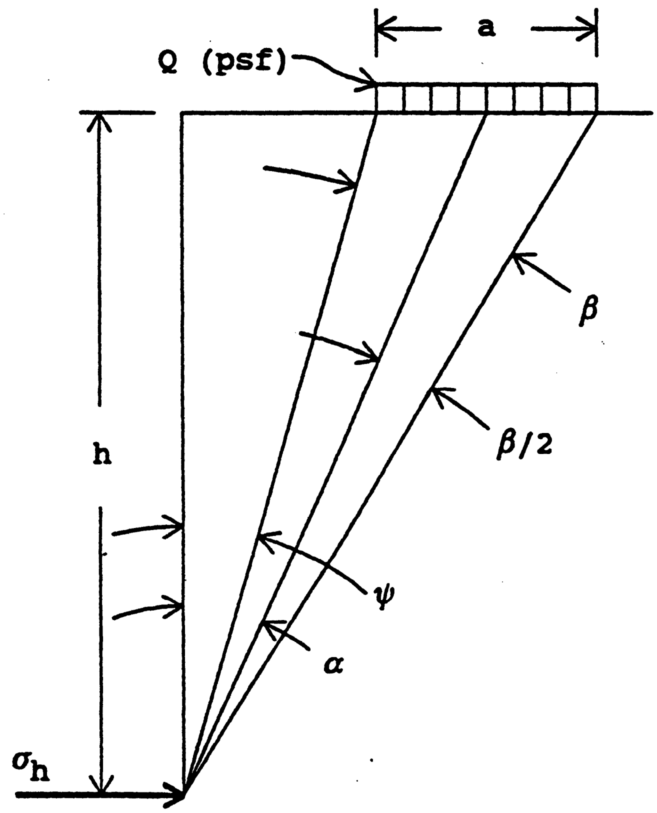
This surcharge load results in a uniform lateral pressure of 72 psf. It shall be used when making an engineering analysis of all types of shoring systems. This surcharge is intended to provide for the normal construction loads imposed by small vehicles, equipment, or materials, and workmen on the area adjacent to the trench or excavation. It should be added to all basic earth pressure diagrams. Thisminimum surcharge can be compared to a soil having parameters of γ = 109 pcf and K_a = 0.33 for a depth of 2 feet [(0.33)(109)(2) = 72 psf]

Surcharge loads which produce lateral pressures greater than 72 psf would be used in lieu of the prescribed minimum.

SURFACE LOADS

Any number of surcharge loads can be added to the soil pressure diagram as long as they are analyzed by a proper and proven method. The Boussinesq Strip is one such method and can be used for all surface surcharge loads (unless the load is treated as a soil embankment).

Boussinesq Strip Method: $\sigma_h = 2Q[\beta_R - (\sin\beta)(\cos 2\alpha)]/\pi$



a = width of surcharge strip

σ_h is the intensity of lateral pressure at distance h below top of the excavation (psf). β_R is in radians.

$$\beta_R = \beta(\pi/180)$$

Q is the surface load (psf).

Note: When the surface load starts at the edge of the excavation:

$$\beta = 2\alpha$$

Pay attention to signs; cosines of angles greater than 90° are negative.

In absence of soil data, the soil failure plane angle (ψ) may be assumed as 35° for level surface conditions only.

There are formulas for line and point load surcharges. See the USS Steel Sheet Piling Design Manual. The strip formula can be used satisfactorily for most situations.

BOUSSINESQ STRIP FORMULA

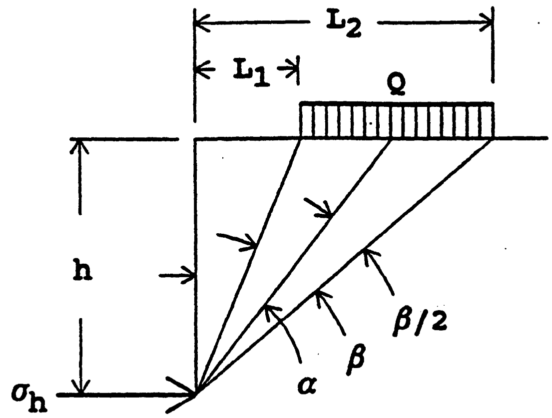


FIGURE 18

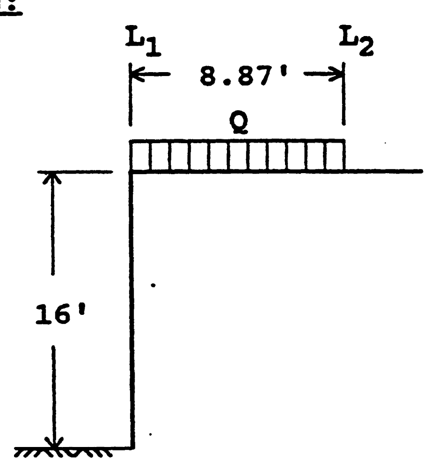
$$\sigma_h = [2Q/\pi][\beta_R - (\sin\beta)(\cos 2\alpha)]$$

When $L_1 = 0$, $\beta = 2\alpha$

Two calculator programs and an extensive table are available at the end of this section for determining lateral pressures due to surcharge.

SAMPLE PROBLEM NO. 4 - BOUSSINESQ STRIP METHOD

GIVEN:



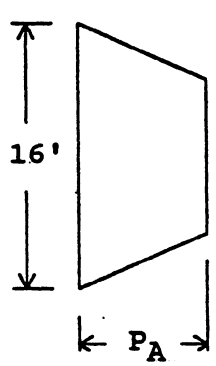
$Kw = 35$ pcf

$Q = 840$ psf

$L_1 = 0$

$L_2 = a = 8.87'$

BASIC SOIL PRESSURE:



Assume for this example that the system is a strutted trench.

$P_A = 0.8KwH = (0.8)(35)(16) = 448$ psf

SURCHARE PRESSURES:

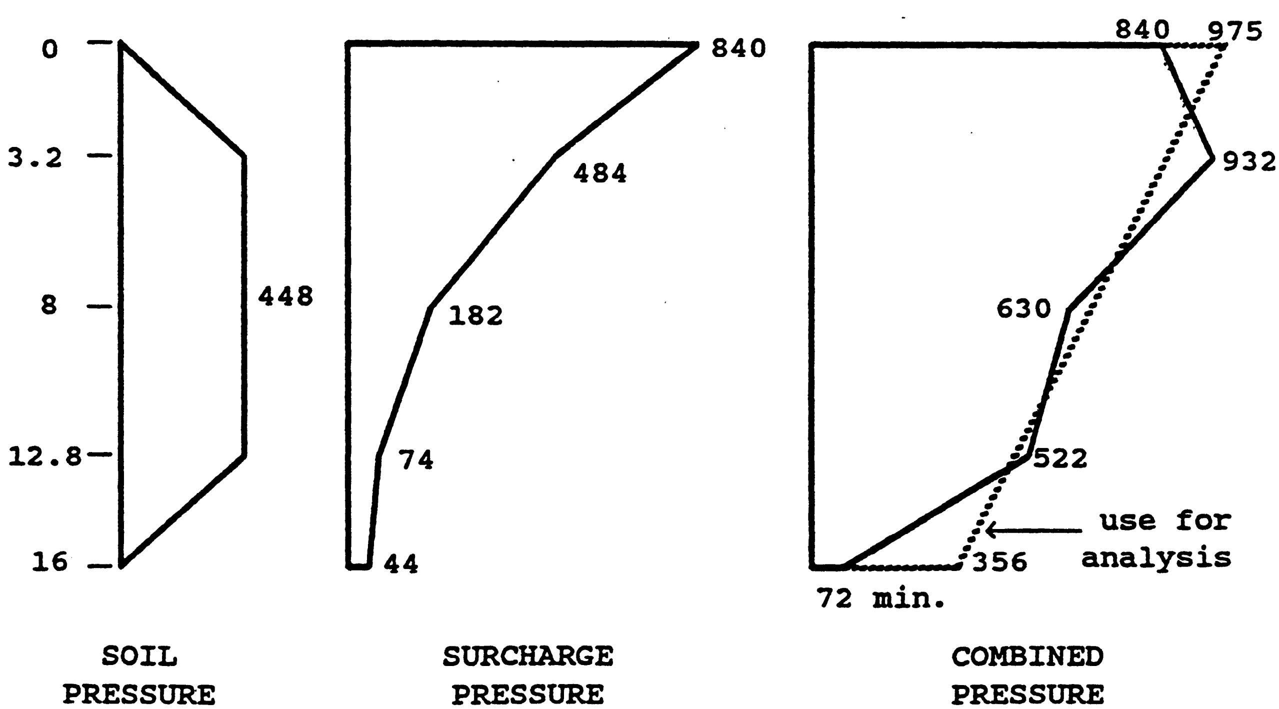
$\sigma_{3.2}$ arctan(8.87/3.2) = 70.16° = 1.224 Radians = β_R
sin β = 0.941 cos 2α = cos β = 0.339
$\sigma_{3.2}$ = [(2)(840)/π][1.224 - (0.941)(0.339)] = 484 psf

σ_8 arctan(8.87/8) = 47.95° = 0.837 Radians = β_R
sin β = 0.743 cos 2α = cos β = 0.670
σ_8 = [(2)(840)/π][0.837 - (0.743)(0.670)] = 182 psf

$\sigma_{12.8}$ arctan(8.87/12.8) = 34.72° = 0.606 Radians = β_R
sin β = 0.570 cos 2α = cos β = 0.822
$\sigma_{12.8}$ = [(2)(840)/π][0.606 - (0.570)(0.822)] = 74 psf

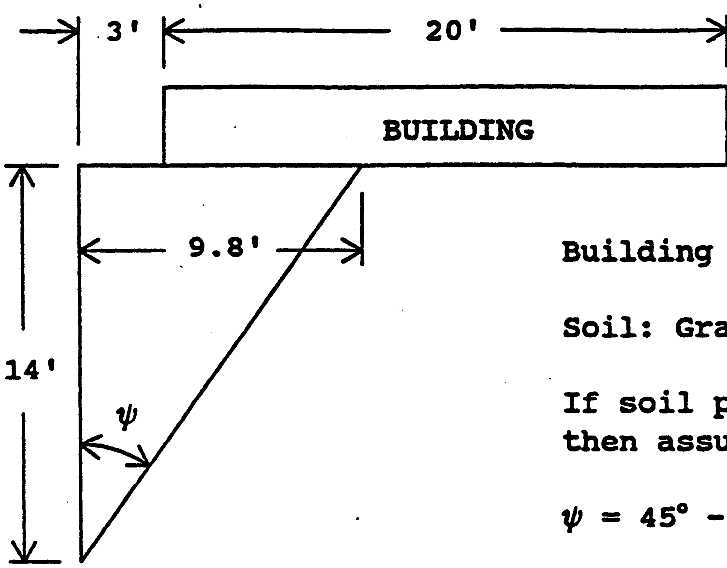
σ_{16} arctan(8.87/16) = 29.00° = 0.506 Radians = β_R
sin β = 0.485 cos 2α = cos β = 0.875
σ_{16} = [(2)(840)/π][0.506 - (0.485)(0.875)] = 44 psf

SUMMARY OF LATERAL PRESSURES:



This method gives consistent and acceptable results. Other surcharge loads may then be added (Q_2, Q_3,....) to the combined diagram to use for design or analysis as was done above.

SAMPLE PROBLEM NO. 5 - BOUSSINESQ STRIP LOAD (FAILURE WEDGE)



Building surcharge Q = 800 psf

Soil: Gravel/sand mix

If soil parameters are not given then assume $\phi = 20°$, $\gamma = 115$ pcf

$\psi = 45° - 20°/2 = 35°$

(14)(tan 35°) = 9.8'

DETERMINE BOUSSINESQ STRIP LOAD pressures for 2 conditions:*

1.) Limit surcharge to failure wedge width (9.8').

2.) Compute value for full width of building surcharge load.

Depth to pressure plane (feet)	Pressure for condition 1 (psf)	Pressure for condition 2 (psf)
0.1	24 < 72	30 < 72
1.0	213	272
2.0	332	446
4.0	341	543
6.0	261	513
8.0	187	456
10.0	132	397
12.0	94	341
14.0	69 < 72	292

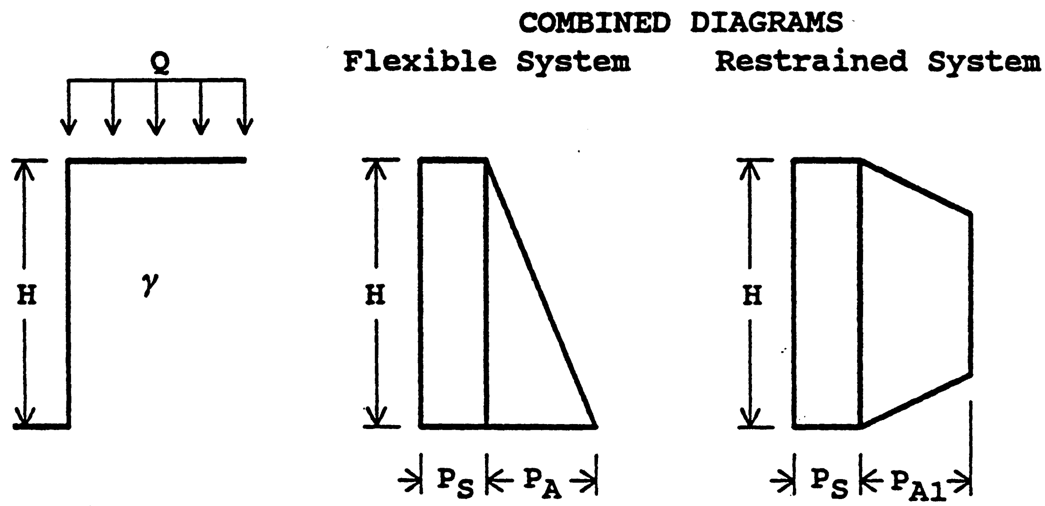
CONDITION 2 CONTROLS

Building, Highway and similar surcharge loadings cannot be limited to the width of the soil failure wedge.

*Note: Soil pressures would be the same for both conditions.

UNIFORM SURCHARGE

A very-wide uniform-load causes an equal&crease in pressure at all depths. This is represented by a rectangular pressure diagram.



$P_S = K_a Q$
$P_A = K_a \gamma H$

Note that the K_a value for the level surcharge will be the same as for the basic soil.

For the restrained system shown above:
$P_{A1} = 0.8P_A$

SAMPLE PROBLEM NO. 6 - UNIFORM SURCHARGE

GIVEN: Restrained system
Depth of trench (H) = 12'
Soil unit weight (γ_1) = 110 pcf
$K_a = 0.36$

Surcharge load consists of a uniform stockpile of materials which weigh 195 pcf, and the stockpile height is 4 feet.

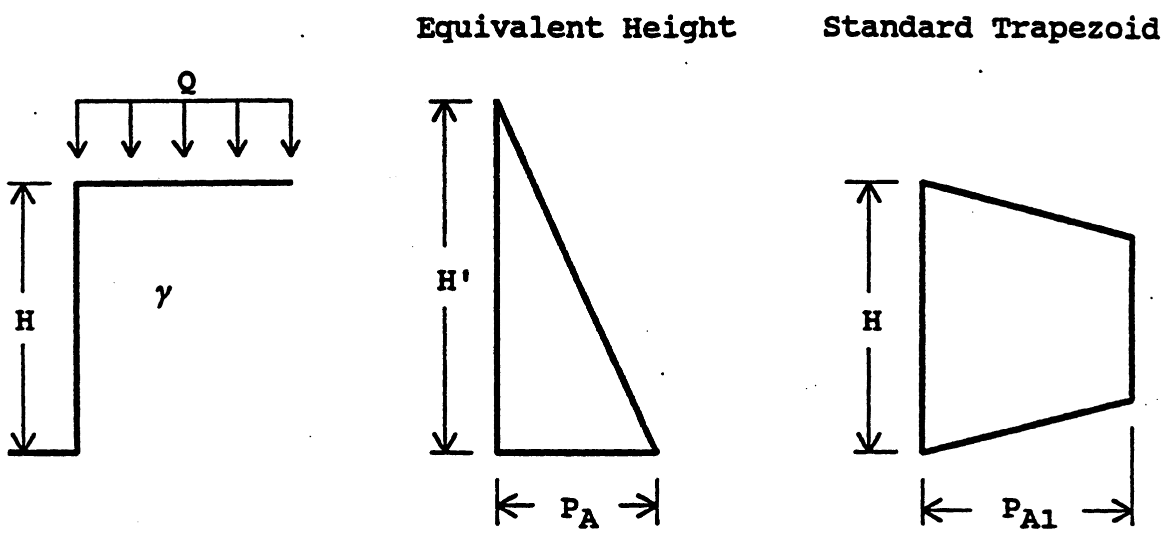
SOLUTION: γ_2 = 195 pcf Q = (195)(4) = 780 psf

$P_S = K_a Q = (0.36)(780)$ = 281 psf > 72 min
$P_A = (0.36)(110)(12)$ = 475 psf

Convert to trapezoid pressure diagram: $P_{A1} = (0.8)(475) = 380$ psf.

UNIFORM SURCHARGE - EQUIVALENT HEIGHT

A uniform load can also be represented by an equivalent height of soil. The height of the original excavation is increased by an amount equal to the surcharge pressure divided by the density of the soil.



SAMPLE PROBLEM NO. 7 - EQUIVALENT HEIGHT

GIVEN: (same as previous example)

$H = 12'$
$\gamma = 110$ pcf
$K_a = 0.36$

SOLUTION: $H_S = Q/\gamma = 780/110 = 7.1'$
$H' = H + H_S = 12 + 7.1 = 19.1'$
$P_A = K_a \gamma H' = (0.36)(110)(19.1) = 756$ psf

Convert to trapezoid distribution:

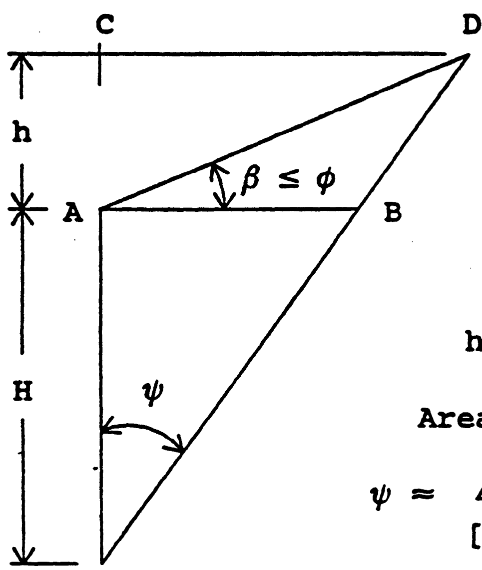
$P_{A1} = (0.8)(756) = 605$ psf

EMBANKMENT SURCHARGE

A recurring shoring problem involves embankments, spoil piles, or surcharge loads adjacent to the excavations which must be considered in the shoring design.

SLOPING EMBANKMENTS:

Conventional analysis (Rankine, Coulomb, or Log-Spiral) should be used for slopes with angles equal to or less than the soil internal friction angle(ϕ)



$$AB = H\tan\psi$$

$$CD = (H + h)\tan\psi$$

$$h = CD\tan\beta$$

$$(H + h)\tan\psi = h/\tan\beta$$

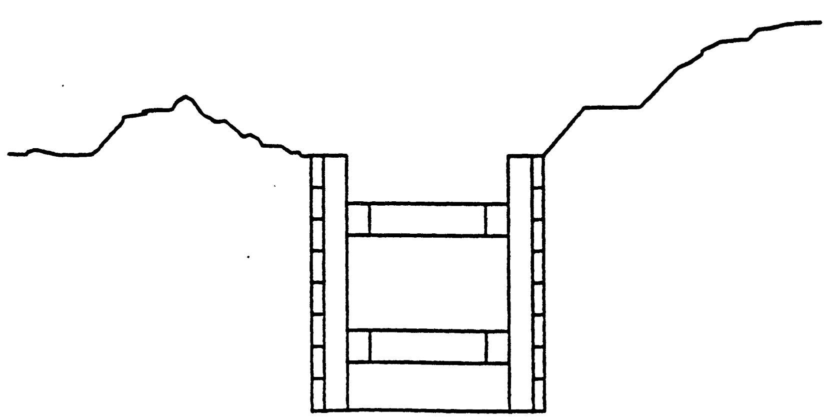
$$h/H = (\tan\psi)(\tan\beta)/\{1 - (\tan\psi)(\tan\beta)\}$$

$$\text{Area } ABD = (h)(AB)/2$$

$\psi \approx 45° - \phi/2$ (or $\approx 35°$ if ϕ not known)
[Theoretically for level surfaces only!]

IRREGULAR EMBANKMENTS:

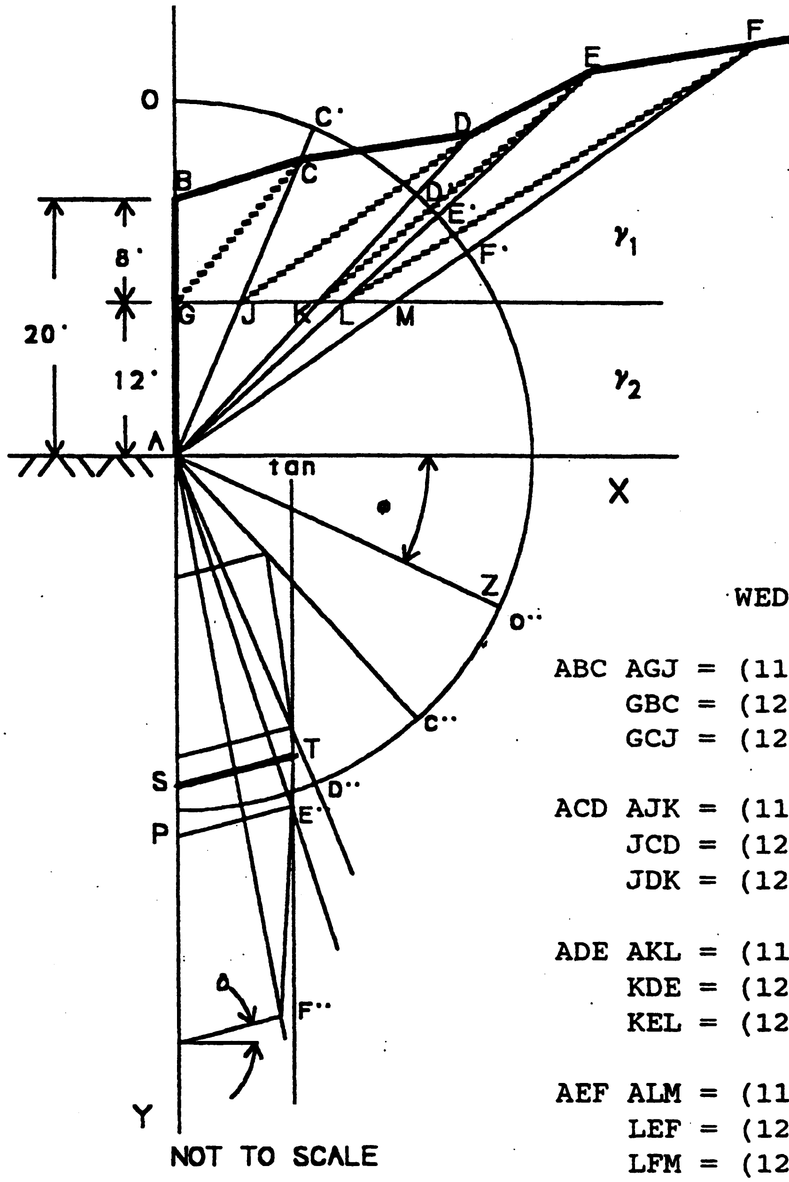
Irregular embankment slopes cannot rationally be converted to the conventional slope analysis method depicted above. Irregular embankment sloping conditions should be analyzed by the Trial Wedge method.



SURCHARGES

TRIAL WEDGE

The trial Wedge Method is semi-graphical. It is applicable to irregular slopes and varying soil strata.



Known:

ϕ = 25°
γ_1 = 120 pcf
γ_2 = 110 pcf
H = 20'
δ = 15°
C = 0

WEDGE WEIGHT CALCULATIONS:

ABC	AGJ =	(110)(5.0)(12.0)/2 =	3,300
	GBC =	(120)(5.3)(14.6)/2 =	4,643
	GCJ =	(120)(3.8)(14.6)/2 =	3,329
			11,272
ACD	AJK =	(110)(6.0)(12.0)/2 =	3,960
	JCD =	(120)(6.2)(22.1)/2 =	8,221
	JDK =	(120)(3.5)(22.1)/2 =	4,641
			16,822
ADE	AKL =	(110)(2.0)(12.0)/2 =	1,320
	KDE =	(120)(2.3)(27.9)/2 =	3,850
	KEL =	(120)(1.3)(27.9)/2 =	2,176
			7,346
AEF	ALM =	(110)(4.0)(12.0)/2 =	2,640
	LEF =	(120)(5.0)(37.9)/2 =	11,370
	LFM =	(120)(2.1)(37.9)/2 =	4,775
			18,785
		TOTAL:	54,225

ST = P (Total resultant force on system): scales = 11,000 Lb
P_H (Horizontal component) = (11,000)(cos 15^o) = 10,625 Lb

PROCEDURE FOR TRIAL WEDGE SOLUTION

1. Plot wall and soil profile (ABCDEF).

2. Draw AX perpendicular to back of the wall or system.

3. Draw AY vertical.

4. Draw an arc from AO to AP (any convenient radius).

5. Draw rays through break points arid any selected intermediate points (AB, AC, etc) to intersect arc drawn in 4.

6. Compute the individual wedge weights (sea previous sheet).

7. Plot, at a convenient scale, the accumulative wedge weights on AY, down from A (Lb),

8. Draw AZ at angle ϕ, from AX.

9. From AZ, draw rays duplicating those drawn in 5 (AB, AC, etc). The arc distances from AO to each ray is equal to the arc distances from AZ to each duplicate ray.

10. Draw lines at an angle equal to 6 from the wedge weight points along AY to their respective rays drawn in 9.

11. Plot a curve intersecting the points from 10.

12. Establish a point of tangency "T" (with a line parallel to line AY) to that portion or the curve furthermost from line AY.

13. Establish point "S" by sketching a line from point "T", parallel to the lines delineated in step 10. Measure between points S and T, using the same scale used to plot wedge weights, to get the resultant force acting on the shoring at the angle 6.

14. To resolve ST to it's horizontal component (P_H), multiply ST by the cos δ. The total resultant force may be depicted as a force arrow acting on a pressure triangle at 1/3 the height of the vertical excavation. P_A then equals $2P_H/H$.

15. Resolve to the appropriate pressure diagram, which will be dependent upon soil and system type.

COMPARISON OF TEE VARIOUS METHODS USED IN DETERMINING LATERAL SOIL PRESSURES

The following example problems demonstrate differences in methods used to compute soil pressure. The same shoring configuration and the maximum embankment slope angle β ($\beta=\phi$) allowed for in the Coulomb, Rankine, and Log-Spiral methods is used in all cases.

A summary of results follows the example problems.

BASIC CRITERIA FOR ALL PROBLEMS:

$\beta = \phi = 34°$
Friction angle $\delta = 0$
H = 14'
$\gamma = 130$ pcf
Assume cantilever system
Wall angle, $\omega = 0$

METHODS TO BE ANALYZED:

Rankine
Coulomb
Log-Spiral
Trial Wedge

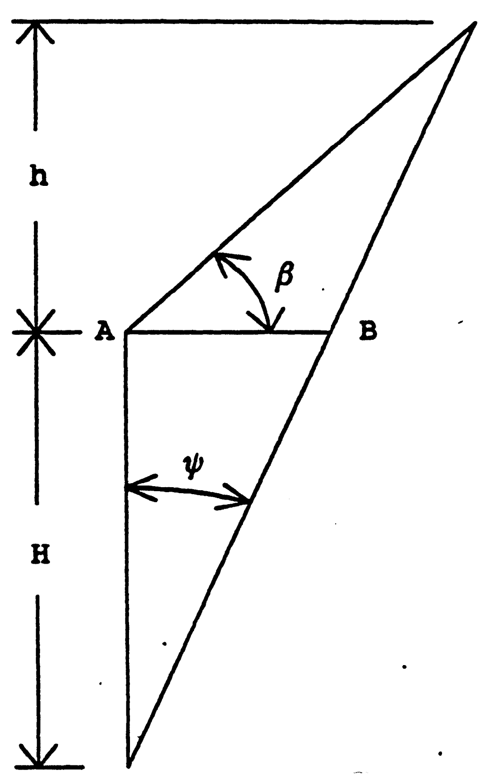
Use approximation for the angle ψ.

$\psi \approx 45° - \phi/2$
$\approx 45° - 17° \approx 28°$

AB = 14(tan 28°) = 7.44'

(14 + h)(tan 28°) = h/(tan 34°)

$\therefore$ h = 7.83'

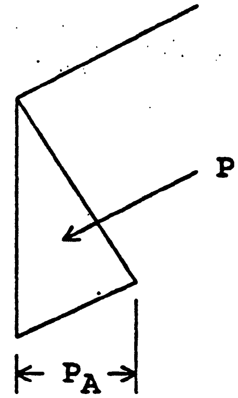


RANKINE

$$K_a = \cos\beta \cdot \left[\frac{\cos\beta - [\cos^2\beta - \cos^2\phi]^{1/2}}{\cos\beta + [\cos^2\beta - \cos^2\phi]^{1/2}} \right]$$

$$= \cos\beta[1.0] = 0.82904$$

$$P_A = K_a\gamma H(\cos\beta) = (0.82904)(130)(14)(\cos 34°)$$
$$= 1251 \text{ psf}$$



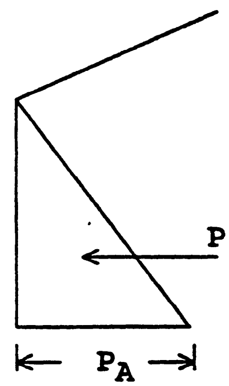
Total Lateral Pressure (P) $= K_a\gamma H^2/2$
$= (0.82904)(130)(14)^2/2 =$ <u>10,562 Lb</u>
(acting at angle β from the horizontal)

COULOMB

$$K_a = \frac{\cos^2\phi}{\cos\delta\left[1 + \sqrt{\dfrac{(\sin(\phi+\delta))(\sin(\phi-\beta))}{(\cos\delta)(\cos\beta)}}\right]^2}$$

$$= \cos^2\phi[1.0] = 0.6873$$

$$P_A = K_a\gamma H = (0.6873)(130)(14)$$
$$= 1251 \text{ psf}$$

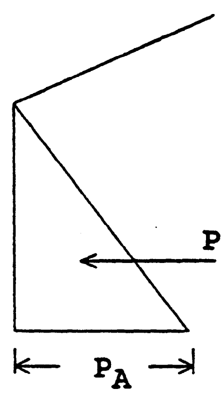


Total Lateral Pressure (P) $= K_a\gamma H^2/2$
$= (0.6873)(130)(14)^2/2 =$ <u>8,756 Lb</u>
(acting at angle δ from the horizontal)

LOG-SPIRAL

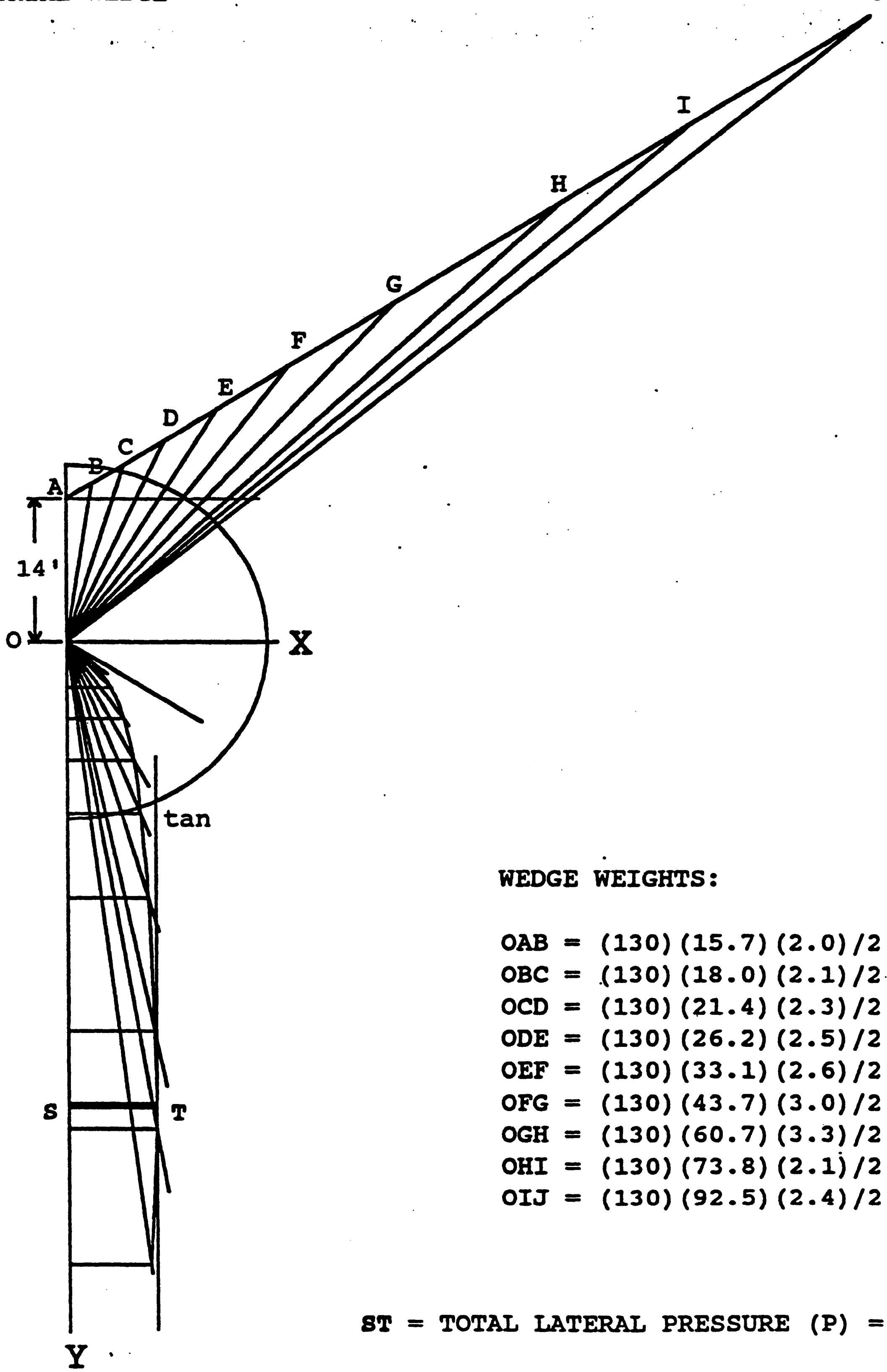
Select K_a from **FIGURE 8**
For $\beta/\phi = 1.0$ and $\delta = 0$ $\quad K_a = 0.78$

$$P_A = K_a\gamma H = (0.78)(130)(14) = 1{,}420 \text{ psf}$$



Total Lateral Pressure (P) $= K_a\gamma H^2/2$
$= (0.78)(130)(14)^2/2 =$ <u>9,937 Lb</u>
(acting at angle δ from the horizontal)

TRIAL WEDGE



WEDGE WEIGHTS:

OAB	=	(130)(15.7)(2.0)/2 =	2,041
OBC	=	(130)(18.0)(2.1)/2 =	2,457
OCD	=	(130)(21.4)(2.3)/2 =	3,199
ODE	=	(130)(26.2)(2.5)/2 =	4,258
OEF	=	(130)(33.1)(2.6)/2 =	5,594
OFG	=	(130)(43.7)(3.0)/2 =	8,522
OGH	=	(130)(60.7)(3.3)/2 =	13,020
OHI	=	(130)(73.8)(2.1)/2 =	10,074
OIJ	=	(130)(92.5)(2.4)/2 =	14,430

ST = TOTAL LATERAL PRESSURE (P) = <u>8,000 Lb</u>

(acting at angle δ from the horizontal)

SUMMARY OF RESULTS - UNIFORM SLOPING EMBANKMENT

Method	Total Lateral Pressure (Lb)*	Horizontal Pressure (Lb)*
Rankine	10,562	8,756
Coulomb	8,756	8,756
Log-Spiral	9,937	9,937
Trial Wedge	8,000	8,000

* Minimum construction surcharge of 72 psf is not included

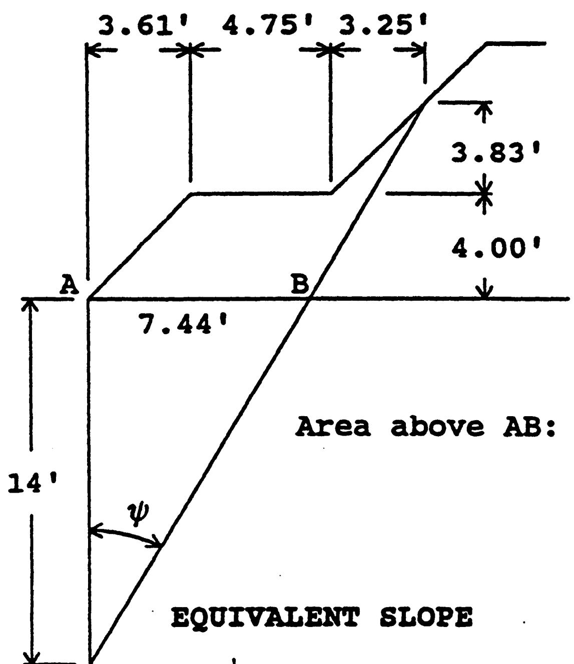
Theoretically the wedge analysis should give the same answer as the Coulomb Method provided $\beta < \phi$, C = 0, and δ = 0.

NOTE: The Log-Spiral method is considered to be the most theoretically correct.

IRREGULAR SLOPED SURCHARGE

As previously stated, the only rational solution for treating irregular sloped embankments is by the Trial Wedge method. Othermeans have been employed, but as the following pages show they are incorrect and should be avoided.

EXAMPLE 11:

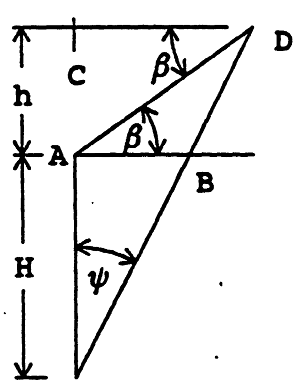


$\phi = 34°$
$\gamma = 130$ pcf
$\delta = 0$
$C = 0$
$\psi \approx \tan(45° - \phi/2) \approx 28°$

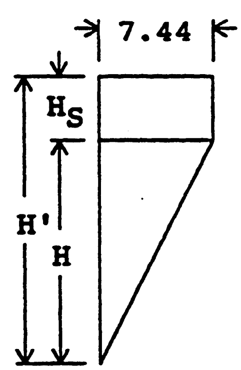
$AB = (14)(\tan 28°) = 7.44'$

Area above AB: $(11.61 + 8)(4)/2 + (3.25)(3.83)/2$
$- (11.61 - 7.44)(7.83)/2$
$= 29.1\ Ft^2/LF$

EQUIVALENT SLOPE



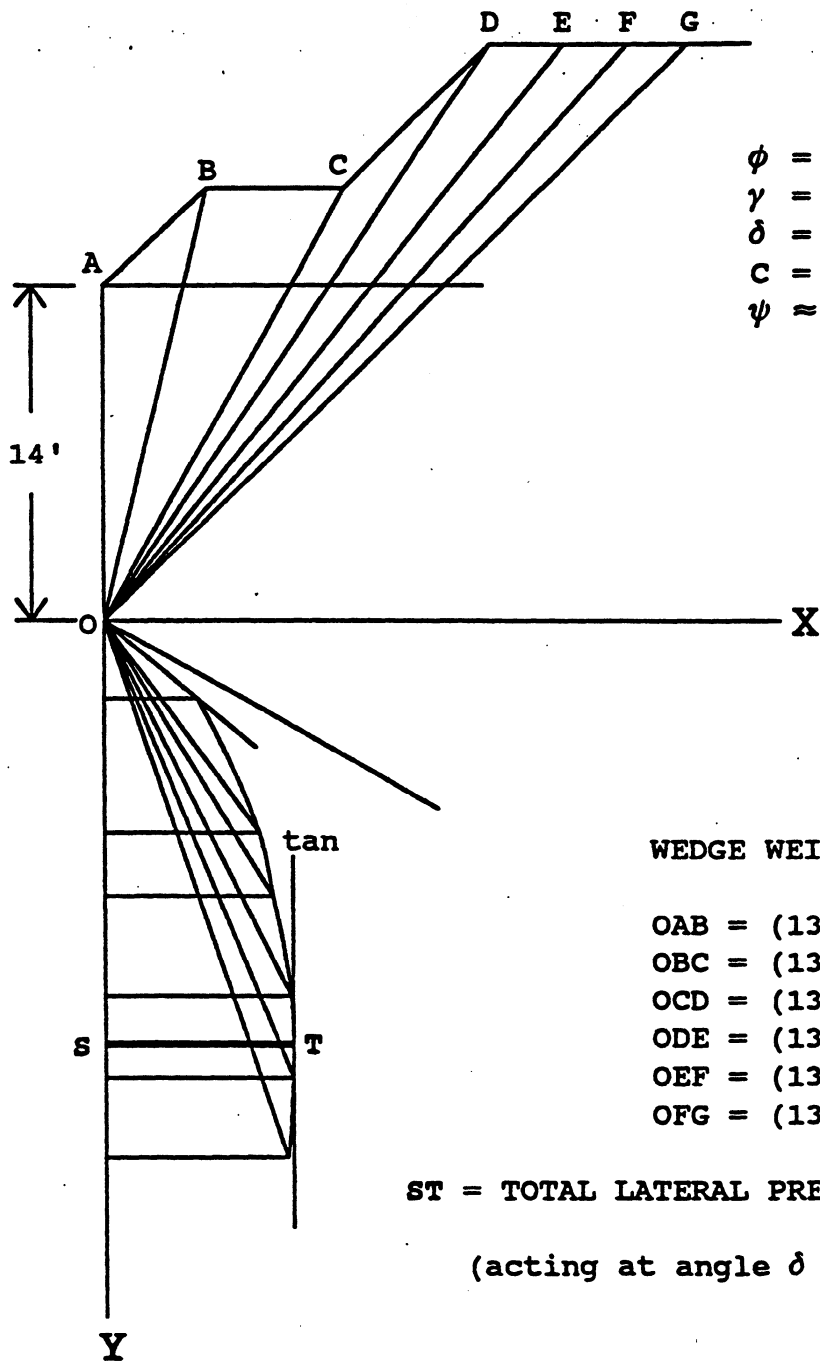
EQUIVALENT SOIL



area = (AB)(h)/2
$h = (2)(29.1)/7.44 = 7.83'$
$CD = (h + H)\tan\psi = 11.61'$
$\tan\beta = h/CD = 0.67$
$\therefore \beta = 34°,\ \beta/\phi = 1.0$
From **FIGURE 8**, $K_a = 0.78$
$P = (K_a)(\gamma)(H)^2/2$
$P = (0.78)(130)(14)^2/2$
$= \underline{9,937\ Lb/LF}$

H_S = area/7.44 = 3.91'
$H' = H + H_S = 14 + 3.91$
$= 17.91'$
$K_a = \tan^2(45° - \phi/2) = 0.28$
$P = (K_a)(\gamma)(H')^2/2$
$= (0.28)(130)(17.92)^2/2$
$= \underline{5,838\ Lb/LF}$

TRIAL WEDGE



$\phi = 34°$
$\gamma = 130$ PCF
$\delta = 0$
$C = 0$
$\psi \approx \tan(45° - \phi/2) \approx 28°$

WEDGE WEIGHTS:

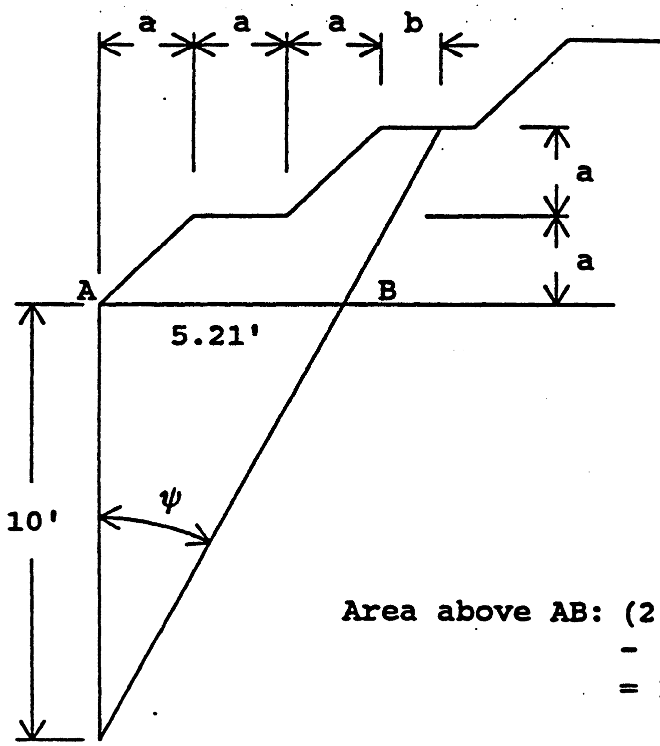
OAB = (130)(25.1 Ft^2) = 3,263
OBC = (130)(42.8 Ft^2) = 5,564
OCD = (130)(20.8 Ft^2) = 2,704
ODE = (130)(31.0 Ft^2) = 4,030
OEF = (130)(27.0 Ft^2) = 3,510
OFG = (130)(25.0 Ft^2) = 3,250

ST = TOTAL LATERAL PRESSURE (P) = <u>6,500 Lb</u>

(acting at angle δ from the horizontal)

NOT TO SCALE

EXAMPLE 2:



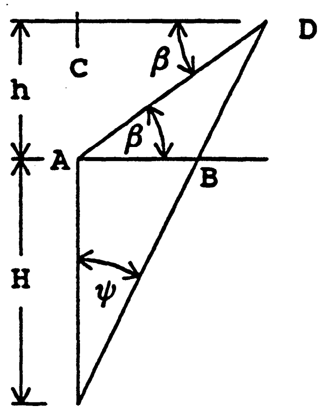
$a = 2.00'$
$b = 1.29'$

$\phi = 35°$
$\gamma = 110$ pcf
$\delta = 0$
$C = 0$
$\psi \approx \tan(45° - \phi/2) \approx 27.5°$

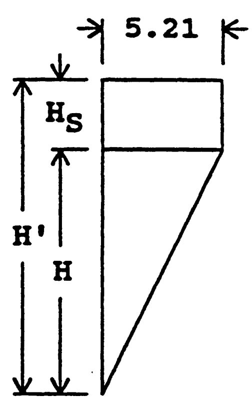
$AB = (10)(\tan 27.5°) = 5.21'$

Area above AB: $(2)(2)/2 + (2)(2) + (3.29)(4)$
$- (2)(2)/2 - (7.29 - 5.21)(4)/2$
$= 13.0\ Ft^2/LF$

EQUIVALENT SLOPE



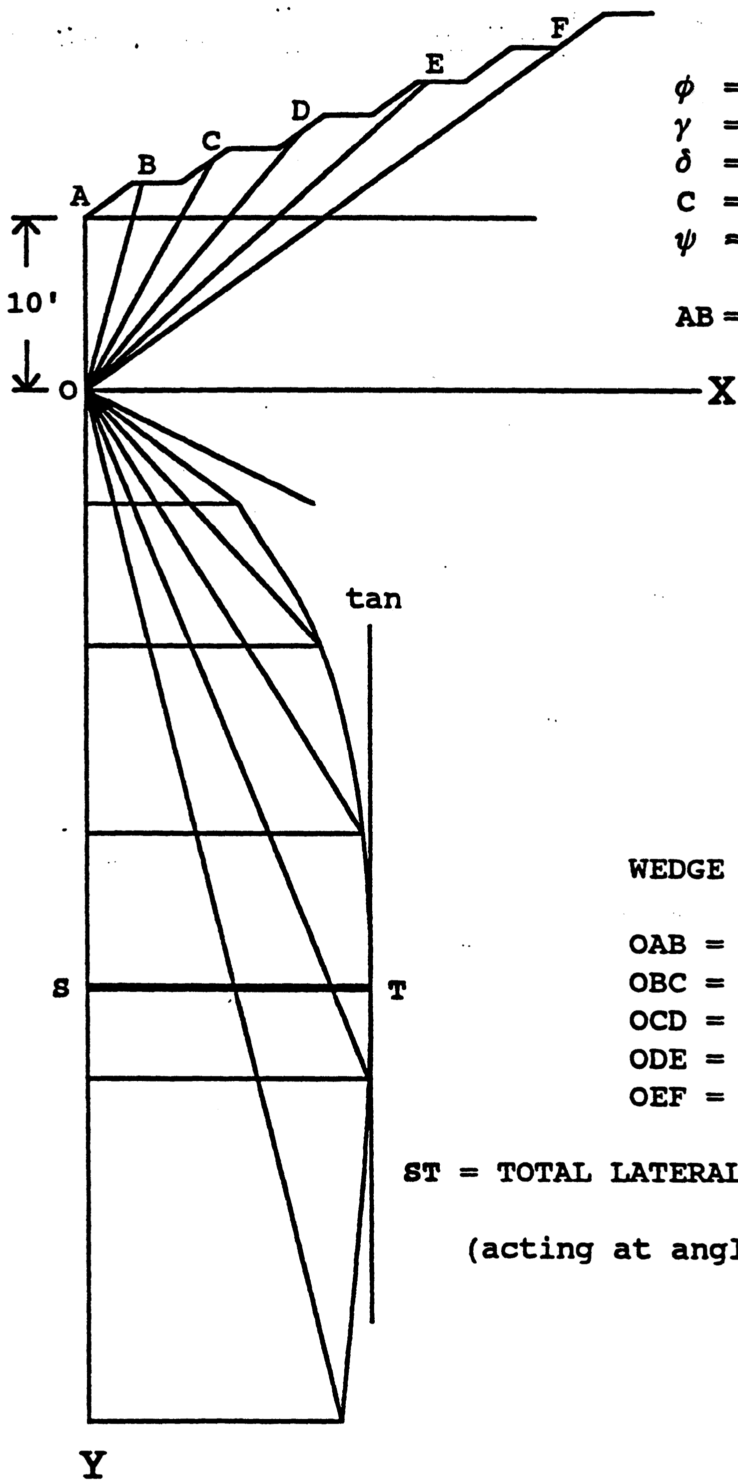
EQUIVALENT SOIL



area = (AB)(h)/2
$h = (2)(13.0)/5.21 = 4.99'$
$CD = (h + H)\tan\psi = 7.80'$
$\tan\beta = h/CD = 0.64$
$\therefore \beta = 32.6°,\ \beta/\phi = 0.93$
From **FIGURE 8**, $K_a = 0.62$
$P = (K_a)(\gamma)(H)^2/2$
$P = (0.62)(110)(10)^2/2$
$=$ <u>3,410 Lb/LF</u>

H_S = area/5.21 = 2.49'
$H' = H + H_S = 10 + 2.49$
$= 12.49'$
$K_a = \tan^2(45° - \phi/2) = 0.27$
$P = (K_a)(\gamma)(H')^2/2$
$= (0.27)(110)(12.49)^2/2$
$=$ <u>2,317 Lb/LF</u>

TRIAL WEDGE



$\phi = 35°$
$\gamma = 110$ pcf
$\delta = 0$
$C = 0$
$\psi \approx \tan(45° - \phi/2) \approx 27.5°$

$AB = (10)(\tan 27.5°) = 5.21'$

WEDGE WEIGHTS:

$OAB = (110)(12.40\ Ft^2) = 1{,}364$
$OBC = (110)(14.93\ Ft^2) = 1{,}642$
$OCD = (110)(19.67\ Ft^2) = 2{,}164$
$ODE = (110)(26.60\ Ft^2) = 2{,}926$
$OEF = (110)(36.40\ Ft^2) = 4{,}004$

ST = TOTAL LATERAL PRESSURE (P) = <u>2,500 Lb</u>

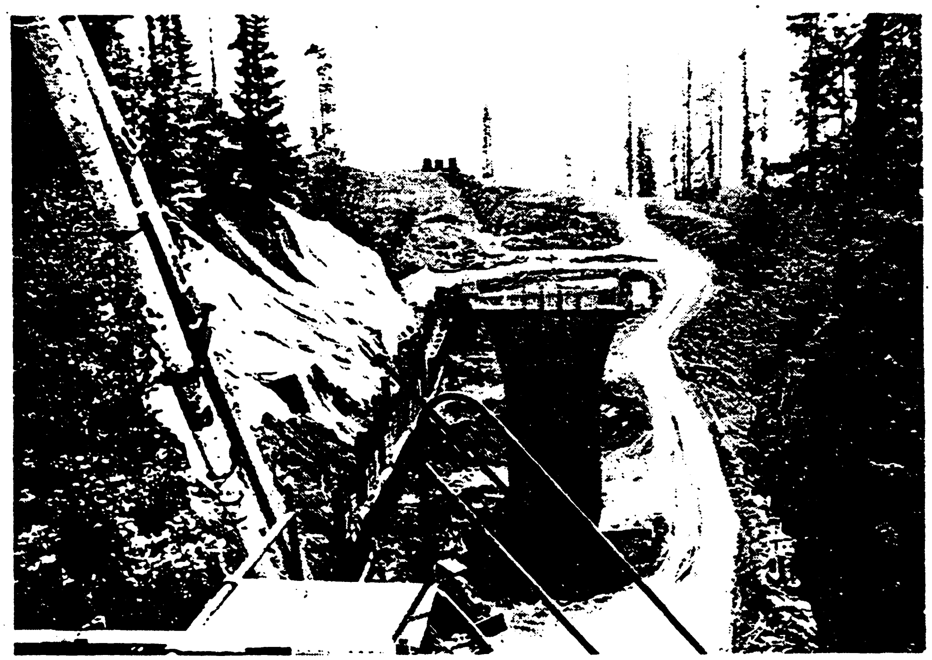
(acting at angle δ from the horizontal)

NOT TO SCALE

SUMMARY OF RESULTS

Trial Wedge	Equivalent Slope	% difference
6,500	9,937	52.9
2,500	3,410	36.4

Trial Wedge	Equivalent Soil	% difference
6,500	5,838	-10.2
2,500	2,317	-7.3



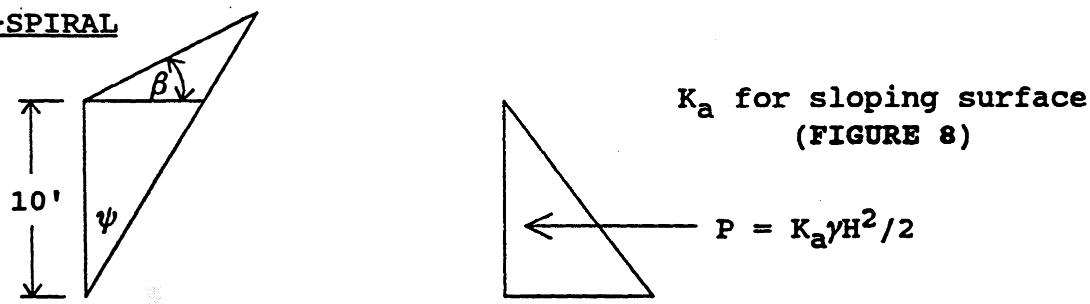
CONSTANT SLOPE COMPARISON

In determining -lateral earth pressures for embankments with a constant slope the Log-Spiralmethod is the easiest and most preferred. solution. The table on the following page compares the Log-Spiral method and two of the more common "shortcut" solutions. From this table it can be seen that the difference between these methods and the more theoretically correct solution (Log-Spiral) can be quite large depending on the parameters used.

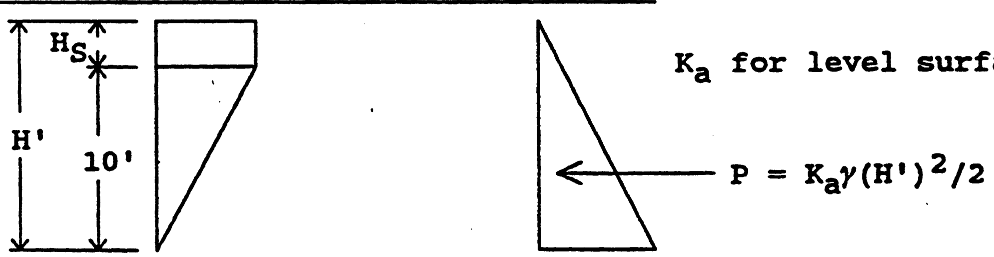
As the ratio between the embankment slope angle (β) and the soil internal friction angle (ϕ) increases, so does the difference between the various methods. When this ratio approaches 0.8, the difference becomes significant. For all practicality when the ratio is 0.6 or less, the slope condition can be treated as a level surface as shown below and by the comparison table on page 6-21. This leaves only a small range where these "shortcut " methods are of any value. Since this is not very practical and as the Log-Spiral method is quite easy to employ, these other methods will not be used for analysis or review.

For all three conditions, γ = 100 pcf, δ = 0, and C = 0.

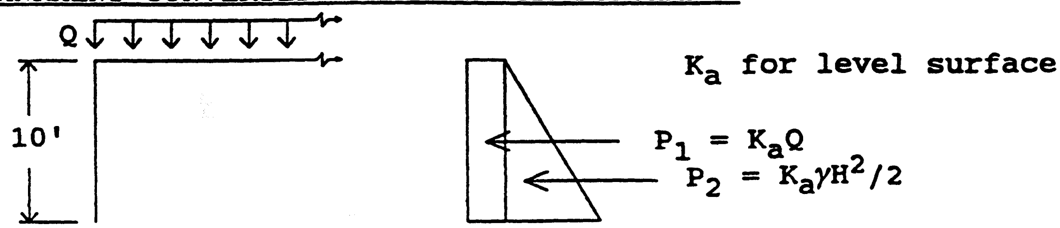
LOG-SPIRAL



EMBANKMENT CONVERTED TO A LEVEL SURFACE



EMBANKMENT CONVERTED TO A UNIFORM SURCHARGE



CONSTANT SLOPE EMBANKMENT
COMPARISON TABLE

	β/ϕ = 0.4	β/ϕ = 0.6	β/ϕ = 0.8	β/ϕ = 1.0
ϕ = 10°				
Log-Spiral	3,700	4,000	4,300	5,500
Conv. to H'	3,742	3,869	4,007	4,158
Uniform Q	3,739	3,861	3,991	4,131
ϕ = 16°				
Log-Spiral	3,150	3,400	3,750	5,250
Conv. to H'	3,108	3,270	3,457	3,677
Uniform Q	3,102	3,255	3,427	3,623
ϕ = 20°				
Log-Spiral	2,750	3,000	3,350	4,950
Conv. to H'	2,725	2,897	3,104	3,360
Uniform Q	2,718	2,879	3,065	3,288
ϕ = 26°				
Log-Spiral	2,250	2,500	2,850	4,500
Conv. to H'	2,211	2,384	2,604	2,899
Uniform Q	2,203	2,362	2,557	2,805
ϕ = 30°				
Log-Spiral	1,850	2,150	2,500	4,300
Conv. to H'	1,908	2,074	2,293	2,604
Uniform Q	1,900	2,052	2,244	2,500
ϕ = 35°				
Log-Spiral	1,600	1,750	2,150	3,800
Conv. to H'	1,559	1,707	1,916	2,235
Uniform Q	1,551	1,687	1,867	2,124
ϕ = 40°				
Log-Spiral	1,250	1,400	1,700	3,350
Conv. to H'	1,259	1,388	1,577	1,895
Uniform Q	1,252	1,369	1,531	1,782
ϕ = 45°				
Log-Spiral	1,000	1,100	1,375	2,900
Conv. to H'	999	1,105	1,270	1,575
Uniform Q	994	1,090	1,230	1,468

TRAFFIC LOADS

Traffic near an excavation is one of the more commonly occurring surcharge loads. Trying to analyze every possible scenario would not only be time consuming but not very practical. For normal situations. a surcharge load of 300 psf spreadover the width of the traveled way should be sufficient.

The following example compares the pressure diagrams for a HS20 truck. (using point loads) centered in a 12' lane to a load of Q = 300 psf (using the Boussinesq Strip method). The depth of excavation is 10'.

HS20 TRUCK

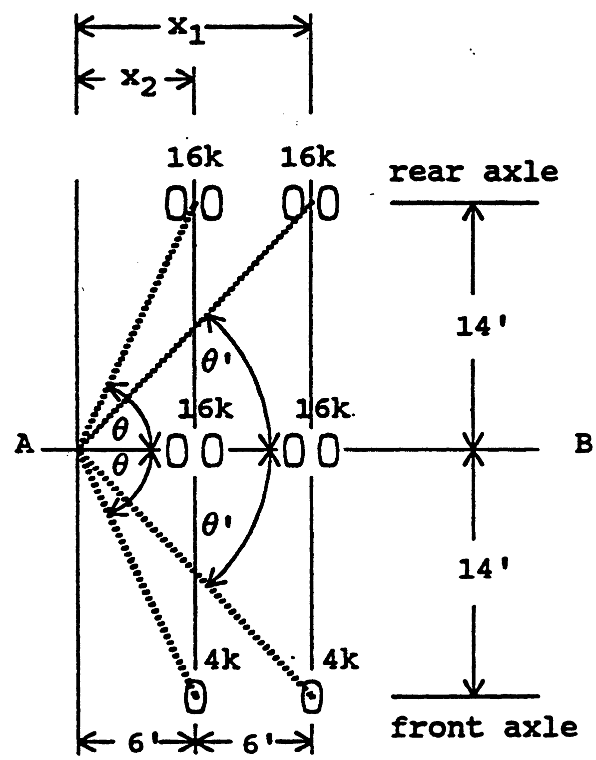
For a detailed explanation on point loads see the USS Steel Sheet Piling Design Manual (pg 15).

$x_1 = m_1H$
$\therefore m_1 = (12/10) = 1.2$

$x_2 = m_2H$
$\therefore m_2 = (6/10) = 0.6$

n = depth/H

Depth	n
2'	0.2
4'	0.4
6'	0.6
8'	0.8
10'	1.0



For line AB, $\sigma_H = 1.77 \dfrac{Q_P}{H^2} \dfrac{(m^2n^2)}{(m^2 + n^2)^3}$ (for m > 0.4)

For loads at an angle to AB, $\sigma_H' = \sigma_H \cos^2(1.1\theta)$

Front and rear rt wheels: $\theta = 66.8°$, $\therefore \cos^2[(1.1)(66.8°)] = 0.08$
Front and rear lt wheels: $\theta' = 49.4°$, $\therefore \cos^2[(1.1)(49.4°)] = 0.34$

1.) Rt rear wheels:
$\sigma_H = (0.08)(1.77)(16{,}000)(0.6^2)(n^2)/[10^2(0.6^2 + n^2)^3]$

2.) Lt rear wheels:
$\sigma_H = (0.34)(1.77)(16{,}000)(1.2^2)(n^2)/[10^2(1.2^2 + n^2)^3]$

3.) Rt center wheels:
$\sigma_H = (1.77)(16{,}000)(0.6^2)(n^2)/[10^2(0.6^2 + n^2)^3]$

4.) Lt center wheels:
$\sigma_H = (1.77)(16{,}000)(1.2^2)(n^2)/[10^2(1.2^2 + n^2)^3]$

5.) Rt front wheels:
$\sigma_H = (0.08)(1.77)(4{,}000)(0.6^2)(n^2)/[10^2(0.6^2 + n^2)^3]$

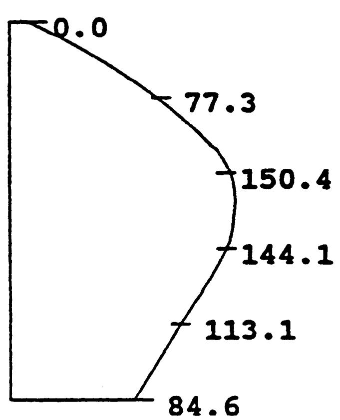
6.) Lt front wheels:
$\sigma_H = (0.34)(1.77)(4{,}000)(1.2^2)(n^2)/[10^2(1.2^2 + n^2)^3]$

Combine and simplify similar equations:

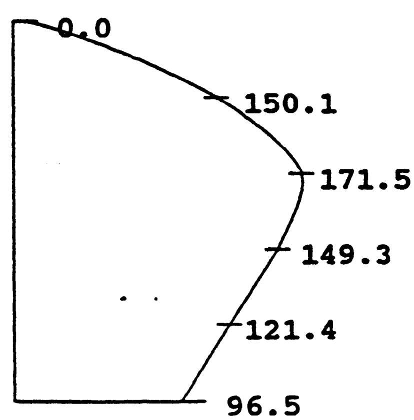
a.) $\sigma_H = (112.2)(n^2)/(0.36 + n^2)^3$
b.) $\sigma_H = (581.1)(n^2)/(1.44 + n^2)^3$

Depth	n	a.) σ_H	b.) σ_H	$\Sigma\sigma_H$
0'	0.0	0.0	0.0	0.0
2'	0.2	70.1	7.2	77.3
4'	0.4	127.7	22.7	150.4
6'	0.6	108.2	35.9	144.1
8'	0.8	71.8	41.3	113.1
10'	1.0	44.6	40.0	84.6

HS20 Truck
(point loading without impact)

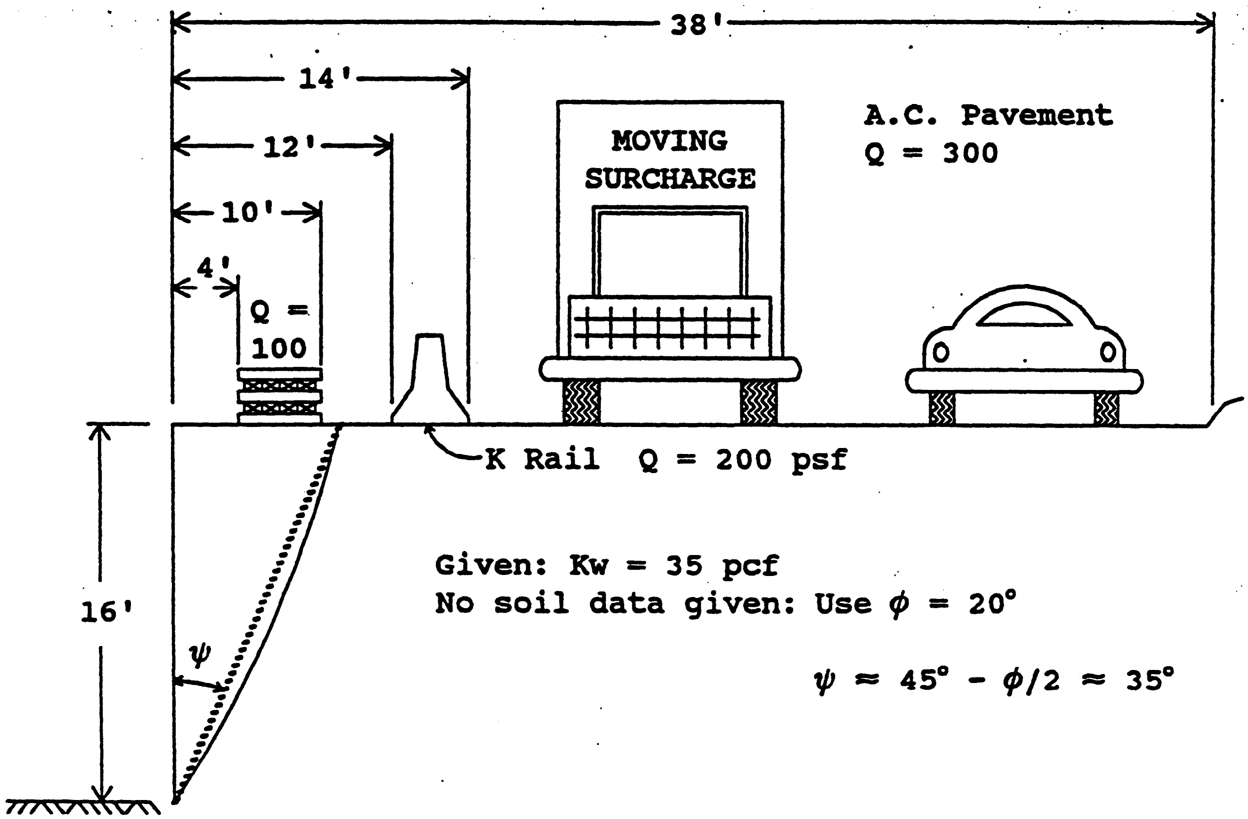


12' strip
(Q = 300 psf)



CONCLUSION: Strip load of Q = 300 psf compares favorably to a point load evaluation for HS20 truck loadings.

SAMPLE PROBLEM NO. 8 - SURCHARGE LOADS



Surcharge Lateral Pressures (psf)

Depth	Q = 100	Q = 200	Q = 300	Sum
0.1	1.9	0.3	1.7	72*
1	17.9	3.0	17.1	72*
2	30.2	5.8	33.8	72*
4	35.7	10.1	63.7	109.5
6	29.5	12.3	87.1	128.9
8	21.9	12.7	130.3	164.9
10	15.9	11.9	112.6	140.4
12	11.5	10.5	116.4	138.4
14	8.5	9.0	116.1	133.6
16	6.3	7.6	112.9	126.8

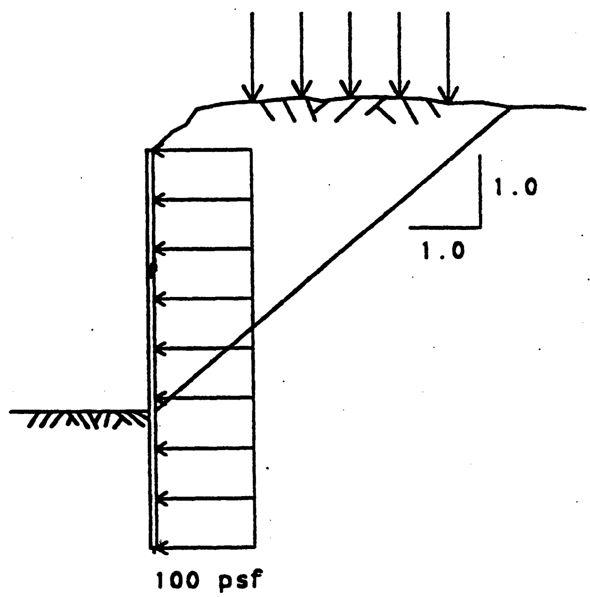
* Minimum construction surcharge load.

Add soil pressures to sum of surcharge loads to derive combined pressure diagram.

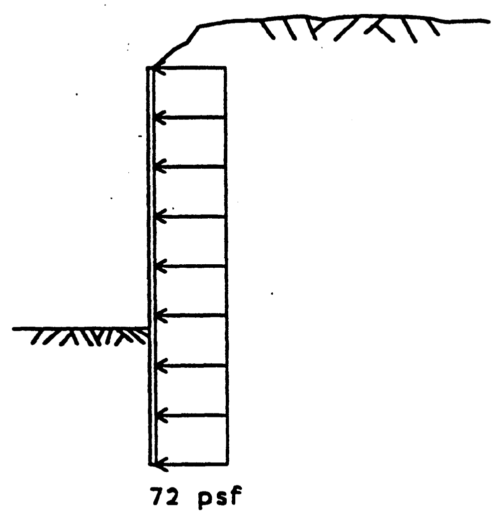
ALTERNATIVE SURCHARGE LOADING

An acceptable alternative to the Boussinesq analysis consists of imposing imaginary surcharges behind the shoring such that the resulting pressure diagram is a rectangle extending to the computed depth of the shoring members and of a uniform width of 100 psf. This 100 psf loading is analogous to the 72 psf pressure diagram used for minimum surcharge loading.

Generally, traffic and equipment surcharge loads beyond the limits of an inclined plane rising at an angle of 1.0 horizontal to 1.0 vertical from the bottom of the excavation may be ignored. Other loadings due to structures, or stockpiles of soil, materials or heavy equipment will need to be considered separately.



SHORING WITH TRAFFIC.



SHORING WITHOUT TRAFFIC

Shoring without traffic, structures or stockpiles.
Minimum Surcharge

RAILROAD SURCHARGES

Railroads adjacent to an excavation will result in very large surcharge pressures, and since railroad loadings are consideredto be a dynamic load, the short term load duration overstress factor of 1.33 cannot be used, Note also that wall friction (6) is not allowed for basic earth pressure computations.

The American Railway Engineering Association (AREA) specifies the use of a Boussinesq Formula for a railroad surcharge. The Southern Pacific Transportation Company (SPTC) concluded that values given by the AREA Boussinesq Formula were not realistic, (the maximum pressure was too high and occurred to on near the ground surface) so they developed their own live load surcharge earth pressure curve. The pressures from the SPTC curve are about half those given by the AREA Boussinesq curve. The SPTC curve is a part of the SPTC Supplement to Section 20a of the AREA specifications (See Appendix C). The SPTC curve, for Cooper E72 and E80 railroad loadings, is shown on the next page.

The SPTC Live Load Surcharge curve is to be used for all railroads. Note that all major railroads now require E80 design.

For a simplified engineering analysis (H < l0'), the railroad loading surcharge pressure may be assumed rectangular with an ordinate (P_S) equal to 0.8 of the maximum. pressure ordinate as given by the appropriate railroad curve. When using the railroad live load curves, the plot of the curve starts at the top of the railroad roadbed (bottom of ties), The portion that is used extends from the top of the excavation to the bottom of the shoring system. Depth of ballast may normally be assumed to be 1.5 feet.

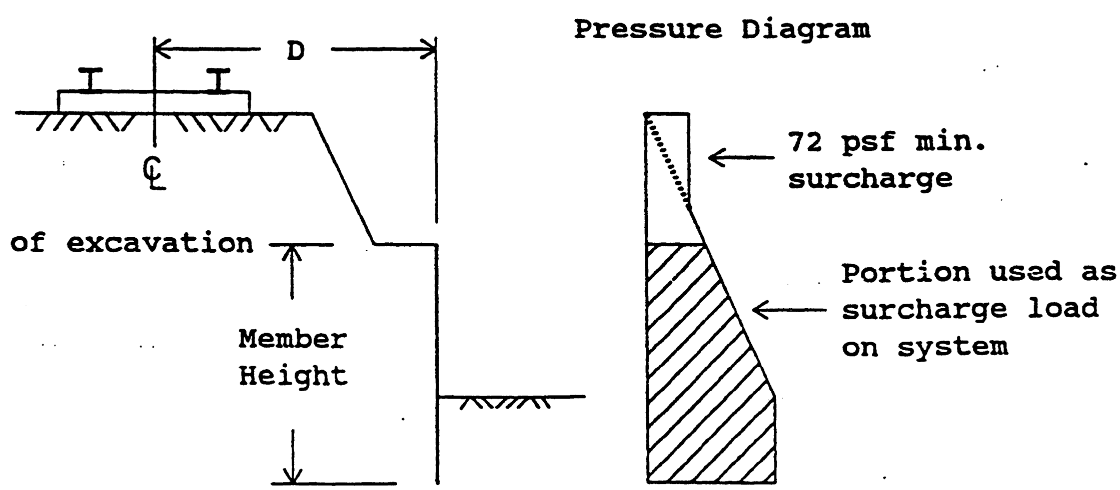
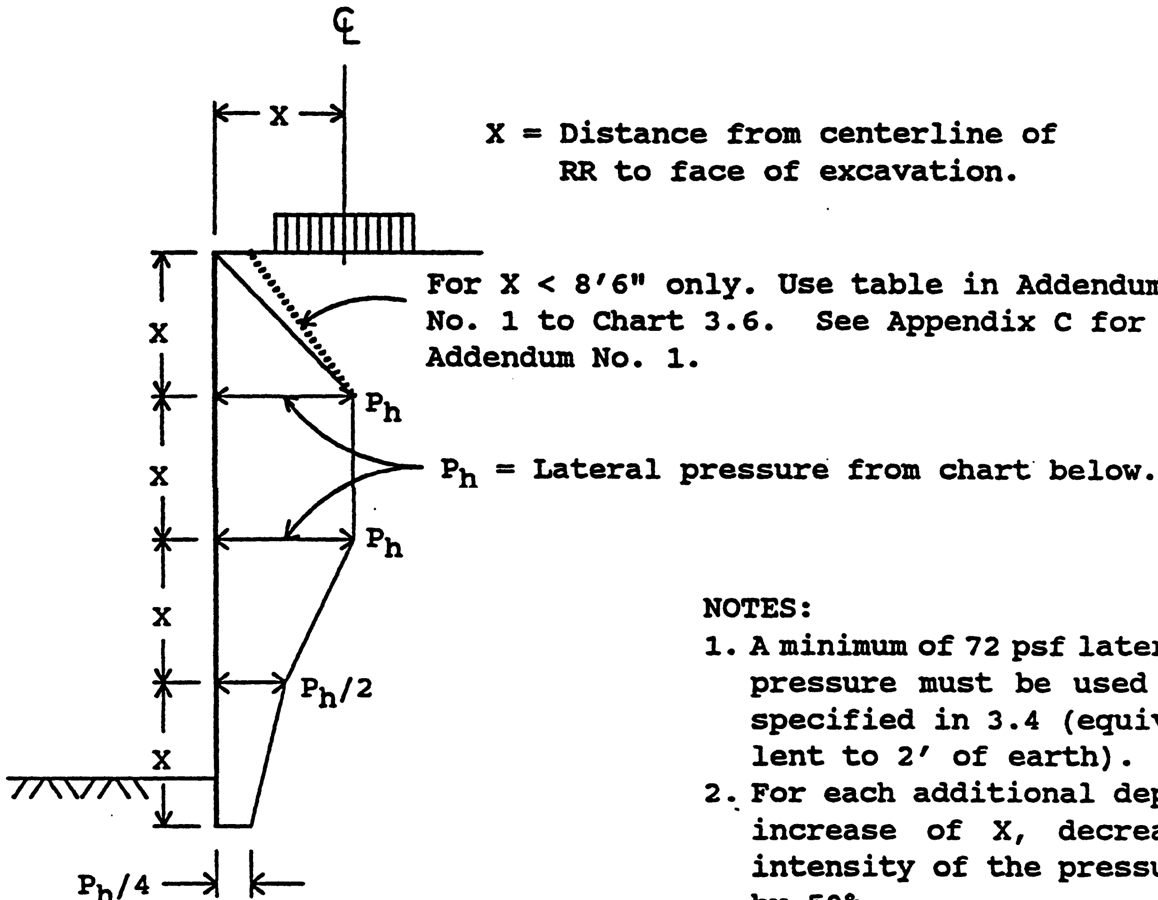
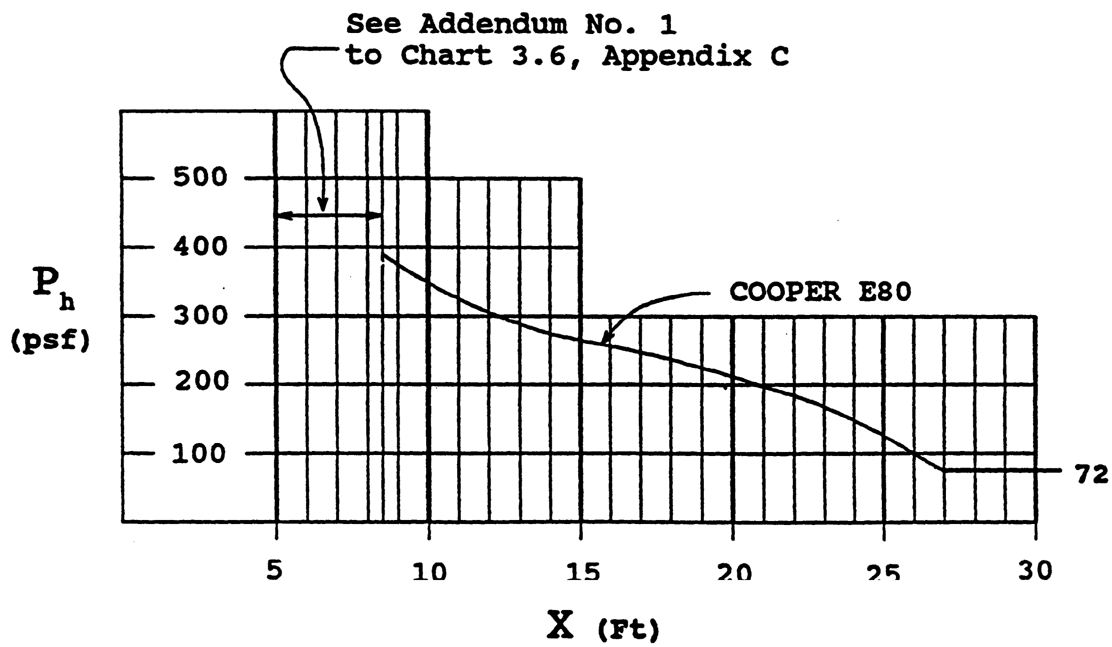


CHART 3.6 LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD



NOTES:

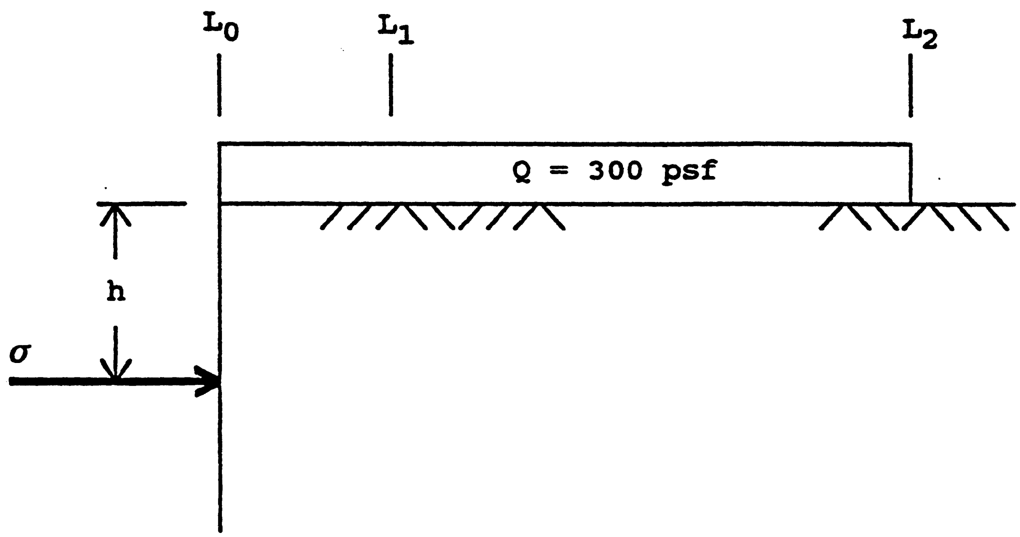
1. A minimum of 72 psf lateral pressure must be used as specified in 3.4 (equivalent to 2′ of earth).
2. For each additional depth increase of X, decrease intensity of the pressure by 50%.



TABULAR VALUES FOR STRIP LOADS

The following- tabular values may be used to obtain horizontal pressures due to surcharge loadings.

Tabular values are for a Boussinesq strip surcharge of Q = 300 psf for a length of surcharge beginning at the face of the excavation (L_0) to the end of the strip load (L_2). Surcharge pressures are listed for one foot increments of excavation to a depth of 20 feet.



For surcharges not beginning at the face of the excavation (L_1 subtract tabular values for distance L_1 from the tabular values for L_2. Prorate other Q values by using the ratio Q/300 (difference in L values).

Note: When L_0 = 0, the pressure at h = 0 is Q (300 psf).

EXAMPLE:

Begin Boussinesq strip load 6 feet from excavation, L_1 = 6'
End Boussinesq strip load 20 feet from excavation, L_2 = 20'
Surcharge load Q = 250 psf
Determine surcharge pressure at h = 12'

σ_{12} = 250/300(112.53 - 12.16) = 83.6 psf

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_0 TO L_1 OR L_2

h	1	2	3	4	5	6	7	8
1	54.51	135.06	181.25	208.27	225.57	237.49	246.16	252.74
2	12.16	54.51	99.55	135.06	161.47	181.25	196.40	208.27
3	4.15	24.15	54.51	85.43	112.53	135.06	153.52	168.69
4	1.85	12.16	31.23	54.51	77.97	99.55	118.58	135.06
5	0.97	6.81	18.95	35.70	54.51	73.39	91.21	107.48
6	0.57	4.15	12.16	24.15	38.76	54.51	70.29	85.43
7	0.36	2.70	8.18	16.88	28.13	40.97	54.51	68.07
8	0.24	1.85	5.73	12.16	20.85	31.23	42.64	54.51
9	0.17	1.32	4.15	8.99	15.77	24.15	33.70	43.94
10	0.13	0.97	3.10	6.81	12.16	18.95	26.92	35.70
11	0.09	0.74	2.37	5.27	9.53	15.08	21.73	29.24
12	0.07	0.57	1.85	4.15	7.59	12.16	17.73	24.15
13	0.06	0.45	1.47	3.33	6.13	9.92	14.61	20.11
14	0.05	0.36	1.19	2.70	5.02	8.18	12.16	16.88
15	0.04	0.30	0.97	2.22	4.15	6.81	10.20	14.27
16	0.03	0.24	0.81	1.85	3.47	5.73	8.63	12.16
17	0.03	0.20	0.67	1.55	2.93	4.86	7.36	10.42
18	0.02	0.17	0.57	1.32	2.50	4.15	6.32	8.99
19	0.02	0.15	0.49	1.13	2.14	3.58	5.46	7.81
20	0.02	0.13	0.42	0.97	1.85	3.10	4.75	6.81

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_o TO L_a or L_2

h	9	10	11	12	13	14	15	16
1	257.90	262.06	265.47	268.32	270.73	272.81	274.61	276.19
2	217.79	225.57	232.04	237.49	242.14	246.16	249.66	252.74
3	181.25	191.77	200.67	208.27	214.84	220.56	225.57	230.01
4	149.24	161.47	172.05	181.25	189.31	196.31	202.68	208.27
5	122.07	135.06	146.57	156.79	165.88	173.99	181.25	187.78
6	99.55	112.53	124.34	135.06	144.75	153.52	161.47	168.69
7	81.20	93.63	105.26	116.02	125.94	135.06	143.42	151.10
8	66.39	77.97	89.06	99.55	109.39	118.58	127.12	135.06
9	54.51	65.08	75.43	85.43	94.97	104.01	112.53	120.53
10	44.99	54.51	64.03	73.39	82.47	91.21	99.55	107.48
11	37.36	45.85	54.51	63.17	71.70	80.03	88.08	95.81
12	31.23	38.76	46.57	54.51	62.45	70.29	77.97	85.43
13	26.27	32.93	39.95	47.18	54.51	61.84	69.10	76.22
14	22.24	28.13	34.41	40.97	47.70	54.51	61.32	68.07
15	18.95	24.15	29.77	35.70	41.86	48.15	54.51	60.86
16	16.25	20.85	25.87	31.23	36.84	42.64	48.55	54.51
17	14.02	18.09	22.58	27.41	32.53	37.85	43.33	48.90
18	12.16	15.77	19.79	24.15	28.81	33.70	38.76	43.94
19	10.60	13.81	17.42	21.36	25.59	30.07	34.75	39.57
20	9.29	12.16	15.39	18.95	22.81	26.92	31.23	35.70

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_0 TO L_i or L_2

h	17	18	19	20	21	22	23	24
1	277.58	278.82	279.93	280.93	281.84	282.66	283.41	284.10
2	255.47	257.90	260.09	262.06	263.84	265.47	266.95	268.32
3	233.95	237.49	240.67	243.55	246.16	248.55	250.73	252.74
4	213.28	217.79	221.87	225.57	228.95	232.04	234.87	237.49
5	193.67	199.00	203.85	208.27	212.33	216.05	219.47	222.64
6	175.26	181.25	186.74	191.77	196.40	200.67	204.62	208.27
7	158.16	164.65	170.63	176.15	181.25	185.98	190.38	194.46
8	142.42	149.24	155.58	161.47	166.95	172.05	176.81	181.25
9	128.03	135.06	141.62	147.77	153.52	158.91	163.95	168.69
10	114.98	122.07	128.76	135.06	140.99	146.57	151.84	156.79
11	103.21	110.26	116.96	123.32	129.35	135.06	140.46	145.58
12	92.63	99.55	106.19	112.53	118.58	124.34	129.83	135.06
13	83.16	89.89	96.39	102.65	108.66	114.42	119.94	125.21
14	74.70	81.20	87.51	93.63	99.55	105.26	110.75	116.02
15	67.17	73.39	79.48	85.43	91.21	96.82	102.24	107.48
16	60.47	66.39	72.23	77.97	83.59	89.06	94.39	99.55
17	54.51	60.12	65.69	71.21	76.63	81.95	87.15	92.21
18	49.21	54.51	59.81	65.08	70.29	75.43	80.48	85.43
19	44.50	49.49	54.51	59.53	64.52	69.47	74.36	79.16
20	40.30	44.99	49.74	54.51	59.28	64.03	68.74	73.39

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_o TO L_1 or L_2

h	25	26	27	28	29	30	31	32
1	284.74	285.32	285.87	286.37	286.84	287.28	287.69	288.07
2	269.57	270.73	271.81	272.81	273.74	274.61	275.42	276.19
3	254.60	256.31	257.90	259.38	260.76	262.06	263.26	264.40
4	239.90	242.14	244.22	246.16	247.97	249.66	251.25	252.74
5	225.57	228.30	230.83	233.20	235.41	237.49	239.44	241.27
6	211.67	214.84	217.79	220.56	223.14	225.57	227.86	230.01
7	198.27	201.83	205.16	208.27	211.20	213.96	216.55	219.00
8	185.41	189.31	192.96	196.40	199.63	202.68	205.56	208.27
9	173.14	177.32	181.25	184.96	188.46	191.77	194.90	197.86
10	161.47	165.88	170.05	173.99	177.72	181.25	184.60	187.78
11	150.43	155.03	159.38	163.51	167.43	171.15	174.69	178.05
12	140.02	144.75	149.24	153.52	157.59	161.47	165.17	168.69
13	130.24	135.06	139.65	144.04	148.23	152.23	156.05	159.70
14	121.08	125.94	130.60	135.06	139.33	143.42	147.34	151.10
15	112.53	117.39	122.07	126.57	130.90	135.06	139.05	142.89
16	104.56	109.39	114.07	118.58	122.93	127.12	131.16	135.06
17	97.14	101.93	106.57	111.06	115.41	119.62	123.68	127.61
18	90.26	94.97	99.55	104.01	108.33	112.53	116.59	120.53
19	83.88	88.49	93.00	97.40	101.68	105.85	109.89	113.83
20	77.97	82.47	86.89	91.21	95.43	99.55	103.57	107.48

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_0 TO L_1 or L_2

h	33	34	35	36	37	38	39	40
1	288.43	288.77	289.09	289.40	289.68	289.95	290.21	290.45
2	76.91	277.58	278.22	278.82	279.39	279.93	280.45	280.93
3	265.47	266.47	267.42	268.32	269.16	269.97	270.73	271.46
4	254.15	255.47	256.72	257.90	259.02	260.09	261.10	262.06
5	242.99	244.62	246.16	247.62	249.00	250.31	251.56	252.74
6	232.04	233.95	235.77	237.49	239.12	240.67	242.14	243.55
7	221.31	223.50	225.57	227.54	229.41	231.18	232.87	234.48
8	210.85	213.28	215.60	217.79	219.88	221.87	223.76	225.57
9	200.67	203.33	205.87	208.27	210.57	212.75	214.84	216.83
10	190.80	193.67	196.40	199.00	201.48	203.85	206.11	208.27
11	181.25	184.31	187.21	189.99	192.64	195.17	197.60	199.92
12	172.05	175.26	178.32	181.25	184.06	186.74	189.31	191.77
13	163.20	166.54	169.74	172.80	175.74	178.55	181.25	183.85
14	154.71	158.16	161.47	164.65	167.70	170.63	173.44	176.15
15	146.57	150.12	153.52	156.79	159.94	162.97	165.88	168.69
16	138.80	142.42	145.89	149.24	152.47	155.58	158.58	161.47
17	131.40	135.06	138.59	142.00	145.29	148.47	151.53	154.50
18	124.34	128.03	131.60	135.06	138.39	141.62	144.75	147.77
19	117.64	121.35	124.94	128.42	131.79	135.06	138.22	141.29
20	111.28	114.98	118.58	122.07	125.46	128.76	131.95	135.06

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_o TO L_1 or L_2

h	41	42	43	44	45	46	47	48
1	290.69	290.91	291.12	291.32	291.51	291.70	291.88	292.04
2	281.40	281.84	282.26	282.66	283.05	283.41	283.77	284.10
3	272.15	272.81	273.44	274.04	274.61	275.16	275.68	276.19
4	262.97	263.84	264.67	265.47	266.22	266.95	267.65	268.32
5	253.87	254.95	255.98	256.96	257.90	258.80	259.67	260.50
6	244.88	246.16	247.38	248.55	249.66	250.73	251.76	252.74
7	236.02	237.49	238.89	240.23	241.52	242.75	243.94	245.07
8	227.30	228.95	230.53	232.04	233.49	234.87	236.21	237.49
9	218.73	220.56	222.30	223.97	225.57	227.11	228.59	230.01
10	210.34	212.33	214.22	216.05	217.79	219.47	221.09	222.64
11	202.14	204.27	206.31	208.27	210.16	211.97	213.71	215.39
12	194.13	196.40	198.58	200.67	202.68	204.62	206.48	208.27
13	186.33	188.73	191.02	193.24	195.36	197.42	199.39	201.30
14	178.75	181.25	183.66	185.98	188.22	190.38	192.46	194.46
15	171.39	173.99	176.50	178.92	181.25	183.51	185.68	187.78
16	164.26	166.95	169.55	172.05	174.47	176.81	179.07	181.25
17	157.36	160.12	162.80	165.38	167.88	170.29	172.63	174.89
18	150.69	153.52	156.26	158.91	161.47	163.95	166.36	168.69
19	144.26	147.14	149.93	152.64	155.26	157.80	160.27	162.66
20	138.07	140.99	143.82	146.57	149.24	151.84	154.35	156.79

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADING FROM L_o TO L_1 or L_2

h	49	50	51	52	53	54	55	56
1	292.21	292.36	292.51	292.66	292.79	292.93	293.06	293.18
2	284.43	284.74	285.04	285.32	285.60	285.87	286.12	286.37
3	276.67	277.14	277.58	278.01	278.43	278.82	279.21	279.58
4	268.96	269.57	270.16	270.73	271.28	271.81	272.32	272.81
5	261.29	262.06	262.79	263.50	264.18	264.83	265.47	266.08
6	253.69	254.60	255.47	256.31	257.12	257.90	258.66	259.38
7	246.16	247.21	248.22	249.19	250.13	251.03	251.90	252.74
8	238.72	239.90	241.04	242.14	243.20	244.22	245.21	246.16
9	231.37	232.69	233.95	235.18	236.35	237.49	238.59	239.64
10	224.13	225.57	226.96	228.30	229.59	230.83	232.04	233.20
11	217.01	218.56	220.07	221.52	222.91	224.27	225.57	226.84
12	210.01	211.67	213.28	214.84	216.34	217.79	219.20	220.56
13	203.13	204.91	206.62	208.27	209.87	211.42	212.92	214.37
14	196.40	198.27	200.08	201.83	203.52	205.16	206.74	208.27
15	189.81	191.77	193.67	195.50	197.28	199.00	200.67	202.28
16	183.37	185.41	187.39	189.31	191.16	192.96	194.71	196.40
17	177.08	179.20	181.25	183.25	185.18	187.05	188.86	190.62
18	170.95	173.14	175.26	177.32	179.32	181.25	183.14	184.96
19	164.98	167.23	169.41	171.53	173.59	175.59	177.53	179.42
20	159.17	161.47	163.71	165.88	168.00	170.05	172.05	173.99

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_o TO L_l or L_a

h	57	58	59	60	61	62	63	64
1	293.30	293.42	293.53	293.63	293.74	293.84	293.94	294.03
2	286.61	286.84	287.06	287.28	287.49	287.69	287.88	288.07
3	279.93	280.28	280.61	280.93	281.24	281.55	281.84	282.12
4	273.28	273.74	274.18	274.61	275.02	275.42	275.81	276.19
5	266.66	267.23	267.78	268.32	268.83	269.33	269.81	270.28
6	260.09	260.76	261.42	262.06	262.67	263.26	263.84	264.40
7	253.56	254.34	255.10	255.84	256.55	257.24	257.90	258.55
8	247.08	247.97	248.83	249.66	250.47	251.25	252.01	252.74
9	240.67	241.66	242.62	243.55	244.45	245.32	246.16	246.98
10	234.33	235.41	236.47	237.49	238.48	239.44	240.37	241.27
11	228.06	229.24	230.39	231.49	232.57	233.61	234.63	235.61
12	221.87	223.14	224.38	225.57	226.73	227.86	228.95	230.01
13	215.77	217.13	218.45	219.72	220.96	222.17	223.34	224.47
14	209.76	211.20	212.60	213.96	215.27	216.55	217.79	219.00
15	203.85	205.37	206.84	208.27	209.66	211.01	212.33	213.60
16	198.04	199.63	201.18	202.68	204.14	205.56	206.93	208.27
17	192.34	194.00	195.61	197.18	198.70	200.18	201.62	203.03
18	186.74	188.46	190.14	191.77	193.36	194.90	196.40	197.86
19	181.25	183.04	184.77	186.46	188.11	189.70	191.26	192.78
20	175.88	177.72	179.51	181.25	182.95	184.60	186.21	187.78

TABLE 18

PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_0 TO L_1 or L_2

h	65	66	67	68	69	70	71	72
1	294.12	294.21	294.30	294.38	294.46	294.54	294.62	294.70
2	288.25	288.43	288.60	288.77	288.93	289.09	289.25	289.40
3	282.40	282.66	282.92	283.17	283.41	283.65	283.88	284.10
4	276.55	276.91	277.25	277.58	277.91	278.22	278.53	278.82
5	270.73	271.17	271.60	272.01	272.42	272.81	273.19	273.56
6	264.94	265.47	265.98	266.47	266.95	267.42	267.87	268.32
7	259.18	259.79	260.38	260.95	261.51	262.06	262.58	263.10
8	253.46	254.15	254.82	255.47	256.11	256.72	257.32	257.90
9	247.78	248.55	249.30	250.03	250.73	251.42	252.09	252.74
10	242.14	242.99	243.82	244.62	245.40	246.16	246.90	247.62
11	236.56	237.49	238.39	239.26	240.11	240.94	241.75	242.53
12	231.04	232.04	233.01	233.95	234.87	235.77	236.64	237.49
13	225.57	226.64	227.69	228.70	229.68	230.64	231.58	232.49
14	220.17	221.31	222.42	223.50	224.55	225.57	226.57	227.54
15	214.84	216.05	217.22	218.36	219.47	220.56	221.61	222.64
16	209.58	210.85	212.08	213.28	214.46	215.60	216.71	217.79
17	204.39	205.72	207.01	208.27	209.50	210.70	211.87	213.00
18	199.28	200.67	202.02	203.33	204.62	205.87	207.09	208.27
19	194.25	195.69	197.10	198.46	199.80	201.10	202.37	203.61
20	189.31	190.80	192.25	193.67	195.05	196.40	197.72	199.00

TABLE 18

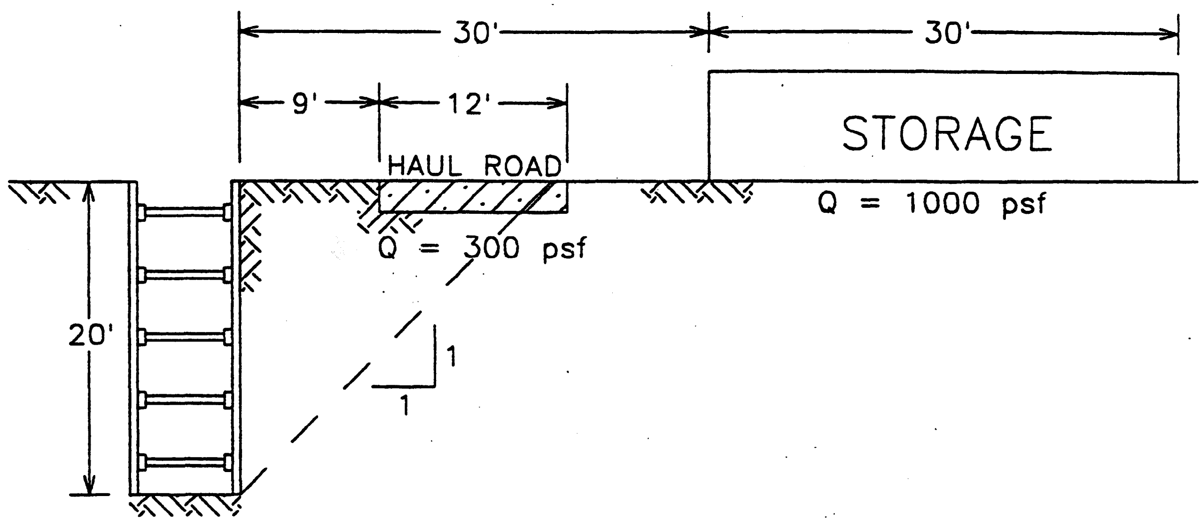
PRESSURE AT DEPTH h FOR UNIFORM LOADINGS FROM L_o TO L_1 or L_2

h	73	74	75	76	77	78	79	80
1	294.77	294.84	294.91	294.97	295.04	295.10	295.17	295.23
2	289.54	289.68	289.82	289.95	290.08	290.21	290.33	290.45
3	284.32	284.53	284.74	284.94	285.13	285.32	285.51	285.69
4	279.11	279.39	279.67	279.93	280.19	280.45	280.69	280.93
5	273.92	274.27	274.61	274.94	275.27	275.58	275.89	276.19
6	268.75	269.16	269.57	269.97	270.36	270.73	271.10	271.46
7	263.60	264.08	264.55	265.02	265.47	265.90	266.33	266.75
8	258.47	259.02	259.56	260.09	260.60	261.10	261.58	262.06
9	253.38	254.00	254.60	255.18	255.76	256.31	256.86	257.39
10	248.32	249.00	249.66	250.31	250.94	251.56	252.16	252.74
11	243.30	244.04	244.77	245.47	246.16	246.83	247.49	248.13
12	238.31	239.12	239.90	240.67	241.42	242.14	242.85	243.55
13	233.38	234.24	235.08	235.90	236.71	237.49	238.25	239.00
14	228.48	229.41	230.30	231.18	232.04	232.87	233.69	234.48
15	223.64	224.62	225.57	226.50	227.41	228.30	229.16	230.01
16	218.85	219.88	220.89	221.87	222.83	223.76	224.68	225.57
17	214.11	215.20	216.25	217.29	218.29	219.28	220.24	221.18
18	209.44	210.57	211.67	212.75	213.81	214.84	215.85	216.83
19	204.82	206.00	207.15	208.27	209.37	210.45	211.50	212.53
20	200.26	201.48	202.68	203.85	204.99	206.11	207.21	208.27

TABLE 18

SURCHARGES

EXAMPLE OF ALTERNATIVE SURCHARGE AND TABULAR VALUES:



1) Determine surcharge pressures at 5 foot increments of depth starting at the ground surface.

2) Compare tabular strip load values to the alternative loading.

Sample calculations for depth = 10 feet:

At haul road: σ_{10} = 140.99 - 44.99 = 96.00 psf
For building: σ_{10} = 1000/{300(237.49 - 181.25)} = 187.47 psf

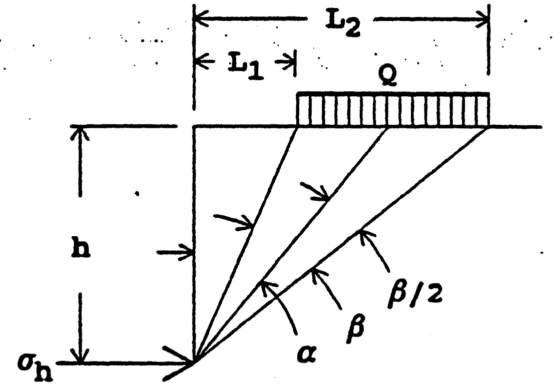
Building σ_{10} + Road σ_{10} = 283.47 psf
Building σ_{10} + Road @ 100 psf = 287.47 psf

COMBINE SURCHARGES:

Depth	Building σ	Road σ	Sum of σ's	Building σ + 100 psf
0	0.0	0.0	72.00 min.	100.00
5	102.77	90.26	193.03	202.77
10	187.47	96.00	283.47	287.47
15	244.03	72.26	316.29	344.03
20	272.33	49.99	322.32	372.33

CALCULATOR PROGRAMS FOR STRIP LOADS

BOUSSINESQ STRIP LOAD PROGRAM FOR HP 41CV



Relationship used:

$\alpha + \beta/2 + (\alpha - \beta/2) = 2\alpha$

$\alpha + \beta/2 - (\alpha - \beta/2) = \beta$

$\sigma_h = (2Q/\pi)(\beta_R - \sin\beta\cos2\alpha)$

To Initiate Program:

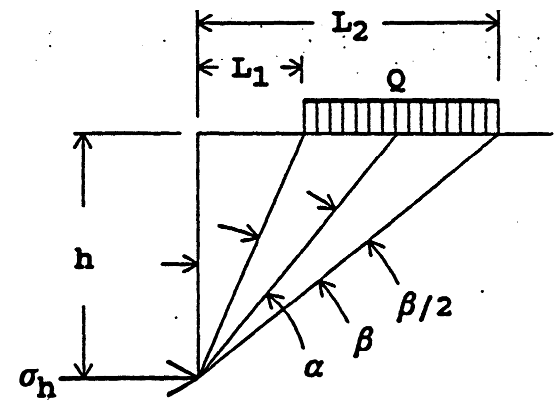
[ ] GTO [·] [·]
[PRGM]
[ ] LBL [ALPHA] BOUSNQ [ALPHA]
[ALPHA] h = ? [ALPHA]
XEQ [ALPHA] PROMPT [ALPHA]
STO 04
RCL 03
[X≷Y]
÷
[ ] TAN
STO 05
RCL 02
RCL 04
÷
[ ] TAN
STO 06
−
[ ] π
×
180
÷
RCL 05
RCL 06
+
COS
RCL 05
RCL 06
−
SIN
×

program continued:

CHS
+
2
×
RCL 01
×
[ ] π
÷
[ ] GTO [·] [·]
[PRGM]

To Run Program:

Store Q in 01
Store L_1 in 02
Store L_2 in 03
XEQ [ALPHA] BOUSNQ [ALPHA]
Calculator indicates h = ?
Enter h value
R/S
For next h:
R/S
Enter h value
R/S
etc.



BOUSSINESQ STRIP LOAD PROGRAM FOR HP 11C

Relationship used:

$\alpha + \beta/2 + (\alpha - \beta/2) = 2\alpha$

$\alpha + \beta/2 - (\alpha - \beta/2) = \beta$

Note that α never displays.
Also f & g refer to function keys

$\sigma_h = (2Q/\pi)(\beta_R - \sin\beta\cos 2\alpha)$

To Initiate Program:

f FIX 4 or 5
g DEG
ENTER q STO 0
ENTER L1 STO 1
ENTER L2 STO 2
g P/R
f CLEAR PRGM

f LBL A	001-42,21,11	
STO 3	002	44 3
RCL 2	003	45 2
$\boxed{X \gtrless Y}$	004	34
÷	005	10
g TAN	006	43 25
STO 5	007	44 5
ENTER	008	36
RCL 1	009	45 1
RCL 3	010	45 3
÷	011	10
g TAN	012	43 25
STO 4	013	44 4
−	014	30
f PSE (= β°)*	015	42 31
f π	016	42 16
×	017	20
1	018	1
8	019	8
0	020	0
÷	021	10
f PSE (= β°_R)*	022	42 31
ENTER	023	36
RCL 5	024	45 5

program continued:

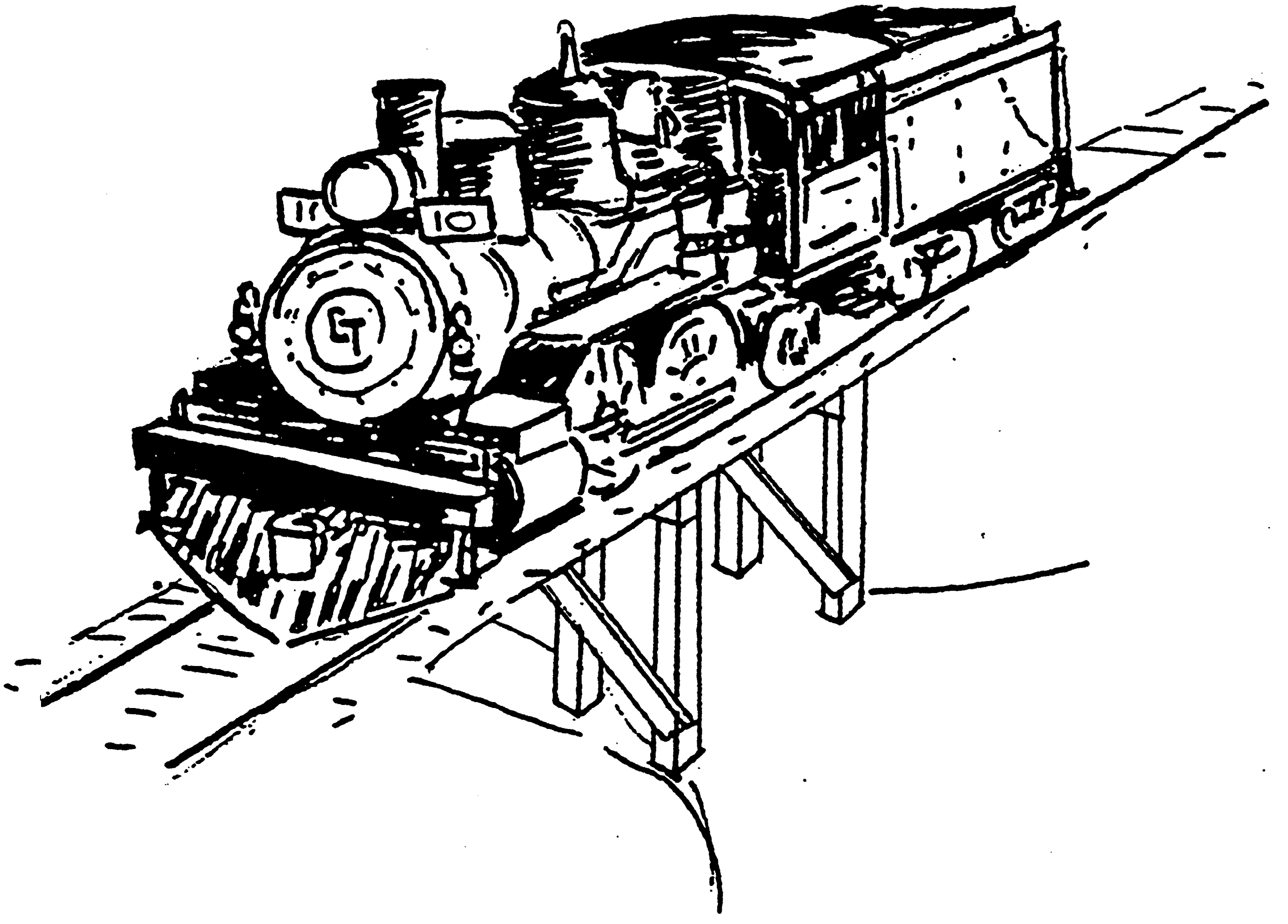
RCL 4	025	45 4
+	026	40
COS	027	24
f PSE (= COS2α)*	028	42 31
RCL 5	029	45 5
RCL 4	030	45 4
−	031	30
SIN	032	23
f PSE (= SINβ)*	033	42 31
×	034	20
CHS	035	16
+	036	40
2	037	2
×	038	20
RCL 0	039	45 0
×	040	20
f π	041	42 16
÷	042	10
g RTN	043	43 32
OFF = ON		

* Program will pause and show this value.

To Run Program:

(which only runs one h at a time)
Turn on (Not in Program Mode)
Enter desired h,
then $\boxed{f}$ then A = $[x]^{1/2}$

RAILROADS



RAILROAD REVIEW

Railroads present additional administrative and engineering problems, Any work adjacent to or over a railroad track, must be approved by the railroad concerned. The first step is review and approval of the Contractor% shoring plans; this must be donebefore any work can start at the jobsite. The Office of Structure Construction is the liason between the job and the railroad. The review and approval procedure is the same as it is for falsework plans. The Resident' Engineer or Bridge Representative submits the Contractor% shoring plans to the Sacramento Office of Structure Construction. A supplemental review is performed by the Office of Structure Construction in Sacramento and the shoring plans are transmitted to-therailroad for their review. The railroad replies to OSC Sacramento, which in turn will inform the Resident Engineer who approves to the Contractor. It is important that this procedure is followed strictly in order that we get approval in minimum time from the railroad. For normal shoring projects, the average railroad review time is about 6 weeks. It is important that plans are prepared and include additional features that railroads require such as noted clearances, horizontal and vertical.

Section 194.02, "Preservation of Property" of the Standard Specifications includes a provision stipulating that shoring plans be submitted at least 8 weeks before the Contractor intends to begin any excavation requiring shoring.

Most business is with the major railroads, Southern Pacific Transportation Co. (SPTC), Union Pacific System (UP), and the Atchison, Topeka & Santa Fe Railway Co (AT&SF). The SPTC have Supplemental Specifications to Part 20 of Chapter 8 of the AREA Manual for Railway Engineering in regard to shoring. These have been in effect since l2-12-75. All of the major railroads have agreed to accept that SPTC Specifications for review of shoring systems. For lesser railroads, such as *Sierra Railroad', use the SPTC Supplemental Specification unless there are specific instructions from the railroad concerned. <u>A complete cow of the Supplemental Specifications is in Appendix C of this manual.</u>

Limiting lines for no shoring required and trainmen walkway requirements are different for the AT&SF Railway- See sheet depicting slopes entitled 'Requirements for Excavation Shoring - The Atchison Topeka, and Santa Fe Railway Company' in this section. Exceptions can be made by the railroads.

The following is a summary of the SPTC Supplemental Specifications to Part 20 of Chapter 8 of AREA Manual for Railway Engineering:

1.1 This section establishes the minimum horizontal distance. that any part of a shoring system can be from center of the track, 8'-6" on tangents. To obtain a waiver of this requirement would require a very unusual situation.

1.2 This section restricts the type of shoring that can be used. Between 8'-6" and 10' from center of track (13' if excavation is in fill ground other than compaction controlled fill), the shoring must be a of a type that precludes the possibility of disturbance or loss of soil or base supporting the track. This means that progressive lagging (soldier beam) cannot be used; driven sheet piles or concrete walls with struts placed as excavation develops would be acceptable. The sheet piles or concrete wall would have to be placed between train movements, or during temporary shut-down of track.

1.4 Requirements for handrails. A walkway and standard handrail is required within 13' of centerline of track. This is for normal access of trainmen to track, not the protection of trench excavation as required by DOSH. Such walks and handrails are to be shown on the shoring plans.

2.1 Soil classification. The soil classification is to be shown on the plans. An equivalent classification to the AREA System is acceptable. Include groundwater conditions anticipated.

3.1
to
3.7 These sections pertain to minimum loads on earth retaining systems. Note that the railroad exempts strutted trenches -the earth pressure for such will be the minimums as required by DOSH or by calculation, using actual soil parameters; this procedure is discussed elsewhere in the manual. For flexible systems, such as cantilevered walls, use the minimum equivalent fluid pressure of 36 pcf, or the pressures calculated from actual soil properties. An exception to AREA is the railroad live load surcharge. SPTC has developed an empirical curve in lieu of the Boussinesq curve defined by AREA (CHART 3.6). This live load surchargecurve will be used for all earth retaining systems (Section 4.l as well as 3.l). restrained or flexible.

5. Section 5 deals with the allowable unit stresses and factors of safety. There are some differences from the policy given in this manual, however they are minor. Use controlling railroad allowable stresses when a railroad is involved.

6.1 This section pertains to the preparation of the shoring plans. A good clear well-engineered plan is the best way to get an early approval from the railroad. Be sure at pertinent minimum clearance dimensions are included. Railroads require that the shoring plan be prepared by a professional engineer.

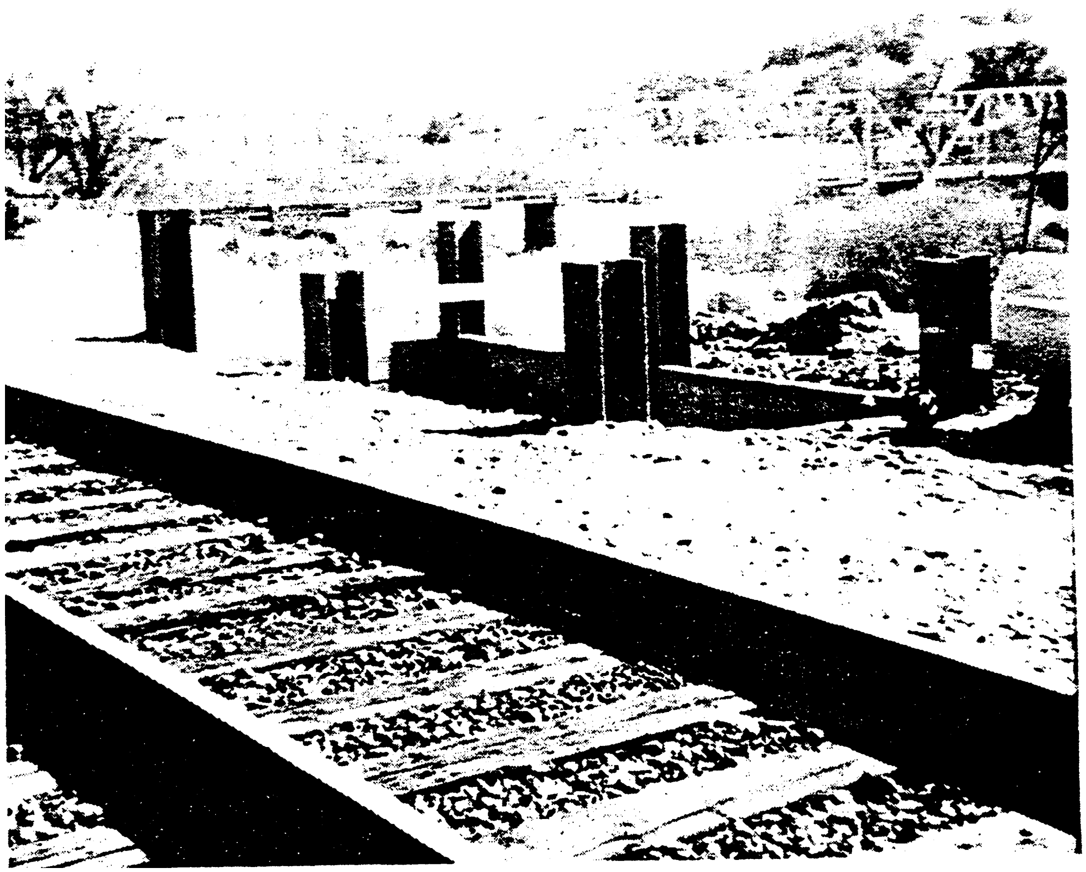
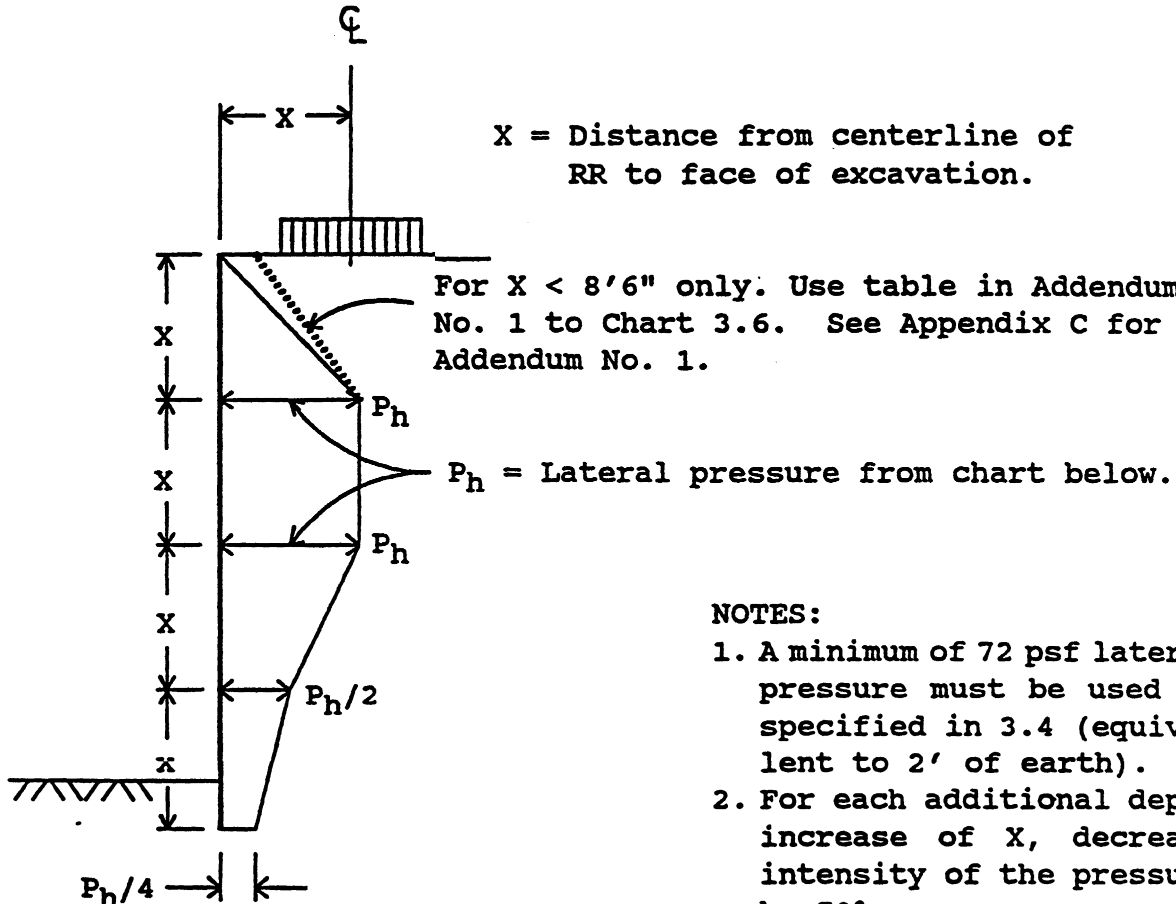
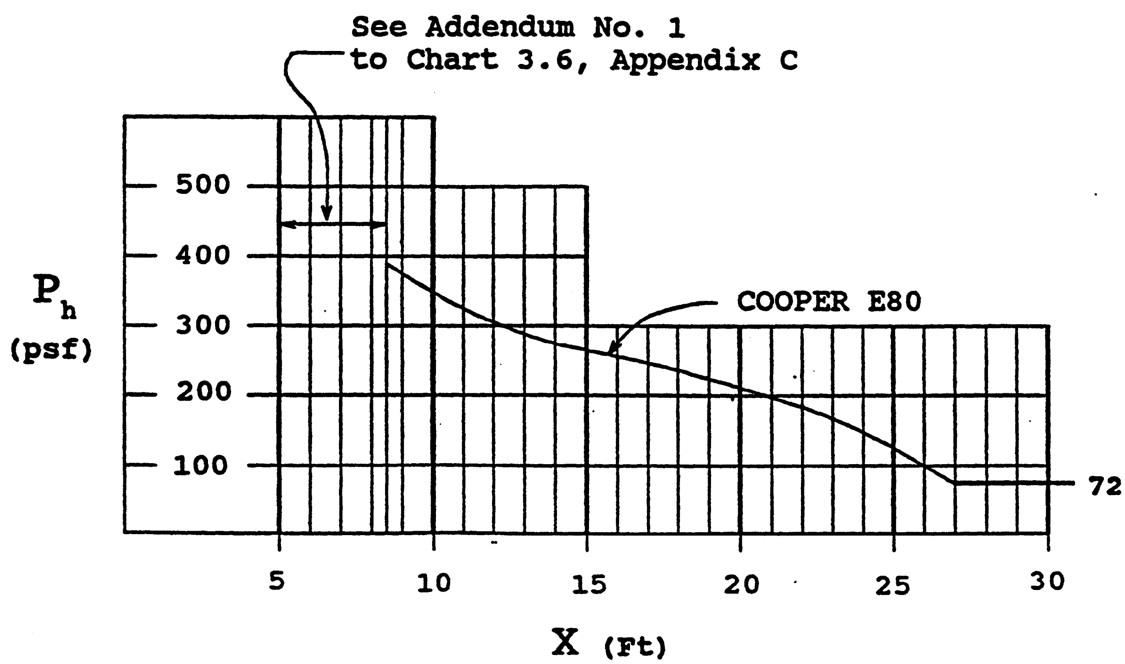


CHART 3.6 LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD

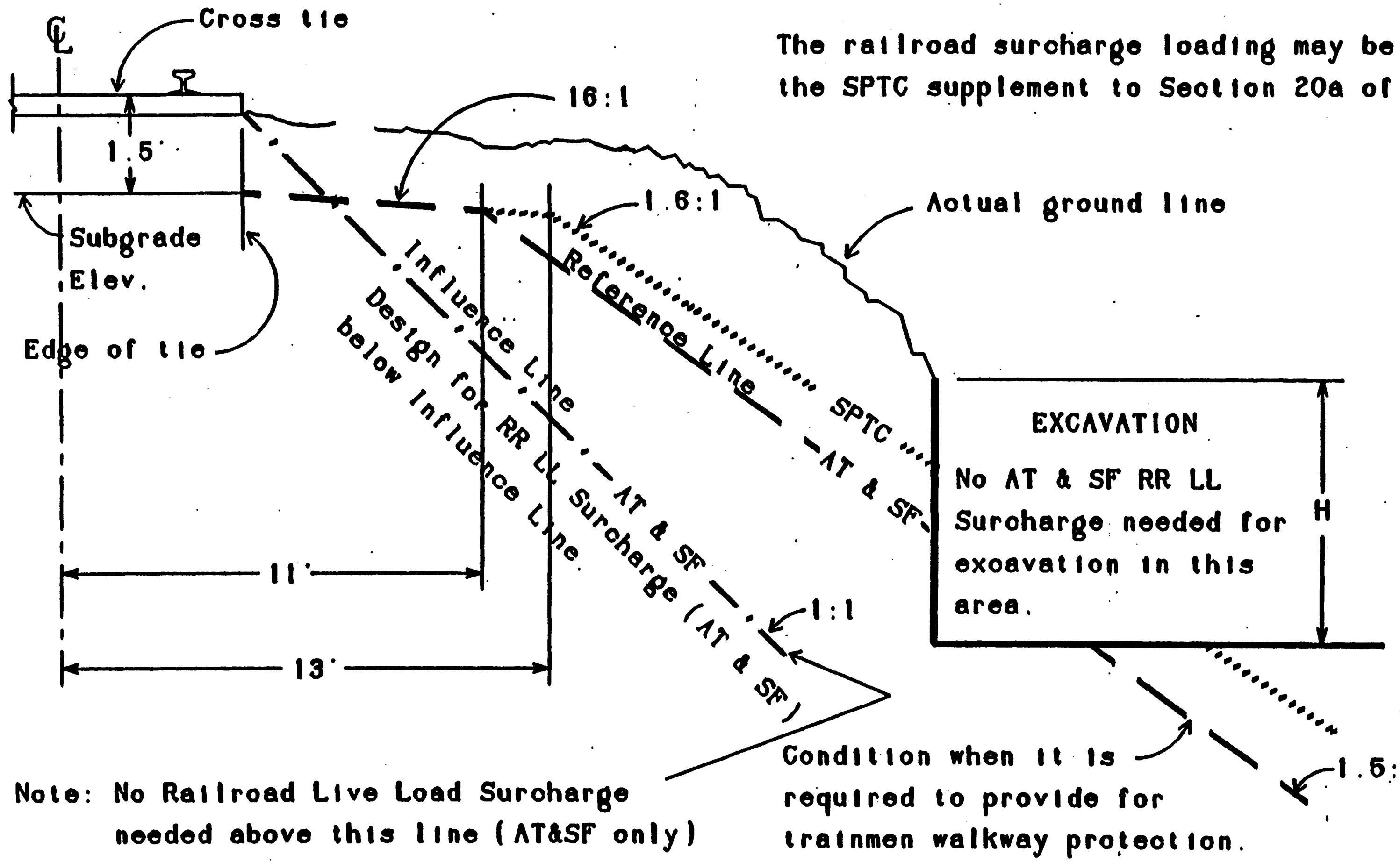


NOTES:

1. A minimum of 72 psf lateral pressure must be used as specified in 3.4 (equivalent to 2' of earth).
2. For each additional depth increase of X, decrease intensity of the pressure by 50%.

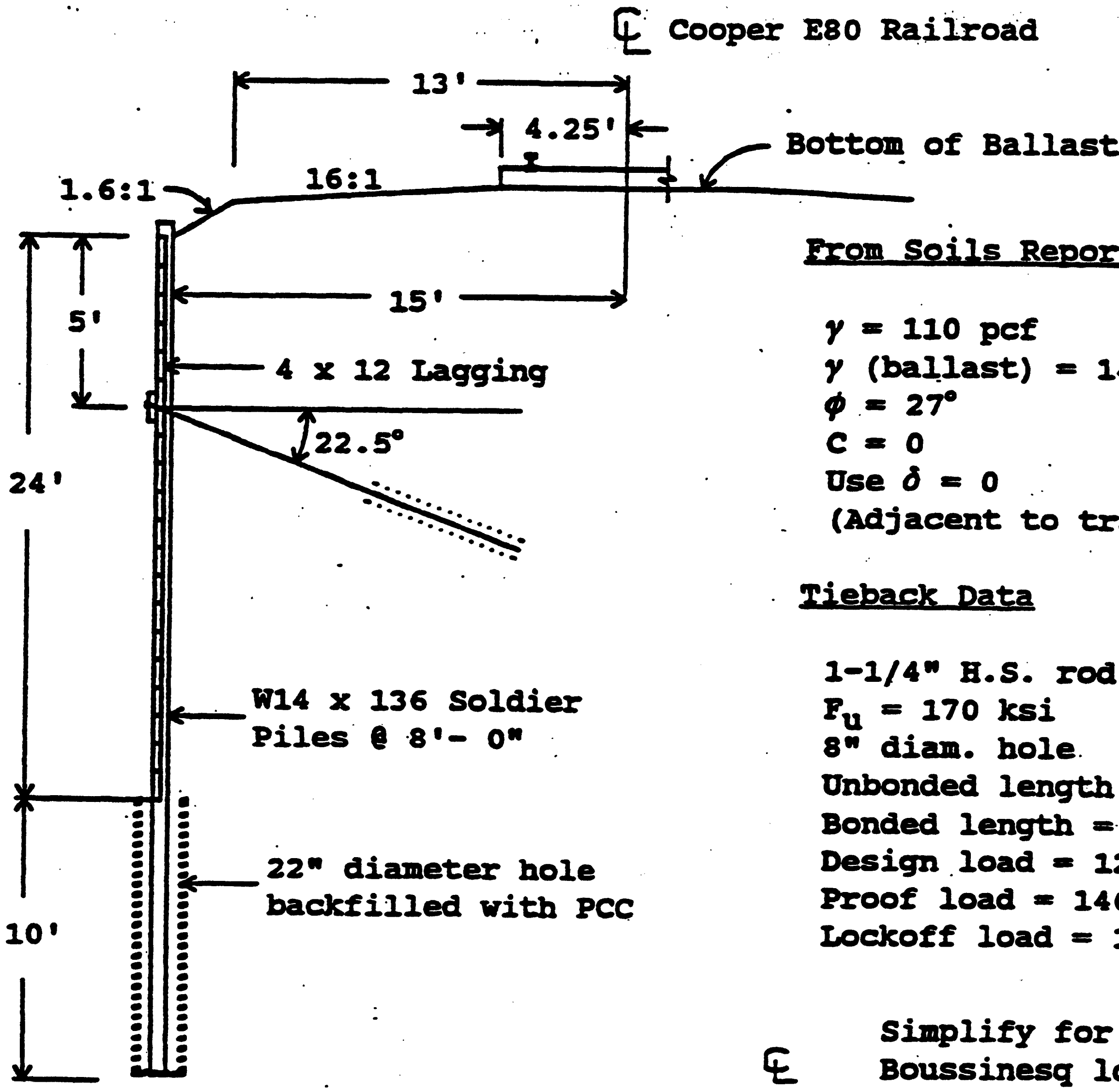


REQUIREMENTS FOR EXCAVATION SHORING - THE ATCHISON TOPEKA, AND SANTA FE RAILWAY COMPANY



Generally no shoring will be required if the bottom of excavation is above a railroad reference line and the ground will stand on it's own. Also, for a trench condition where H > 5', shoring is required per Cal-OSHA. The dotted line shows reference line for SPTC (Rev 8-30-74). The SPTC criteria applies to UPRR, WPRR, NWP, SD & AE. Railroads should be notified prior to start of work. Railroad approval will be required for any work within 15'- 0" of the center line of track. Shoring that intersects the AT & SF Influence Line must be designed to withstand railroad live load lateral forces. The Influence Line is not to be used for unsupported slope excavations.

SAMPLE PROBLEM NO. 9 - SOLDIER PILE WITH RAILROAD SURCHARGE

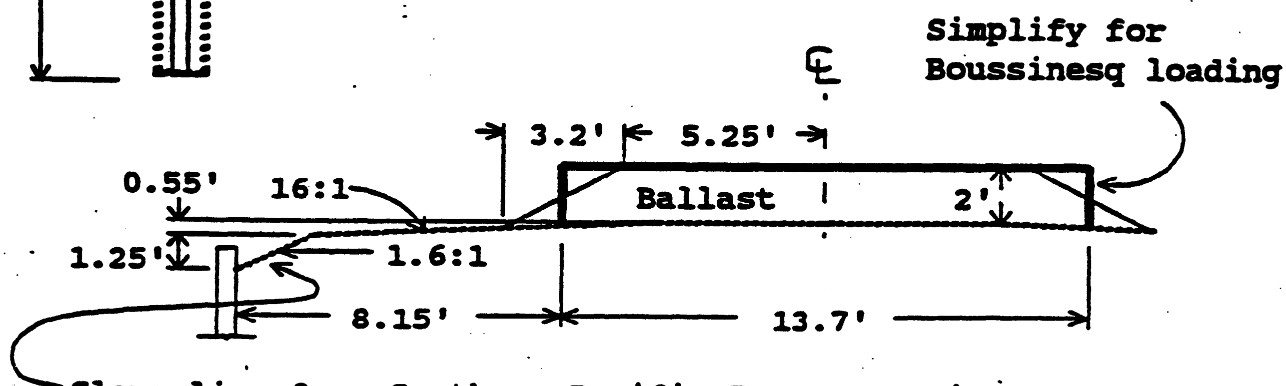


From Soils Report

γ = 110 pcf
γ (ballast) = 140 pcf
ϕ = 27°
C = 0
Use δ = 0
(Adjacent to tracks)

Tieback Data

1-1/4" H.S. rod @ 8'
F_u = 170 ksi
8" diam. hole
Unbonded length = 10'
Bonded length = 48'
Design load = 122 k
Proof load = 146 k
Lockoff load = 135 k



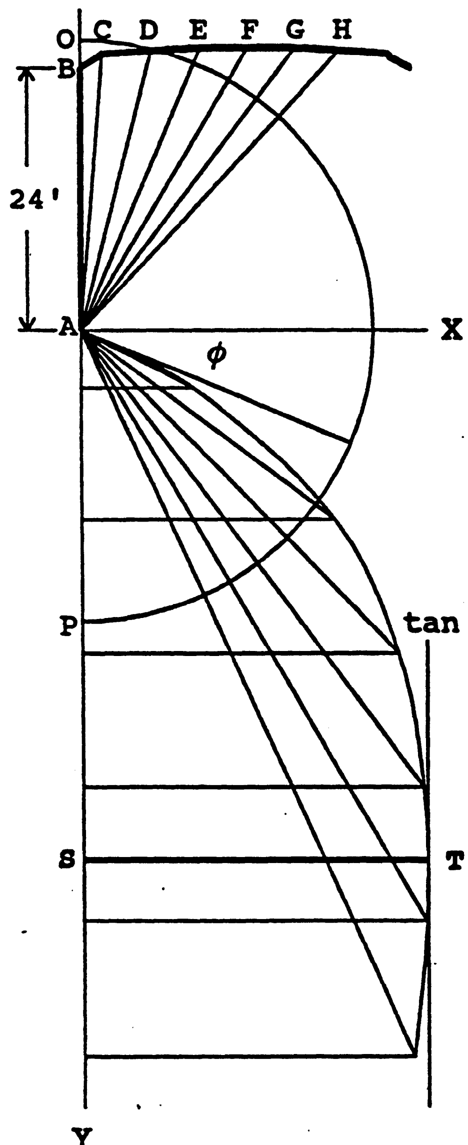
Slope line from Southern Pacific Transportation Company Engineering Common Standard Drawing No. C.S.582, dated August 30, 1984 (see previous page). Adequate shoring will be required for excavations below this line.

If the soil parameters have been determined from a qualified soil analysis, the values for ϕ & C must be reduced by 15%. This allows for the dynamic effects of train loadings on the retained material. If soil parameters have not been furnished, then the following minimum values must be used. These values are from the AREA Manual:

$Kw = 36$ pcf
$\gamma = 110$ pcf
$\phi = 30°$

Since a soils report was furnished, the values from the report will be used. Reduce ϕ by 15%. $(27°)(0.85) \approx 23°$

DETERMINE LATERAL SOIL PRESSURES (Use the Trial Wedge method)



Known:

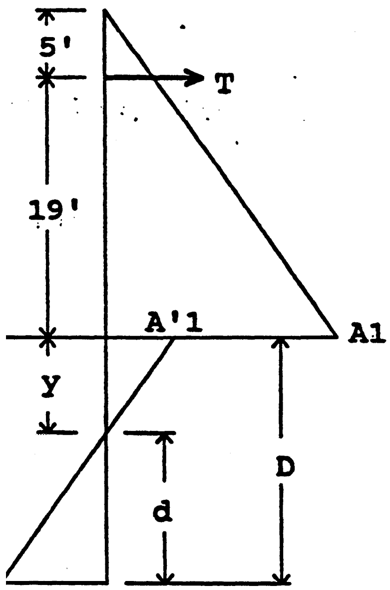
$\phi = 23°$
$\gamma = 110$ pcf
$\delta = 0$
$H = 24'$

WEDGE WEIGHT CALCULATIONS:

ABC = (110)(24.0) = 2,640 Lb/LF
ACD = (110)(55.0) = 6,050 Lb/LF
ADE = (110)(54.9) = 6,039 Lb/LF
AEF = (110)(55.0) = 6,050 Lb/LF
AFG = (110)(55.9) = 6,149 Lb/LF
AGH = (110)(55.8) = <u>6,138</u> Lb/LF

TOTAL: 33,066Lb/LF

ST = TOTAL LATERAL PRESSURE (P) = 15,700 Lb/LF



$P_{A1} = 2P/H = (2)(15{,}700)/24 = 1{,}308$ psf

Not a level surface, therefore cannot use standard equation or charts for K_a. Use:

$K_a \approx P_A/\gamma H \approx 1308/(110)(24) \approx 0.50$

For a level surface:

$K_p = \tan^2(45° + \phi/2)$
$= \tan^2(45° + 23/2) = 2.28$

Arching capability = 1.0

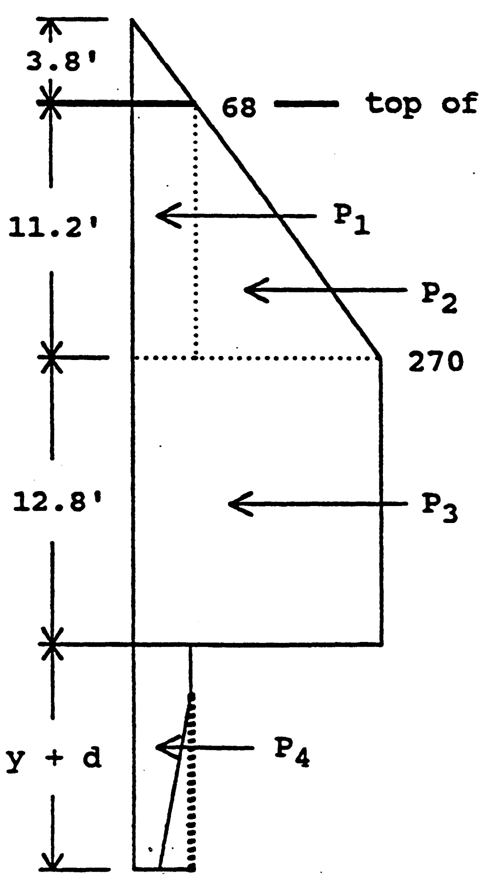
$f = (22/12)(1.0)/8 = 0.23$

$P_{A'1} = fP_{A1} = (0.23)(1{,}308) = 300.8$ psf

$= P_{A'1}/f\gamma(K_p - K_a) = 300.8/(0.23)(110)(2.28 - 0.50) = 6.68'$

$= f\gamma(K_p - K_a)d = (0.23)(110)(2.28 - 0.50)d = 45.0d$

<u>ilroad surcharge</u>



top of excavation

Note: The pressure diagram begins at the top of the ballast. Only the surcharge pressure below the top of excavation is used.

$P_1 = (68)(11.2) = 762$ Lb/LF

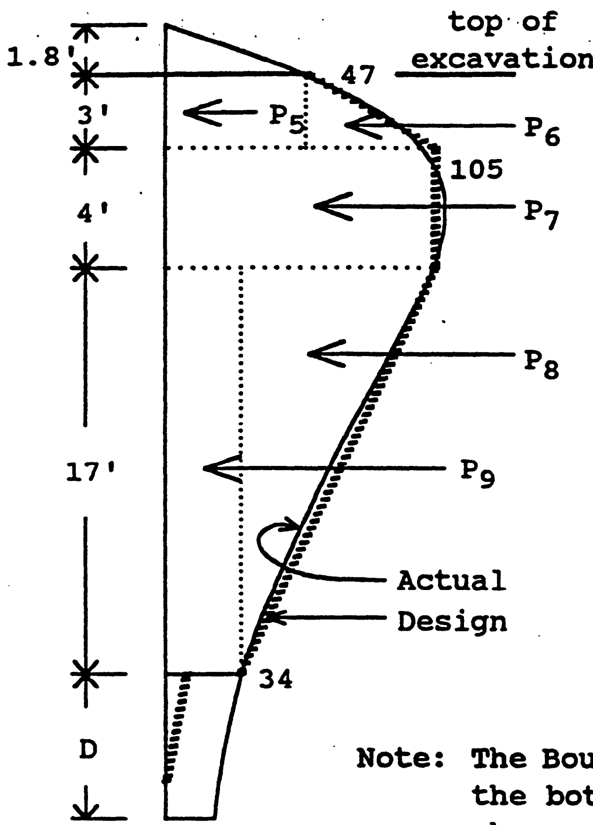
$P_2 = (270 - 68)(11.2)/2$
$= 1{,}131$ Lb/LF

$P_3 = (270)(12.8) = 3{,}456$ Lb/LF

Below excavation: Consider arching effect. Assume rectangular shape for simplicity.

$P_4 = (0.23)(270)(y + d)$
$= (0.23)(270)(6.68 + d)$
$= 414.8 + 62.1d$ Lb/LF

BALLAST SURCHARGE



$P_5 = (47)(3) = 141$ Lb/LF

$P_6 = (105 - 47)(3)/2$
$= 87$ Lb/LF

$P_7 = (105)(4) = 420$ Lb/LF

$P_8 = (105 - 34)(17)/2$
$= 604$ Lb/LF

$P_9 = (34)(17) = 578$ Lb/LF

Below excavation:
Consider arching effect.
(34)(0.23) = 7.8 psf (neglect)

Note: The Boussinesq pressure diagram begins at the bottom of the ballast. Only the surcharge pressure below the top of excavation is used. The equivalent soil height method could have been used for the ballast in lieu of the Boussinesq surcharge method.

DETERMINE D

$\Sigma M_T = 0$

Moment due to surcharge:

$$(414.8 + 62.1d)[19 + (6.68 + d)/2] + 762[11.2/2 - 5]$$
$$+ 1{,}131[(11.2)(2/3) - 5] + 3{,}456[(12.8/2) + 6.2]$$
$$- 141[3/2 + 2] - 87[3/3 + 2] + 420[0] + 604[17/3 + 2]$$
$$+ 578[17/2 + 2$$
$$M_{surch} = 31.1d^2 + 1{,}594.7d + 66{,}004.4 \text{ Ft-Lb/LF}$$

Moment due to soil:

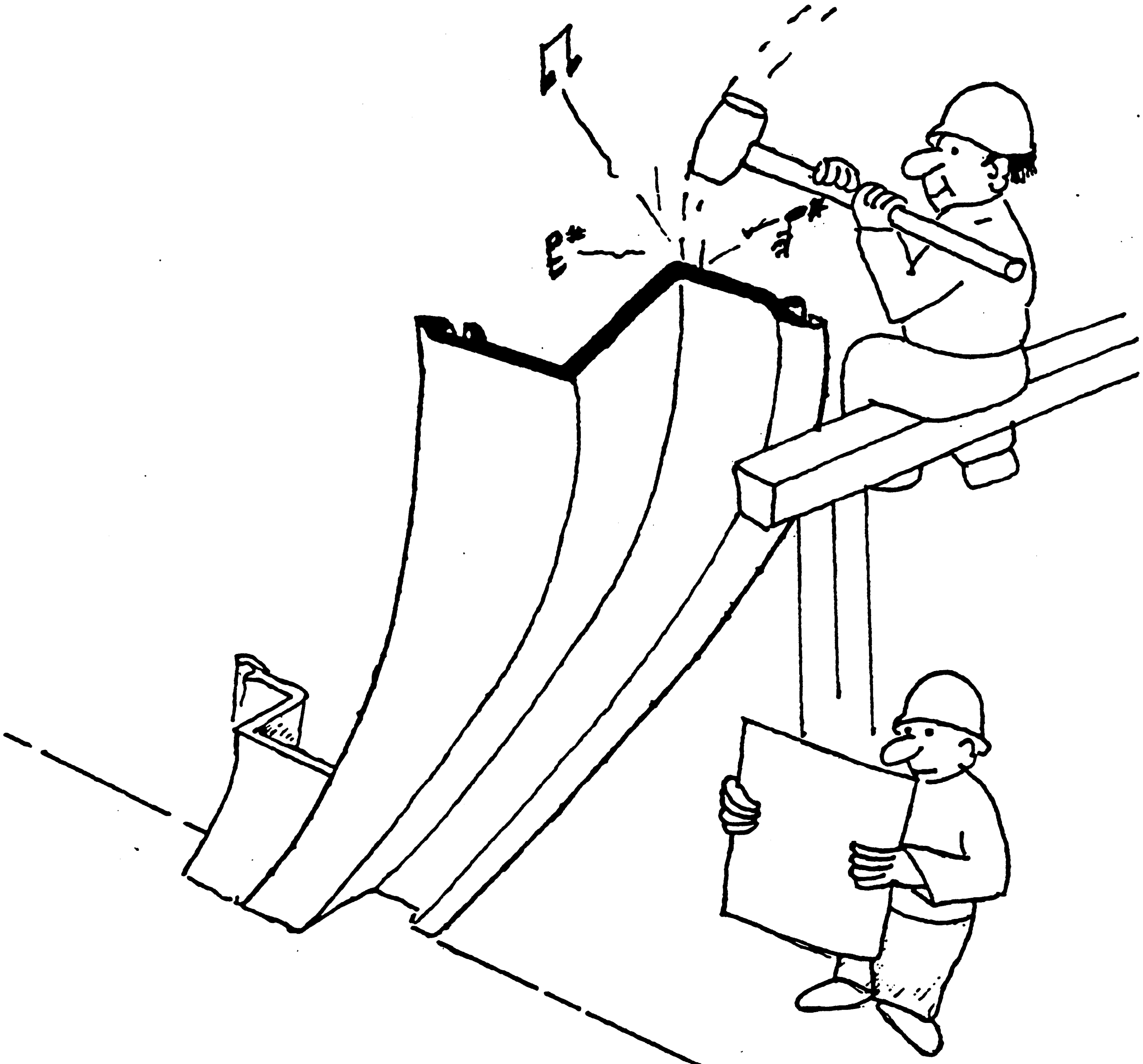
$$15{,}700[(24)(2/3) - 5] + \{(300.8)(6.68)/2\}[6.68/3 + 19]$$
$$- \{(45.0d)(d)/2\}[19 + 6.68 + (2/3)(d)]$$
$$M_{soil} = -15d^3 - 577.8d^2 + 194{,}025.8 \text{ Ft-Lb/LF}$$

Combined moment $= -15d^3 - 546.7d^2 + 1{,}594.7d + 260{,}030.2 = 0$
or $= d^3 + 36.4d^2 - 106.3d - 17{,}335.3 = 0$

By trial and error $d = 18.72'$ $\therefore D = d + y = 18.72 + 6.68 = 25.4'$

A low passive arching capability results in a high D value. For this example, an alternate design would be more practical. The remainder of this problem is not presented.

SHEET PILING



SHEET PILING

SHEET PILING WALLS

Cantilever sheetpiling walls depend on the passive resisting capacity of the soil below the depth of excavation to prevent overturning. The depth of sheetpiling walls below the bottom of the excavation are determined by using the difference between the passive and active pressures acting on the wall. The theoretical depth of pile penetration below the depth of excavation is obtained by equating horizontal forces and by taking moments about an assumed bottom of piling. The theoretical depth of penetration represents the point of rotation of the piling. Additional penetration is needed to obtain some fixity for the piling. Computed piling depths are generally increased 20% to 40% to obtain some fixity and to prevent lateral movement at the bottom of the piling.

It is not within the scope of this text to go into great detail concerning the design and analysis of sheet piling. A few of the more common situations complete with sample problems are presented on the following pages. A more adequate. and lengthy dissertation with example problems can-be found in the USS Steel Sheet Piling Design Manual.

The cohesive value of clay adjacent to sheet pile walls approaches zero with the passage of time. Design and analysis for clay soil conditions must generally meet the conditions of cohesionless soil design if the sheet piling support system is to be in use for more than a month. For those few cases where a clay analysiswill be appropriate, reference is made to the USS Steel Sheet Piling Design Manual.

It is possible to have negative pressure values with cohesive soils. Since cohesive soil adjacent to sheet pile walls loses its effective cohesion with the passage of time it is recommended that negative values be ignored. Do not use negative pressure values for the analysis of sheet piling systems. Any theoretical negative values should be converted to zero.

Friction:

The friction value at the soil-wall interface, or adhesion between the clay and the wall, should be ignored with sheet piling walls when the walls are in close proximity to pile driving or other vibratory operations - including functional railroad tracks. Similarly, above the depth of excavation, the cohesive value of the clay of a combined clay-sand soil should be ignored under the same circumstances.

Wall Stabilitv:

The stability of cantilever steel sheet pile walls will need to be considered in cohesive soils. The sheetpiling will fail if this height is exceeded. The stability number relates to kick out at the toe of the sheetpiling wall. Therefore, for design of sheetpiling walls in cohesive soils, the first step should be the investigation of the limiting height. A stability number S has been defined for this analysis as:

$$S = C/\gamma_e H'$$

and was derived from the net passive pressure in front of the wall in the term:

$$4C - \gamma_e H > 0$$

Teng found that adhesion of the cohesive soil to the sheets would allow modification to the stability equation and adjusted S from 0.25 to 0.31. A minimum stability number of 0.31 times an appropriate factor of safety could be used in design. However, when dynamic loadings near or at the sheets is considered (such as trains, pile driving operations, heavy vibrational motions, etc.) the adhesional effect must be excluded from the design and a stability number of S = 0.25 is to be used with an appropriate factor of safety in the height determination equation.

$$S(S.F.) \leq C/\gamma_e H'$$

$$\text{or:} \quad H' \leq C/\{S(S.F.)(\gamma_e)\}$$

Where:

- S = Stability number = 0.25 or 0.31
- S.F. = Safety factor (in the range of 1.25 to 1.50)
- γ_e = Effective density of the soil above the excavation line
- H' = H plus equivalent soil height of any uniform surcharge
- C = $q_u/2$
- q_u = unconfined compressive strength

Rakers:

When rakers are supported on the ground the allowable soil bearing capacity for the raker footing must be considered. Cohesionless soil having small internal friction angles (ϕ) will have lower soil bearing capacity. Additionally, when the footings are sloped relative to the ground surface reduced soil bearing capacities will result. A Department of the Navy publication (NAVFAC DM-7) includes reduction factors for footings near the ground surface. See the graphs entitled, "Ultimate Bearing Capacity of Continuous Footings With Inclined Loads" in Appendix B.

The NAVFAC figures assume that soil will be replaced over the footings. When such is the case a factor of D/B (bottom of footing distance below ground surface divided by the footing width) may be used. When the bottom of footing is at grade no D/B factor is to be used. Good judgement will be needed to determine an effective D/B value based on anticipated construction.

The safety factor of 3 for footings recommended by NAVFAC is generally considered to be for permanent installations. For short term shoring conditions a safety factor of 2 might be used. A reduced safety factor, however, could allow greater soil settlement, which in turn would permit additional outward wall rotation. Therefore, when wall deflection or rotation is not deemed critical a safety factor of 2 may be used for short term conditions.

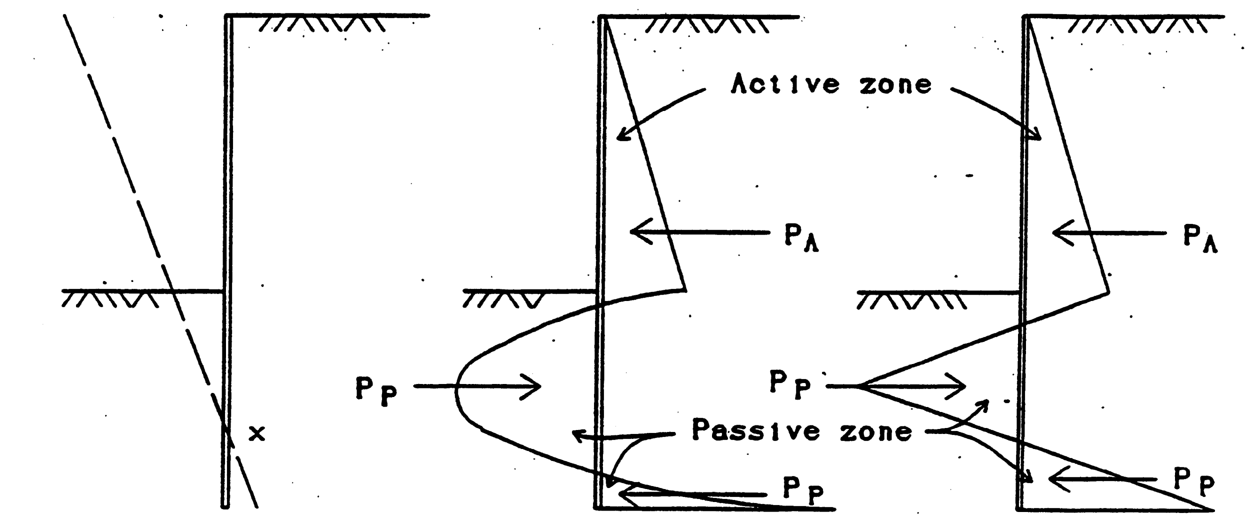
Tieback Walls:

See the Chapter on TIEBACKS for analysis of any tieback systems. Tieback sheetpiling wall sample problems are included in the tieback chapter.

Sample Problems:

Sample problems are included in this chapter to demonstrate the principles of sheetpiling design for both cohesionless and for cohesive soils. Additional soil pressure diagrams which relate to sheet piling are presented in the section on soldier piles.

CANTILEVER SHEET PILING - GRANULAR SOIL



Assumed elastic line of sheet piling with rotation about pt. x

Probable soil pressure distribution

Simulated pressure diagram (Granular soil, no ground water)

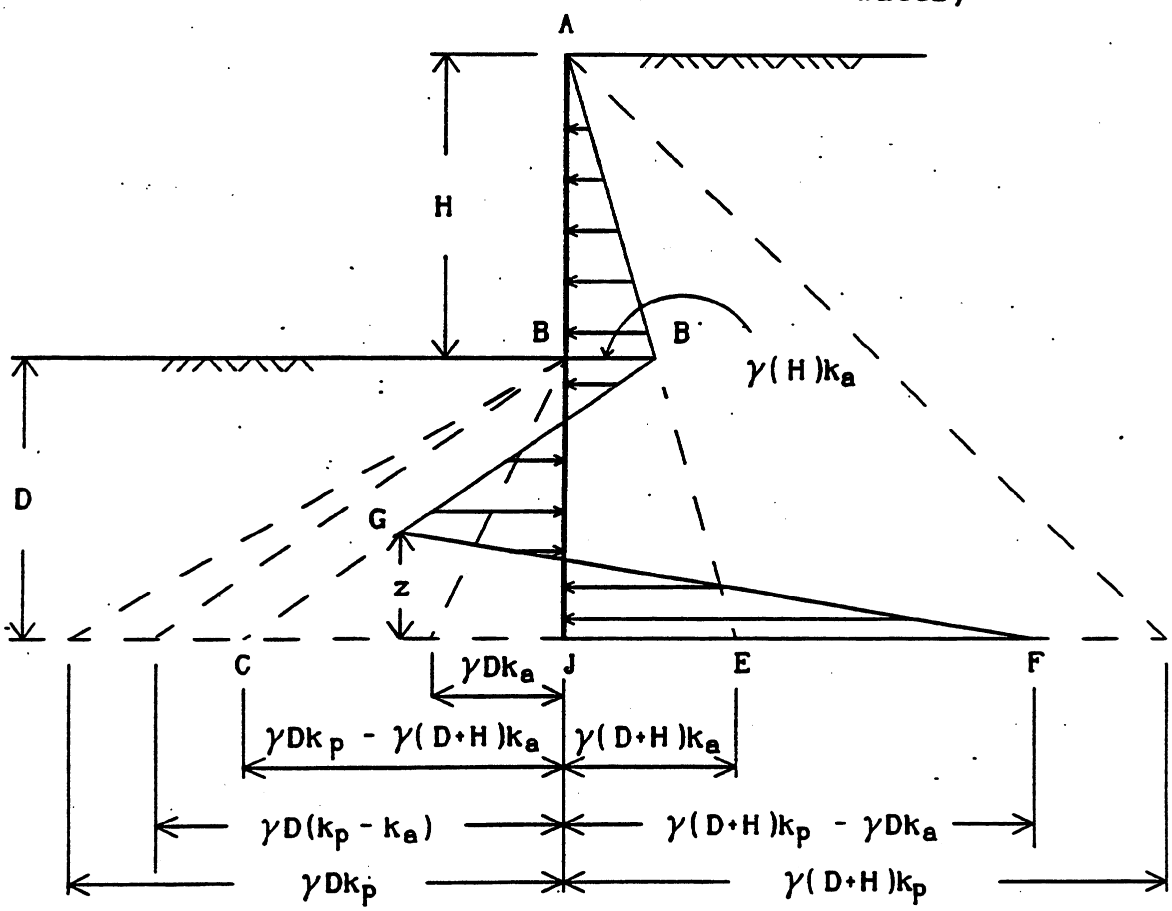


FIGURE 8-1

CANTILEVER SHEET PILING - GRANULAR SOIL (CONVENTIONAL METHOD)

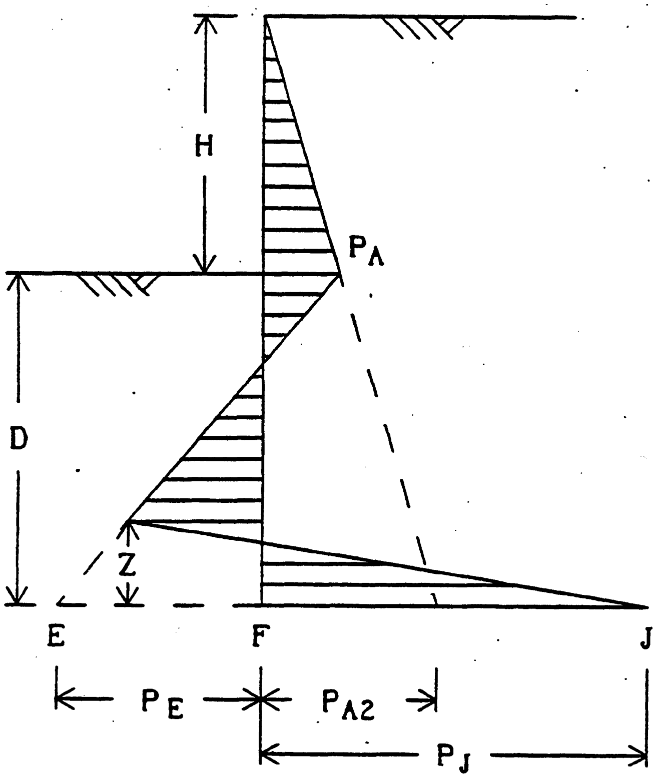


FIGURE 8-2

Granular soil.

No surcharge.

$P_A = \gamma H K_a = KwH$

$P_{A2} = \gamma(D + H)K_a$

$P_E = \gamma D K_p - \gamma(D + H)K_a$

$P_J = \gamma(D + H)K_p - \gamma D K_a$

$$\Sigma F_H = 0 = (H)(P_A)/2 + (P_A + P_{A2})(D)/2 + (P_E + P_J)(Z)/2 - (P_E + P_{A2})(D)/2$$

$$\therefore Z = \{(P_E + P_{A2})(D) - (H)(P_A) - (P_A + P_{A2})(D)\}/(P_E + P_J)$$
$$= \{(P_E - P_A)(D) - (H)(P_A)\}/(P_E + P_J)$$

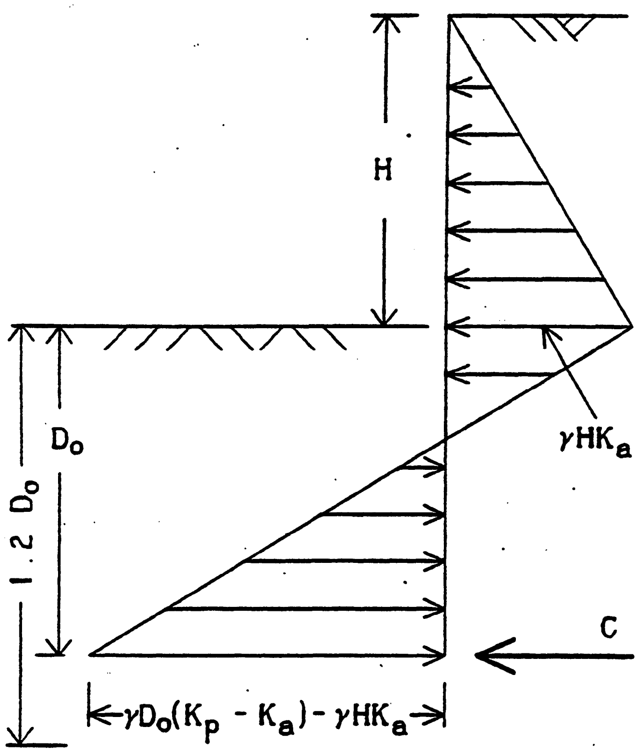
$$\Sigma M_F = 0$$
$$= \{(H)(P_A)/2\}[H/3 + D] + (P_A)(D)[D/2] + \{(P_{A2} - P_A)(D/2)[D/3]\} + \{(P_E + P_J)(Z)/2\}[Z/3] - \{(P_E + P_{A2})(D/2)\}[D/3]$$

$$\therefore Z^2 = \{(P_E - 2P_A)[D^2] - 3(H)(P_A)[H/3 + D]\}/(P_E + P_J)$$

Solve the two equations simultaneously for D (or use trial and error methods).

In most real situations there will be some sort of surcharge present. Simplifying the resulting pressure diagrams (using sound engineering judgement) should not alter the results significantly and will make the problems much easier to resolve. The surcharge pressures can be added directly to the soil diagram or may be drawn separately. Passive resistance may be initially reduced by dividing K_P by 1.5 to l.75, which will increase the moment requirement; or alternatively increase the computed D by 20% to 40% to fix the pile tips.

CANTILEVER SHEET PILING: GRANULAR SOILS
SIMPLIFIED METHOD[1] (After Teng)



Passive pressures on the right side of the sheet piling are replaced by a force C (Not used in the computations).

FIGURE 8-3

The simplified method is useful in the initial design of cantilever sheet piling in homogenous granular soils, but the conventional method must be used for final analysis.

To use the simplified method:

1. Determine K_a, K_p, and K_p/K_a (Log-spiral may be used).
2. Determine α (depth of water table in relation to H) as shown in Figure 8-4 on the following page.
3. From Figure 8-4 determine values for the ratios D/H (D_0/H), and $M_{max}/(\gamma' K_a H^3)$.
4. Compute D (D_0) and M_{max}.
5. Increase D (D_0) by 20% to 40%. Alternatively, K_p could be reduced initially by dividing by 1.5 to 1.75; however this will result in higher moment requirements.

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[1]Taken from USS Sheet Piling Design Manual (1984) pgs 20-23.

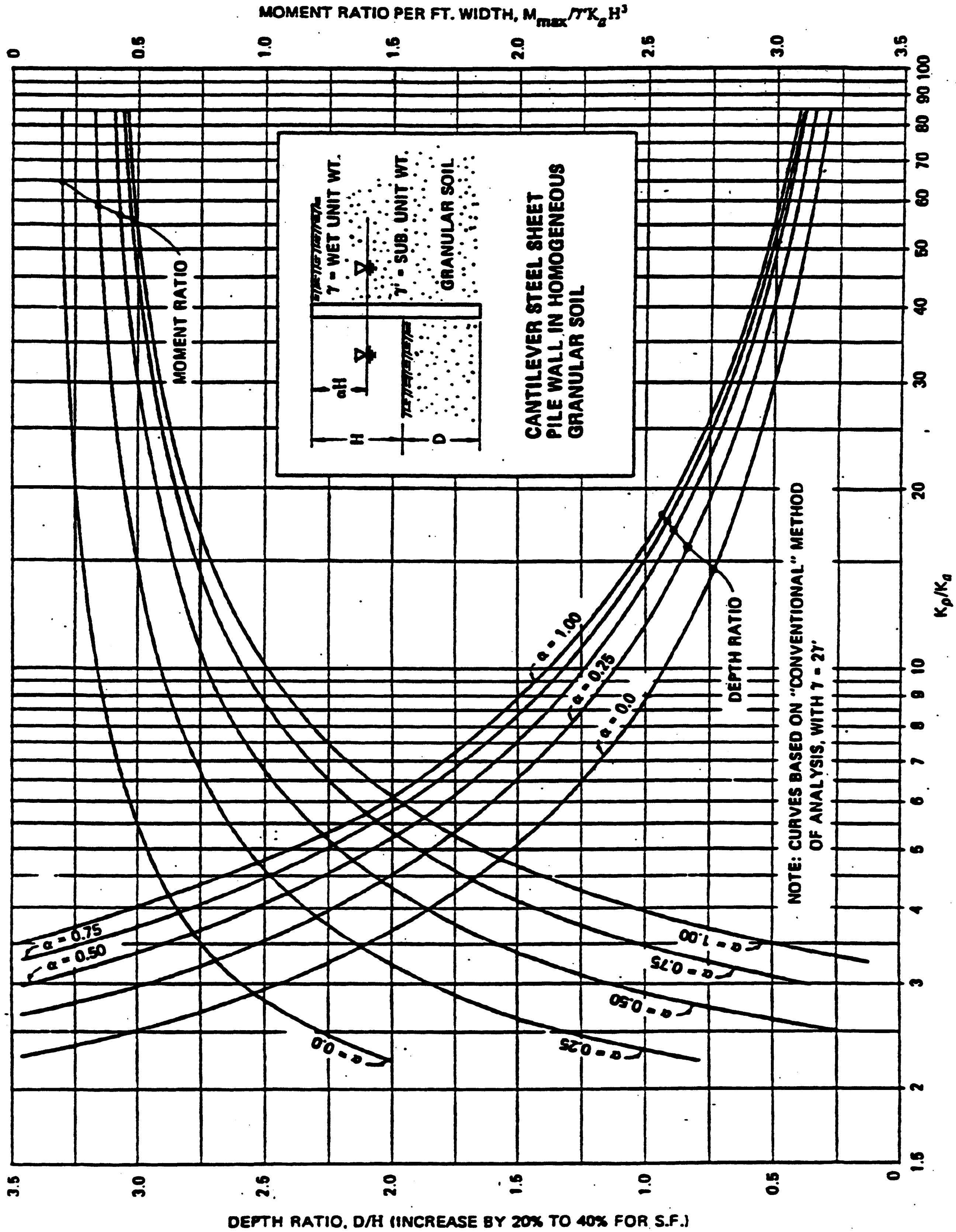
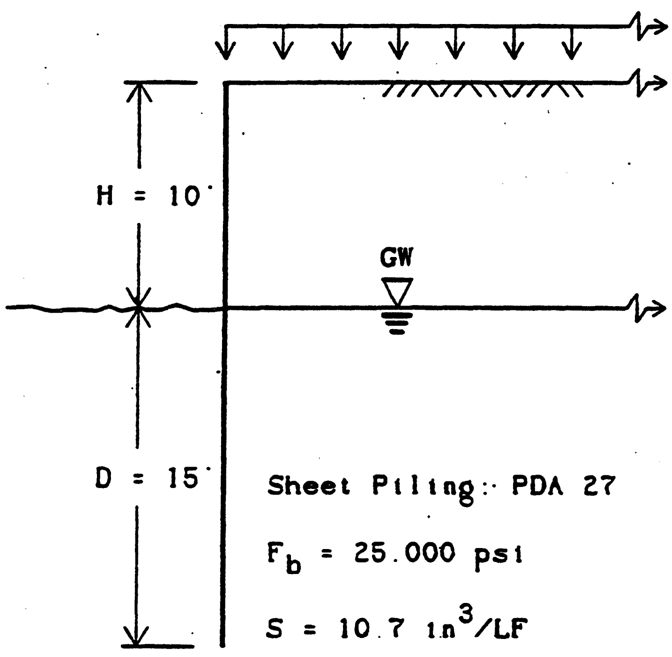

MOMENT RATIO PER FT. WIDTH, $M_{max}/\gamma' K_a H^3$
DEPTH RATIO, D/H (INCREASE BY 20% TO 40% FOR S.F.)
K_p/K_a
CANTILEVER STEEL SHEET PILE WALL IN HOMOGENEOUS GRANULAR SOIL
γ = WET UNIT WT.
γ' = SUB. UNIT WT.
GRANULAR SOIL
αH
H
D
MOMENT RATIO
DEPTH RATIO
NOTE: CURVES BASED ON "CONVENTIONAL" METHOD OF ANALYSIS, WITH γ = 2γ'
α = 1.00
α = 0.75
α = 0.50
α = 0.25
α = 0.0

FIGURE 8-4

SAMPLE PROBLEM 8-1: CANTILEVER SHEET PILING

This problem serves as a comparison between the Simplified Method (after Teng) and the conventional procedure when a surcharge load is included.

Given:



Uniform soil surcharge:
Q = 300 psf

γ = 120 pcf

$\gamma' = \gamma_{sub} = 0.6\gamma = 72$ pcf

$\phi = 30°$

Use friction angle $\delta = 0°$

FIGURE 8-5

Solution:

Use the log-spiral curves (Figure 8) to determine K_a and K_p.

$\beta/\phi = 0$ and $\delta/\phi = 0$

K_p for level surface ≈ 6.4R

R from the table = 0.467
$K_p \approx 6.4(0.467) = 3.0$

$K_a \approx 0.33$ $K_p - K_a = 3.0 - 0.33 = 2.67$

Since the surcharge load cannot be handled in the usual manner, an equivalent H_S will be calculated and added to the existing H.

$H_S = Q/\gamma = 300/120 = 2.5'$

Adjusted $H = H' = 10.0 + 2.5 = 12.5'$

SIMPLIFIED METHOD

Find D/H from Figure 8-4 (which assumes γ' is equal to $\gamma/2$) using $K_p/K_a = 3.0/0.33 = 9.1$

The water/excavation depth ratio (α) = 8/8 = 1.0
Depth ratio = D/H = 1.5, and moment ratio $M_{max}/\gamma' K_a H^3 = 1.1$
Then D = 1.5H = 1.5(10.5) = 15.75′
Increase D by 30% since K_p was not reduced initially.
$\therefore$ D = 1.3(15.75) = 20.5′ > 15′ shown on the shoring plan.

$M_{max} = 1.1\gamma' K_a H^3 = 1.1(120/2)(0.33)(10.5)^3 = 25{,}213$ Ft-Lb

S required = $M/f_s = 25{,}213(12)/25{,}000 = 12.1 > 10.7$ in^3

$\therefore$ Must use sheet pile with greater S or higher grade of steel.

CONVENTIONAL METHOD

γ = 120 PCF

γ' = 72 PCF

ϕ = 30°

δ = 0°

K_a = 0.33

K_p = 3.0

$K_p - K_a$ = 2.67

H = 8′

H′ = 10.5′

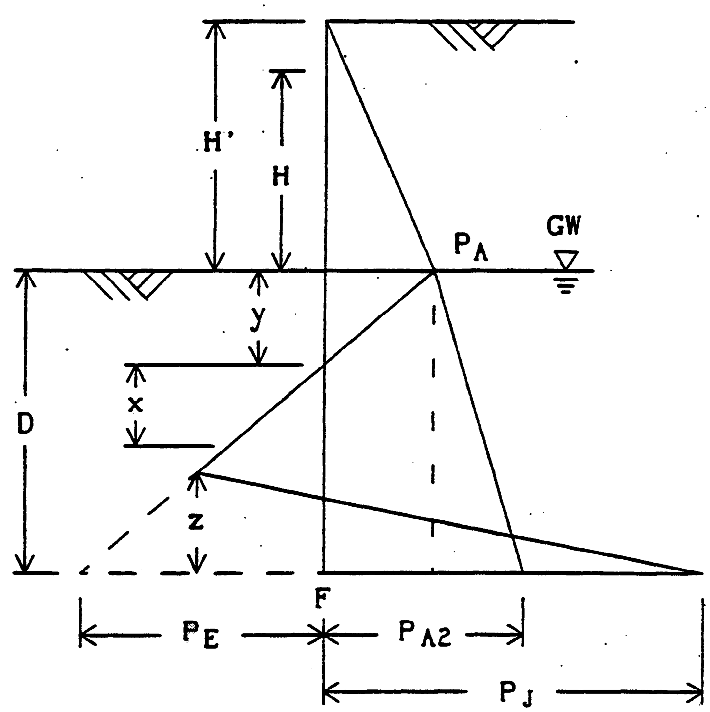


FIGURE 8-6

$P_A = \gamma H' K_a = 120(10.5)(0.33) = 416$ psf

$P_{A2} = P_A + \gamma' D K_a = 416 + 72(0.33)D = 416 + 24D$ psf

$P_E = \gamma' D(K_p - K_a) - P_A = 72(2.67)D - 416 = 192D - 416$ psf

$P_J = \gamma' D(K_p - K_a) + \gamma H' K_p = 72(2.67)D + 120(10.5)(3.00)$

$= 192D + 3{,}780$ psf

$$\Sigma F_H = 0$$
$$= H'(P_A)/2 + D(P_A + P_{A2})/2 + Z(P_E + P_J)/2 - D(P_E + P_{A2})/2$$

$$\therefore Z = [D(P_E + P_{A2}) - H'(P_A) - D(P_A + P_{A2})]/(P_E + P_J)$$

$$[D(192D - 416 + 24D + 416) - 10.5(416) - D(416 + 416 + 24D)]/(192D - 416 + 192D + 3,780)$$

$$= (D^2 - 4.33D - 22.75)/(2D + 17.52)$$

$$\Sigma M_F = 0$$
$$= \{H'(P_A)/2\}[D + H'/3] + D(P_A)[D/2] + \{D(P_{A2} - P_A)/2\}[D/3] + \{Z(P_E + P_J)/2\}[Z/3] - \{D(P_E + P_{A2})/2\}[D/3]$$

$$= \{10.5(416)/2\}[D + 3.5] + 416[D^2/2] + \{D(416 + 24D - 416)/2\}[D/3] + \{Z(192D - 416 + 192D + 3,780)/2\}[Z/3] - \{D(192D - 416 + 416 + 24D)/2\}[D/3]$$

$$0 = D^3 - 6.5D^2 - 68.25D - 2DZ^2 - 17.52Z^2 - 238.88$$

By trial and error $D \approx 14.01'$ and $Z \approx 2.48'$
Increase D by 30% for a safety factor: $1.3(14.01) = 18.2 > 15'$

<u>Find Maximum Moment:</u>

Locate point of zero pressure (At distance x below y):

$$y = P_A/\gamma'(K_p - K_a) = 416/(72)(2.67) = 2.16'$$

$$\Sigma P_A \text{ at } y = (H' + y)(P_A)/2 = (10.5 + 2.16)(416)/2 = 2,633 \text{ Lb}$$

Distance x where $\Sigma P_A = \Sigma P_p$: $2,633 = [\gamma'(K_p - K_a)x^2]/2$

Solve for x : $x = 5.23'$ (This is the plane of zero shear)

$$M_x = \{H'(P_A)/2\}[H'/3 + y + x] + \{y(P_A)/2\}[2y/3 + x] - 2,633[x/3]$$

$$= 2,184[10.89] + 449.28[6.67] - 2,633[1.74] = 22,199 \text{ Ft-Lb}$$

S Required = $22,199(12)/25,000 = 10.66 < 10.7 \text{ in}^3$

Indicating the sheet piling is adequate.

Maximum moment derived by simplified method (25,213) is more than the moment derived by the conventional method (22,199).

<u>Conclusion:</u>

The simplified method does not appear very accurate for determining moment when surcharge loads are involved.

SAMPLE PROBLEM 8-2: STEEL SHEET PILING WITH RAKER

Given:

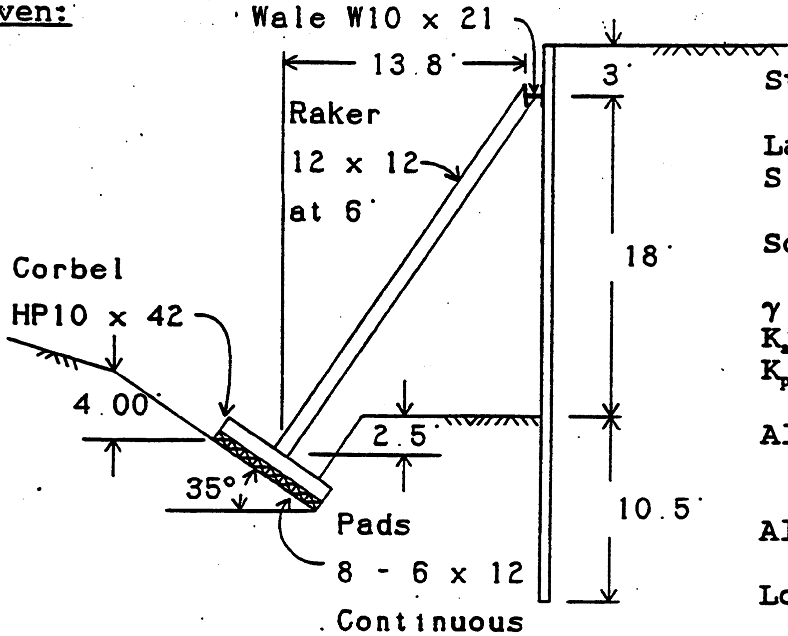


FIGURE 8-7

Steel Sheet Pile:

Larssen IIn
$S = 20.5\ in^3/LF$

Soil Properties:

$\gamma = 110$ pcf $\phi \approx 29°$
$K_a = 0.35 \quad K_p = 2.88$
$K_p - K_a = 2.53$

Allowable Soil Bearing Capacity = 1 tsf

All Lumber = Rough size

Low Risk Condition

Solution:

$Kw = K_a\gamma = 0.35(110) = 38$ pcf

Use minimum surcharge = 72 psf

$P_A = 0.71KwH = 0.71(38)(21) = 566.6$ psf
$0.2H = 0.2(21) = 4.2'$

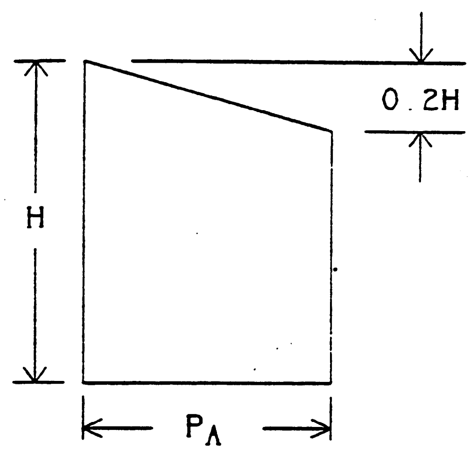


FIGURE 8-8

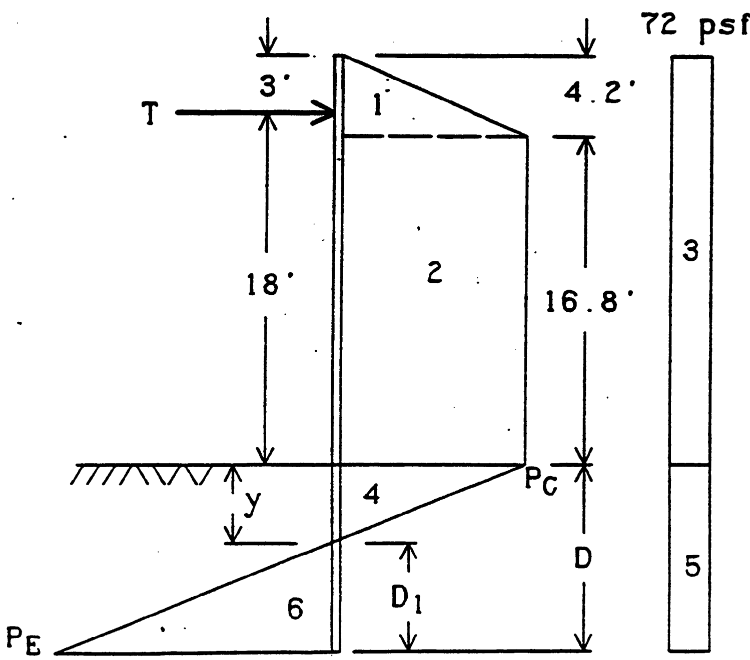


FIGURE 8-9

$P_A = 566.6$

$y = P_A/\gamma(K_p - K_a)$

$y = 566.6/(110)(2.53)$
$= 2.04'$

$P_E = \gamma(K_p - K_a)D_1$

$P_E = 110(2.53)D_1$

$P_E = 278.3D_1$

Determine d_1 by taking moments about T:

Moment Arms		Areas	
1. $\{2(4.2)/3\} - 3$	$= -0.2$	1. $566.6(4.2)/2$	$= 1189.86$
2. $1.2 + 16.8/2$	$= 9.6$	2. $16.8(566.6)$	$= 9518.88$
3. $\{21/2\} - 3$	$= 7.5$	3. $21(72)$	$= 1512.00$
4. $18 + \{2.04/3\}$	$= 18.68$	4. $2.04(566.6)/2$	$= 577.93$
5. $18 + (y + D_1)/2$ $= 19.02 + D_1/2$		5. $72(2.04 + D_1)$ $= 146.88 + 72D_1$	
6. $20.04 + 2D_1/3$		6. $-278.3D_1^2/2$	$= -139.15D_1^2$

$$\Sigma_{Areas} = 12,945.55 + 72D_1 - 139.15D_1^2$$

Moment-Areas

1. -237.97
2. $91,381.25$
3. $11,340.00$
4. $10,795.73$
5. $2,793.66 + 1,442.88D_1 + 36D_1^2$
6. $-2,787.17D_1^2 - 92.77D_1^3$

$$\Sigma = 116,072.67 + 1,442.88D_1 - 2,751.17D_1^2 - 92.77D_1^3 = 0$$

By trial and error, or other means: $D_1 = 6.13'$
$D = D_1 + y = 6.13 + 2.04 = 8.17'$

Add 30% to D: $1.3(8.17) = 10.6' \approx 10.5'$ shown on plan

Determine T:

T = Active Pressures - Passive Pressures = Σ_{Areas}

$= 12,945.55 + 72D_1 - 139.15D_1^2$

$= 12,945.55 + 72(6.13) - 139.15(6.13)^2$

$= 8158$ Lb/LF

Determine Maximum Moment And Section Modulus Required:

P_A at T = 72 + (3/4.2)(566.6) = 476.71 psf
Area = 3(72) + 3(476.71 - 72)/2 = 823.07 Lb/LF
Area at 4.2′ = 72(4.2) + 4.2(566.6)/2 = 1,492.26 Lb/LF

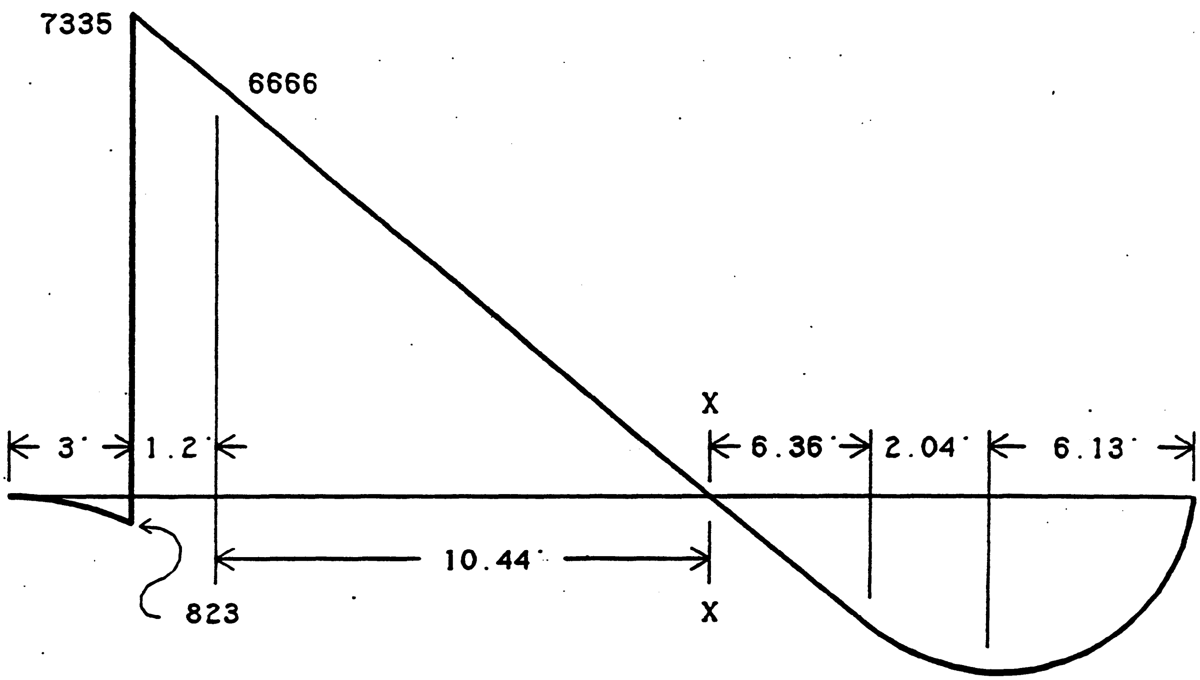
1492.26 - 823.07 = 669.19 Lb/LF ≈ 669 Lb/LF

8,158 - 823.07 = 7,334.93 Lb/LF ≈ 7335 Lb/LF

7,334.93 - 669.19 = 6,665.74 Lb/LF ≈ 6666 Lb/LF

6,665.74/638.6 = 10.44′ (Where 638.6 = P_A + 72 psf)

16.80 - 10.44 = 6.36′



SHEAR DIAGRAM

FIGURE 8-10

ΣM_x = 4.2(566.6)[4.2/3 + 10.44]/2 + 566.6[10.44][10.44/2]
- 8,158(11.64) + 72(14.64)[14.64/2]

= - 42,277 Ft-Lb/LF

S Required = 42,277(12)/25,000 = 20.3 in^3/LF

S Furnished = 20.5 > 20.3 in^3/LF

Check Wales: (W10 X 21)

$M \approx WL^2/10 \doteq 8{,}158(6)^2(12)/10 = 352{,}426$ In-Lb

S Required = M/S = 352,426/22,000 = 16.02 in^3

S Furnished = 21.5 > 16.0 in^3

Check Raker: (12 X 12 @ 6' Spacing)

$L = [(20.5)^2 + (13.8)^2]^{1/2} = 24.71'$

Load per raker = 8,158(6)(24.71/13.8) = 87,645 Lbs

$f_c = P/A = 87{,}645/(12)(12) = 609$ psi

Allowable $F_c = 480{,}000/[(24.71)(12)/12]^2 = 786 > 609$ psi

Check Pads: (6 X 12)

Check soil bearing pressure per NAVFAC for sloping ground condition. See "Ultimate Bearing Capacity of Continuous Footings With Inclined Load", Appendix B.

Compute D/B (ratio of footing depth to footing width):
(D/B) = (4.00)/8.00 = 0.50

From the Figure $N_{\gamma q} \approx 14$

$q_{ult} = C(N_{cq}) + (1/2)(\gamma B)(N_{\gamma q})$

$= 0 + 1/2(110)(8)(14) = 6{,}160$ psf

Safety Factor = 2, $\therefore q_{Allowable} = 6{,}160/2 = 3{,}080 > 2{,}000$ psf

Soil bearing area needed = 87,645/2,000 = 43.82 Ft^2

With 8' width: "L" needed = 43.82/8 = 5.48 < 6.00' spacing

Length for flexure = pad length/2 - flange width/2

= 5.48/2 - 0.84/2 = 2.32'

Moment per foot width = $WL^2/2 = 2{,}000(2.32)^2/2 = 5{,}382$ Ft-Lb/LF

$S = bh^2/6 = 12(6)^2/6 = 72$ in^3

$f_b = M/S = 5{,}382(12)/72 = 897 < 1500$ psi

Length for horizontal shear = 5.48/2 - 0.84/2 - 0.50 = 1.82'

$f_v = 3V/2A = [3(2{,}000)(1.82)]/[2(6)(12)] = 75.83 < 140$ psi

Check Corbel: (HP10 x 42)

Load per foot = 5.48(2,000) = 10,960 Lb/LF

Length of cantilever = 8/2 - 1/2 = 3.5′

$M = WL^2/2 = 10{,}960(3.5)^2(12)/2 = 805{,}560$ In-Lb

$f_b = M/S = 805{,}560/43.4 = 18{,}561 < 22{,}000$ psi

Summary:

Corbel: 12 x 12 Timber could not be used in lieu of the HP10 x 42 due to excessive compressive crushing.

Pads: Pads are shown continuous. No splices may be allowed at critical flexure and shear locations. Continuous pads are needed under raker corbels for a length of 5′ - 6" minimum.

Sheet Piling: A slightly less stiff sheet piling section could have been used . When sheet piling is sufficiently flexible it may be of advantage to use Rowe's Moment Reduction Theory. For stiff sheet piling Rowe's theory will provide no advantage for moment reduction.

Raker: Substitution of a steel member for the 12 x 12 timber would have permitted wider spacing of the rakers if the pad configuration does not control. Placement of the wale at a lower elevation would decrease the axial load on the raker (due to angular change) and might also benefit the design of the pads and corbels. Installation of ribbons on the rakers is suggested to limit lateral deflection.

Note: Corbels and wales should be checked for web stiffeners at point of contact with the rakers. Also check the wall/wale/raker connection for the vertical component of the raker force.

CANTILEVER SHEET PILING - COHESIVE SOIL (ϕ = 0 METHOD) *

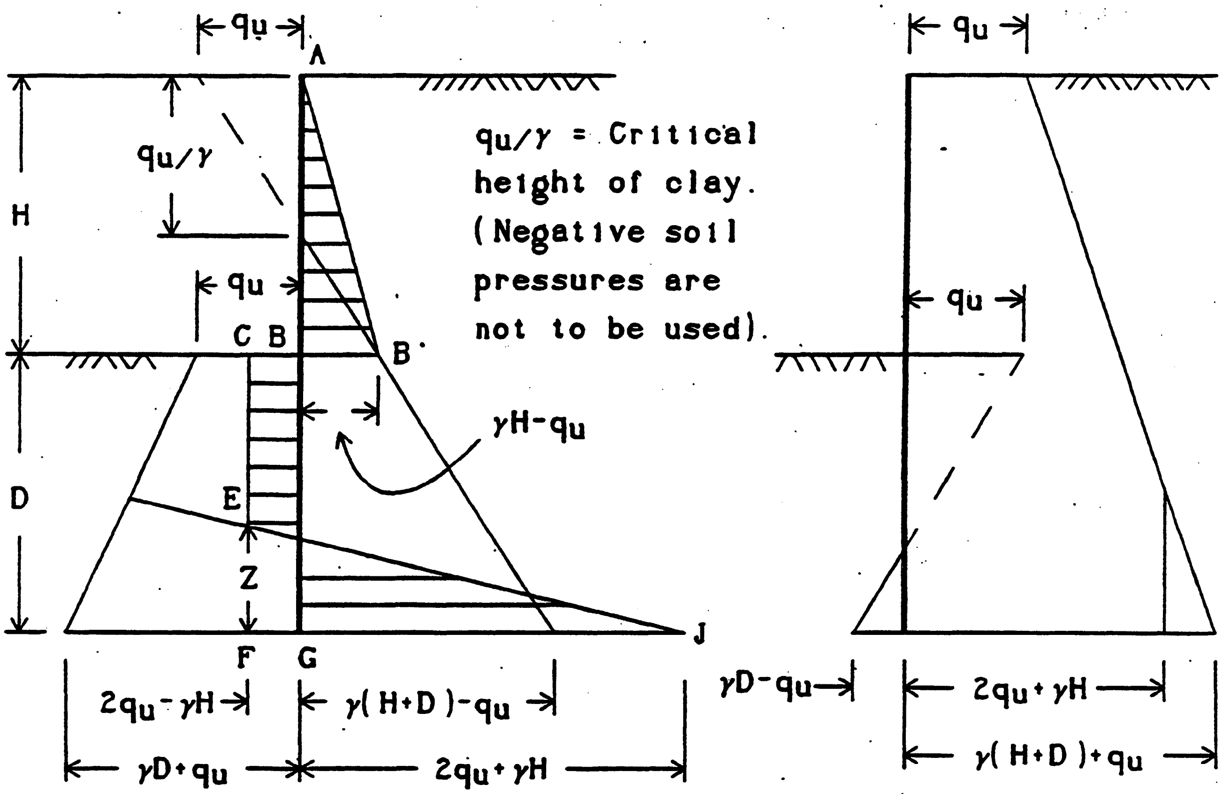


FIGURE 8-11

Use a safety factor by reducing the shear strength of the clay by 50% to 70%, or increase D by 20% to 40%. $H_c = 2q_u/\gamma$ = Critical height of the wall. ($H_c = 4C/\gamma$ since $C = q_u/2$). Theoretically the wall will fail if $H > 4C/\gamma$.

$BB' = \gamma H - q_u \geq 0$ (If not, see note below)

$\Sigma F_H = 0$ = Area <u>ABB'</u> - Area <u>BCFG</u> + Area <u>JEF</u>

$\Sigma M_G = 0$

Solve the two equations simultaneously for D and Z. Determine maximum moment and section modulus required.

When applicable, or when the system will be in use for more than one month, investigate the condition when the clay pressures approach those for a granular soil above the depth of excavation. Hence, assume C approaches 0 and ϕ = 20° to 30° (See the following page).

* Note: If $\phi \neq 0$ or if BB' < 0, see following page.

CANTILEVER SHEET PILING - COHESIVE SOIL (ALTERNATE METHOD)

This approach should be used only when $\phi \neq 0$ or $BB' \leq 0$.

If $\phi > 0$, then $BB' = K_a \gamma H$, where $K_a = \tan^2(45° - \phi/2)$ for level ground.

If $BB' < 0$, then assume cohesion $C = 0$ and use $\phi = 20°$ to $30°$.

BB' then $= \gamma H$ vertically and $K_a \gamma H$ acting horizontally.

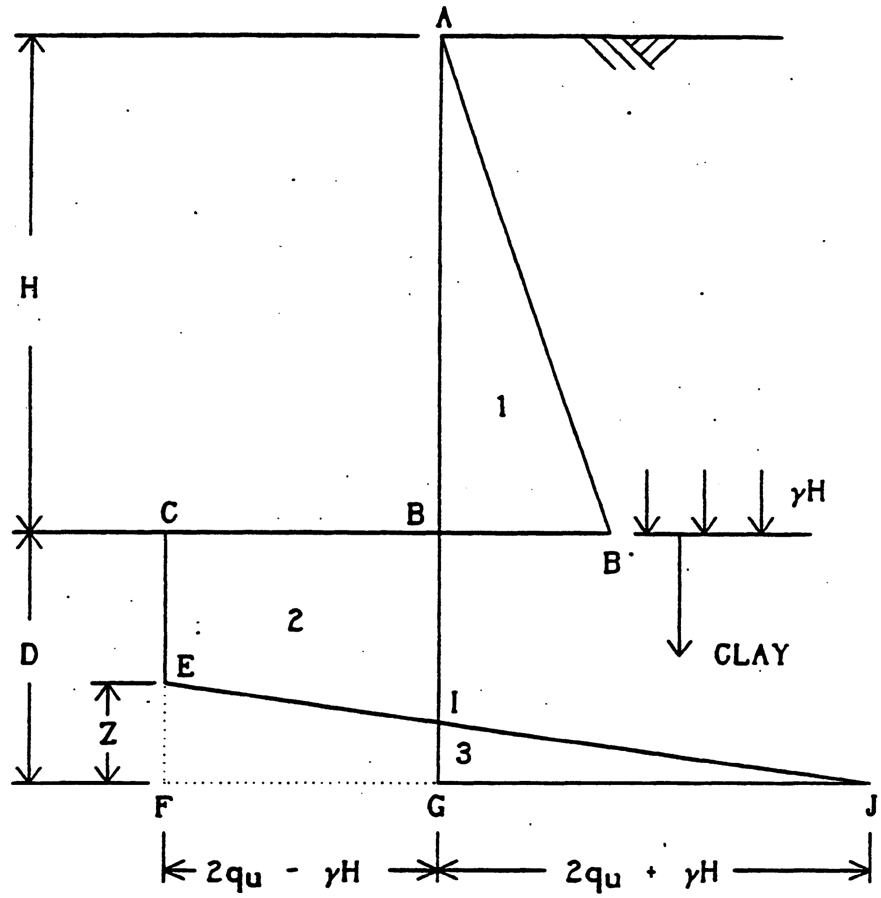
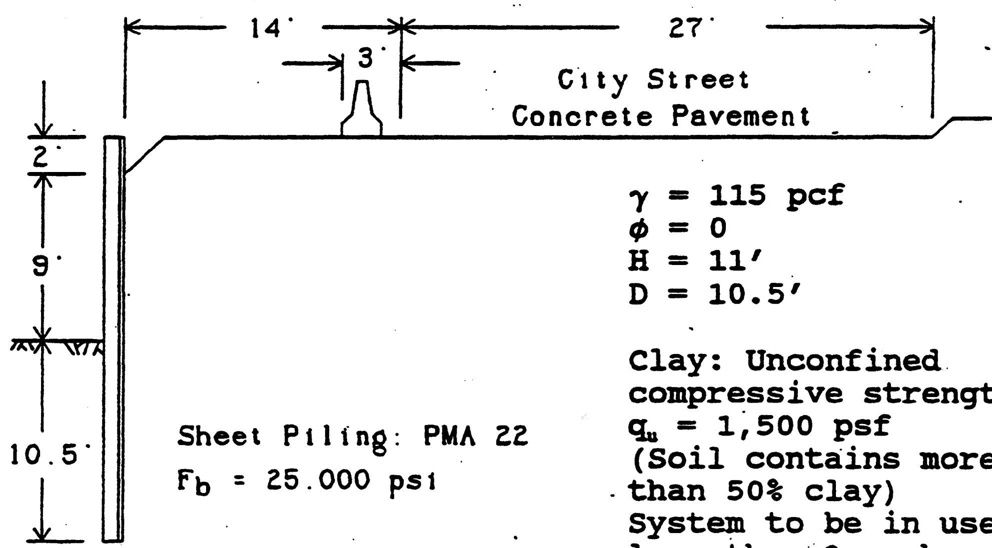


FIGURE 8-12

The procedure from this point is identical to the "$\phi = 0$ method" discussed on the previous page.

SAMPLE PROBLEM 8-3: CANTILEVER SHEET PILE WALL (CLAY)

Given:



γ = 115 pcf
ϕ = 0
H = 11'
D = 10.5'

Clay: Unconfined compressive strength q_u = 1,500 psf (Soil contains more than 50% clay)
System to be in use less than 3 weeks.

FIGURE 8-13

Solution: (Using surcharge values from Chapter 6 tables:)

Depth	K Rail (Q = 200 psf)	Traffic (Q = 300 psf)	Totals
0	0.0	0.0	0
2	6.7	35.2	42
4	11.5	66.7	78
6	13.6	91.4	105
8	13.6	108.7	122
10	12.3	119.1	131
11	11.5	122.1	134

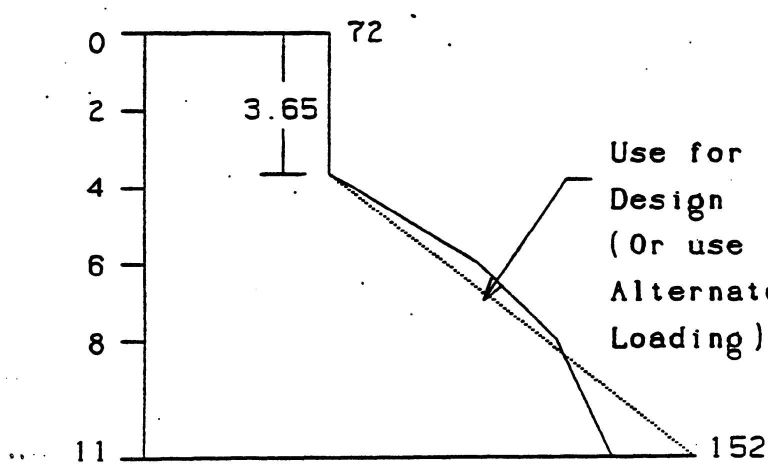


FIGURE 8-14

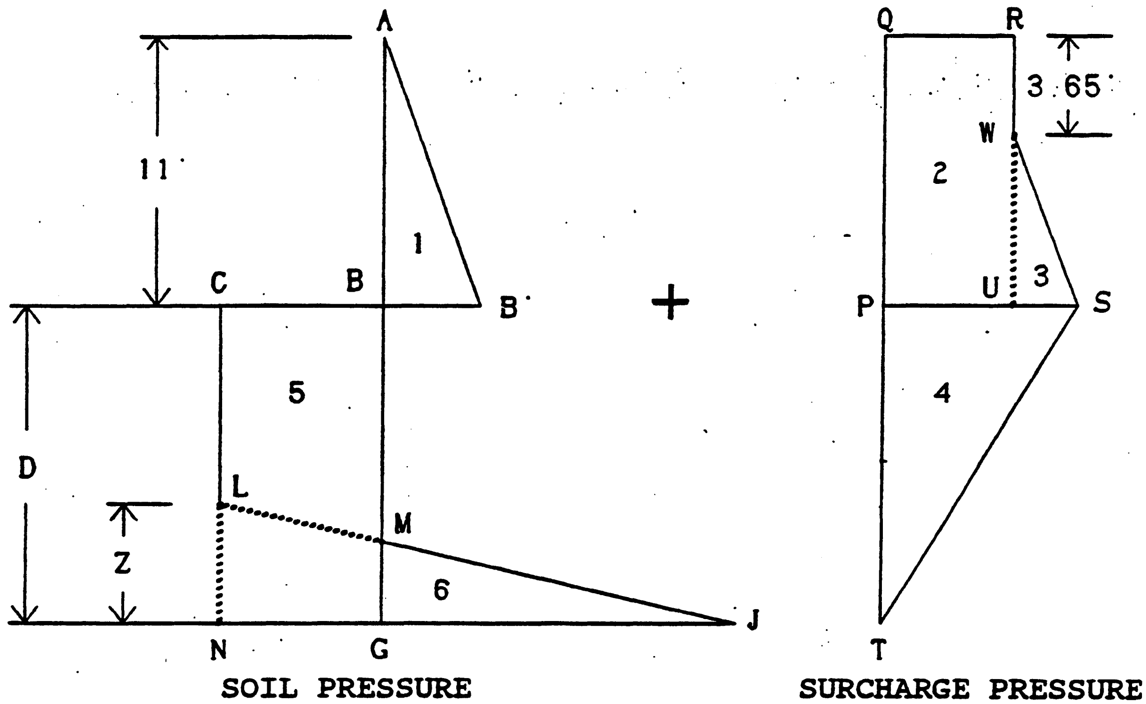


FIGURE 8-15

Use a safety factor of 1.5 for the clay.

$C = q_u/2 = 1500/2 = 750$ psf   For F.S. = 1.5 use C/1.5

$C/1.5 = 750/1.5 = 500$ ($\therefore$ Effective $q_u = 2C = 1000$ psf)

Check critical height of the clay.

$H_c = 2q_u/\gamma = 2(1{,}000)/115 = 17.4' > 11'$

The limiting height of the wall is:

$H' \leq C/[0.31(S.F.)(\gamma_e)]$

For C = 1500/2 = 750 psf, and S.F. = 1.5:

$H' = 750/[0.31(1.5)(115)] = 14'$ ($14' > 11'$ Used $\therefore$ OK)

$BB' = \gamma H - q_u = 115(11) - 1{,}000 = 265$ psf

$GJ = 2q_u + \gamma H = 2(1{,}000) + 115(11) = 3{,}265$ psf

$CB = 2q_u - \gamma H = 2(1{,}000) - 115(11) = 735$ psf

Dissipate surcharge to zero at depth D (Area 4).

Areas:

1	[ABB′]	0.5(11)(265)	=	1,458			
2	[QRPU]	11(72)	=	792			
3	[WUS]	0.5(7.35)(152 −72)	=	294			
4	[PST]	0.5(152)D	=		76D		
5	[CBGN]	(−735D)	=				−735D
6	[JLN]	0.5(735 + 3,265)Z	=			2,000Z	
$\Sigma F_H = 0$				2,544	+ 76D	+ 2,000Z	− 735D

$\therefore Z = (659D - 2{,}544)/2{,}000 = 0.330D - 1.272$

$$\Sigma M_G = 0 = 1{,}458[D + 11/3] + 792[D + 11/2] + 294[D + 7.35/3] + 76D[2D/3] - 735D[D/2] + 2{,}000Z[Z/3]$$

$$0 = 1{,}458D + 5{,}346 + 792D + 4{,}356 + 294D + 720 + 51D^2 - 368D^2 + 667Z^2$$

$\therefore D^2 - 8.03D - 32.88 - 2.10Z^2 = 0$

By simultaneous solution Z = 2.7′, D = 12.1′ > 10.5′

Use D = 12.1′ (D need not be increased since safety factor was applied to clay).

Locate point of zero shear (x = distance below excavation).
$2{,}544 + x[152 + (152)(12.1 - x)/12.1]/2 - 735x = 0$
$x^2 + 117x - 405 = 0 \quad \therefore x = 3.37$

Surcharge at x = 152(12.1 − 3.37)/12.1 = 110 psf

Pressure area between PS and x = 3.37(152 + 110)/2 = 441 psf

$$M_{max} = 1{,}458[3.37 + 11/3] + 792[3.37 + 11/2] + 294[3.37 + 7.35/3] + 110(3.37)[3.37/2] + \{3.37(152 - 110)/2\}[2/3(3.37)] - 735(3.37)[3.37/2]$$
$$= 15{,}605 \text{ Ft-Lb/LF}$$

S required = 15,605(12)/25,000 = 7.49 > 5.4 in^3 furnished

Anticipate sheet piling deflection of 0.005H = 0.005(11) = 0.055′ or ≈ 5/8". Expect settlement behind sheet piling to a distance of 3 times H = 3(11) = 33′. This settlement distance will have an adverse effect on the adjacent concrete pavement. A propped or tieback sheet pile wall should result in less deflection and settlement problems. Ties to deadmen situated beyond the passive failure wedge or to pile anchors located near the K-rail could furnish support similar to props or tiebacks.

Sample Problems 8-2 and 8-3 were reanalyzed using no surcharge below the excavation depth. A comparison of results for computed depth and moment are tabulated below. This tabulation may be of help in Checking computer results.

The following tabulation is for comparative purposes only:

	Use Surcharge Below Excavation Depth	No Surcharge Below Excavation Depth
Sample Problem 8-2		
D_1	6.13′	5.83′
D	8.17′	7.86′
130%(D)	10.6′	10.2′
M	42,277 Ft-Lb/LF	41,137 Ft-Lb/LF
Sample Problem 8-3		
Z	2.7′	2.6′
D	12.1′	10.7′
M	14,746 Ft-Lb/LF	14,825 Ft-Lb/LF

The following tables furnish selected properties for various steel sheet piles.

Minimum grade of steel is A328, for which F_b = 25 ksi.
Most suppliers also furnish high strength steel, such as A572 (grade 50) F_b = 30 ksi.
Bethlehem Steel also manufactures A690 for which F_b = 41 ksi.
For sheet piles manufactored prior to 1940 or those with no identified grade of steel, use F_b = 22 ksi (A36 equivalent)

BETHLEHEM STEEL CORPORATION STEEL SHEET PILING

Type	Width	Weight		S	I**
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4)
PZ22	22.0	40.3	22.0	18.1	154.7
PZ27	18.0	40.5	27.0	30.2	276.3
PZ35	22.6	66.0	35.0	48.5	681.5
PZ40	19.7	65.6	40.0	60.7	805.4
PLZ23	24.0	45.2	22.6	30.2	407.5
PLZ25	24.0	49.6	24.8	32.8	446.5
PSA23	16.0	30.7	23.0	2.4	5.5
PS27.5	19.7	45.1	27.5	2.0	5.3
PS31	19.7	50.9	31.0	2.0	5.3

** Note: The moment of inertia (I) listed on this sheet is per pile.

TABLE 8-1

 Revised 06/95

SHEET PILING

UNITED STATES STEEL SHEET PILING

Type	Width	Weight		S	I**
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4)
PZ38	18.0	57.0	38.0	46.8	421.0
PZ32	21.0	56.0	32.0	38.3	386.0
PZ27	18.0	40.5	27.0	30.2	276.0
PDA27	16.0	36.0	27.0	10.7	53.0
PMA22	19.6	36.0	22.0	5.4	22.4
PSA28	16.0	37.3	28.0	2.5	6.0
PSA23	16.0	30.7	23.0	2.4	5.5
PSX32	16.5	44.0	32.0	2.4	3.7
PS32	15.0	40.0	32.0	1.9	3.6
PS28	15.0	35.0	28.0	1.9	3.5

** Note: The moment of inertia (I) listed on this sheet is per pile.

TABLE 8-1

CASTEEL USA STEEL SHEET PILING

Type	Width (in)	Weight per ft of pile (Lb)	Weight per sq ft of wall (Lb)	S per ft of wall (in^3/LF)	I (in^4/LF)	r (in)
CL42	21.7	15.5	8.6	2.55	4.5	1.33
CL47	21.7	17.4	9.6	2.88	5.1	1.38
CL57 *	21.7	21.1	11.7	3.53	6.2	1.38
CS60 *	27.6	28.2	12.3	6.98	20.6	2.36
CS76 *	27.6	35.6	15.6	8.89	26.3	2.36
CU94 *	23.6	37.9	19.3	10.19	39.7	2.71
CU81 *	19.7	27.2	16.6	11.16	52.7	3.31
CU104	23.6	41.9	21.3	11.39	44.4	2.71
CU99	19.7	33.3	20.3	13.28	62.8	3.31
CU118	19.7	39.7	24.2	15.80	74.5	3.31
CU110 *	22.7	42.5	22.5	21.39	149.0	4.76
CU116 *	22.7	44.9	23.8	22.32	158.2	4.76
CU122	22.7	47.2	25.0	23.25	164.8	4.76
CZ84 *	21.7	31.1	17.2	13.62	53.6	3.27
CZ95	21.7	35.2	19.5	15.53	61.2	3.27
CZ107 *	21.7	39.6	21.9	17.48	68.8	3.27
CZ113	21.7	41.7	23.1	18.40	72.7	3.27
CZ114	24.0	46.8	23.4	31.62	211.6	5.55
CZ128	24.0	52.3	26.2	35.34	236.5	5.55
CZ141	24.0	57.9	28.9	39.06	261.4	5.55
CZ148	24.0	60.7	30.3	40.92	273.9	5.55

* Supplied only by special arrangement with the mill.

TABLE 8-1

LARSSEN STEEL SHEET PILING

Type	Width	Weight		S	I	r
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4/LF)	(in)
SL1	14.4	13.8	11.5	2.83	4.5	1.15
SL2	17.7	21.8	14.7	5.58	14.3	1.81
SL3	17.7	25.5	17.3	10.20	40.3	2.80
SL4	17.7	31.5	21.3	15.80	77.6	3.52
31	17.7	30.2	20.5	8.55	25.3	2.05
I	15.8	26.9	20.5	9.3	27.1	2.13
II	15.8	32.8	25.0	15.8	62.2	2.91
III	15.8	41.7	31.8	25.3	123.0	3.62
IV	15.8	50.3	38.3	37.9	231.0	4.53
V	16.5	67.2	48.7	55.1	372.0	5.12
VI	16.5	82.0	59.4	78.1	673.0	6.22
IIn	15.8	32.8	25.0	20.5	109.0	3.84
IIIn	15.8	41.7	31.8	29.8	170.0	4.27
IIs	19.7	46.8	28.5	29.8	201.0	4.90
IIIs	19.7	52.1	32.4	37.2	278.0	5.41
IVs	19.7	59.1	36.1	46.5	401.0	6.16
Vs	19.7	71.2	43.4	59.5	527.0	6.43

TABLE 8-1

ARBED Esch-Belval STEEL SHEET PILING

Type	Width	Weight		S	I	r
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4/LF)	(in)
BZ 0	19.7	34.9	21.3	9.7	25.7	2.03
BZ 0R	19.7	42.2	25.7	11.4	30.2	2.00
BZ 155	21.7	34.4	19.1	14.0	52.2	3.05
BZ 250	19.7	37.7	23.0	22.3	105.5	3.95
BZ 350	19.7	43.9	26.8	31.1	180.4	4.79
BZ 450	19.7	57.1	34.8	48.4	333.2	5.70
BZ 550	19.7	69.9	42.6	59.5	410.1	5.72
BZ IR	16.5	34.4	25.0	16.0	52.6	2.68
BZ IIN	17.7	36.9	25.0	22.3	96.7	3.63
BZ IIR	17.7	42.3	28.7	25.5	111.4	3.64
BZ IIIN	17.7	46.9	31.7	33.3	170.4	4.28
BZ IIIR	17.7	55.9	37.9	39.3	203.2	4.27
BZ IVNE	19.7	53.4	32.6	38.1	217.6	4.77
BZ IVN	17.7	53.2	36.0	43.9	259.2	4.95
BZ IVR	17.7	65.0	44.0	53.2	318.3	4.96
BZ VN	19.7	79.6	48.5	69.2	476.7	5.78
BZ VR	19.7	91.4	55.7	78.5	547.0	5.78

TABLE 8-1

ARBED STEEL SHEET PILING

Type	Width	Weight		S	I	r
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4/LF)	(in)
AZ 13	26.38	48.38	21.92	24.2	144.3	4.72
AZ 18	24.80	49.99	24.17	33.5	250.4	5.94
AZ 26	24.80	65.72	31.75	48.4	406.5	6.59
AZ 36	24.80	82.11	39.73	67.0	606.3	7.21
BZ 6	19.7	42.8	26.1	11.5	30.4	1.99
BZ 7	21.7	34.3	19.0	14.0	52.5	3.07
BZ 8.6	21.7	39.5	21.9	16.0	60.8	3.08
BZ 12	19.7	37.7	23.0	22.3	106.4	3.97
BZ 16.4	19.7	43.3	26.4	30.5	180.0	4.82
BZ 17	19.7	44.0	26.8	31.1	183.7	4.83
BZ 20	19.7	53.9	32.9	37.6	214.8	4.72
BZ 26	19.7	56.9	34.7	48.2	331.9	5.71
BZ 32	19.7	69.8	42.5	59.4	411.7	5.74
BZ 37	19.7	78.6	47.9	67.9	467.7	5.76
BZ 42	19.7	90.9	55.4	78.1	541.3	5.76

TABLE 8-1

HOESCH STEEL SHEET PILING

Type	Width	Weight		S	I	r
	(in)	per ft of pile (Lb)	per sq ft of wall (Lb)	per ft of wall (in^3/LF)	(in^4/LF)	(in)
95	20.7	33.5	19.5	13.95	52.2	3.03
116	20.7	40.9	23.8	22.32	109.9	3.98
122	20.7	43.1	25.0	17.48	65.4	2.99
134	20.7	47.3	27.4	31.62	186.8	4.81
155	20.7	54.7	31.7	37.20	219.7	4.85
175	20.7	61.8	35.8	48.36	323.7	5.56
215	20.7	75.9	44.0	58.59	392.2	5.52

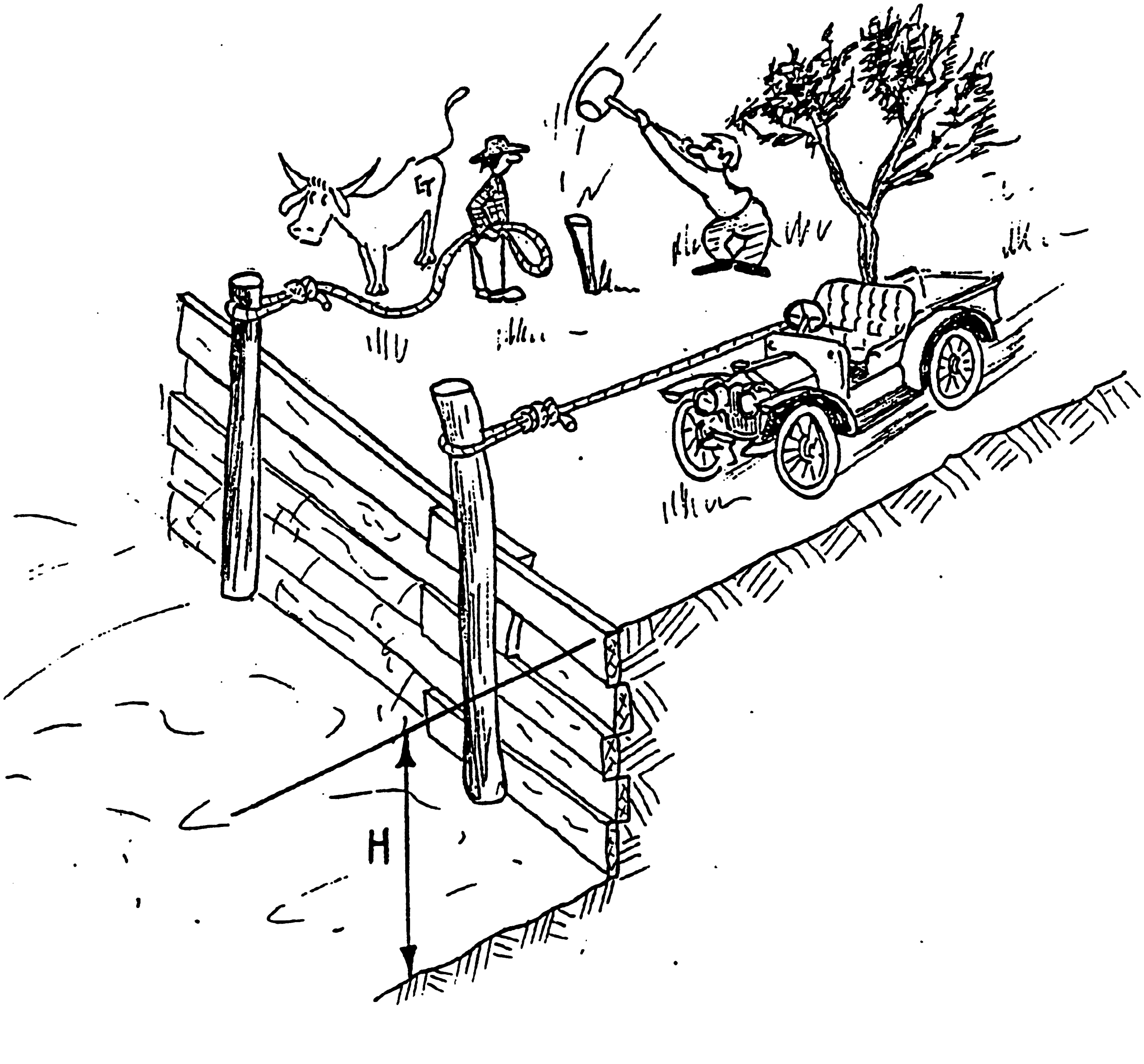
TABLE 8-1

FOSTER STEEL SHEET PILING

Type	Width (in)	Weight per ft of pile (Lb)	Weight per sq ft of wall (Lb)	S per ft of wall (in^3/LF)	I (in^4/LF)	r (in)
FZ-7	24.0	46.8	23.4	21.60		
FZ-9	24.0	52.3	26.2	35.30		
RZ10	21.7	47.7	26.4	30.50	172.1	5.35
RZ11	19.1	47.7	30.0	37.20	220.4	5.35
RZ20	21.7	64.0	35.4	49.50	341.1	5.74
RZ30	21.7	82.0	45.5	71.60	565.3	6.17

TABLE 8-1

TIEBACK SYSTEMS



ANCHORED SHORING SYSTEMS

Anchored shoring systems used for temporary shoring are primarily two types; stressed anchors (typically tiebacks) composed of high strength steel bars or strands grouted into a drilled hole and passive unstressed anchors (typically tie rods with concrete deadmen or anchor piles). It is common to use nongravity retaining walls to retain the soil with anchors (from one or more tiers) providing additional lateral resistance.

Nongravity cantilevered walls may engage discrete vertical elements with structural facing elements for the retention of soil or may be of a type that uses continuous vertical wall elements that also form the structural facing. Typical discrete vertical elements used for temporary shoring are steel piles with facing elements being timber lagging or steel plates. A common material for continuous vertical wall. elements is steel sheet piling.

As used in this manual, nongravity cantilevered walls with discrete vertical elements will be referred to as soldier pile walls' and those walls with continuous elements will be referred to as sheet pile walls'.

Nongravity cantilevered walls derive lateral resistance through embedment of vertical wall elements and support retained soil with facing elements. The discrete vertical elements typically extend deeper into the ground than the facing to provide vertical and lateral support.

The overall stability of anchored shoring systems and the required strength of its members depends on the interaction of a number of factors, such as the relative stiffness of the members, the depth of piling penetration, the stiffness and strength of the soil, the length of tiebacks, or tierods and the amount of anchor movement. Tiedback systems can be considered flexible systems that allow active pressure to develop; however, if sufficient tieback force is applied and the shoring system is sufficiently rigid, the system may approximate a restrained system.

Shoring systems anchored with passive an&hors will not be covered in this chapter. These types of systems will normally experience more movement than would tiedback systems, and therefore would not be suitable for shoring used to protect. adjacent structures or utilities. The design pressure diagrams, structural analysis and general design considerations detailed in this chapter are applicable to tiedback or strutted shoring systems. The design of deadman anchors may be found in Chapter 11, "Special Conditions". Examples of strutted systems or systems supported by rakers are included in Chapter 8, "Sheet Piling", and in Appendix G, "Sample Problems".

Descriptions of single-tier and multi-tier tiedback shoring. systems along with several sample problems are included in this chapter to demonstrate current technology.

SINGLE-TIER TIEDBACK SYSTEM

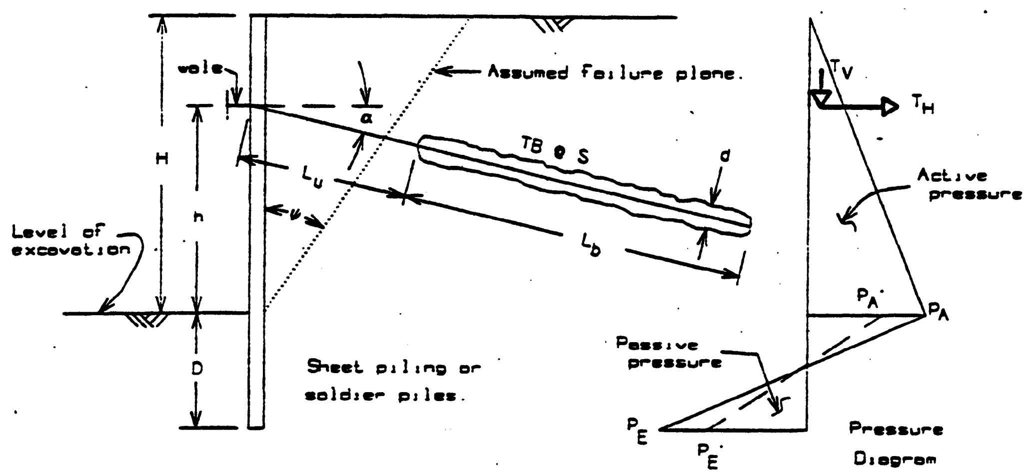


FIGURE 9-1

Nomenclature for Figures 9-1 and 9-2[1]	
H	= Depth of excavation
D	= Embedment depth of piling
h	= Height of tieback above level of excavation - generally about 0.75H
T_H	= The horizontal component of tieback design force
T_V	= The vertical component of the tieback design force
s	= Horizontal spacing of tieback
d	= Diameter of drill hole for tieback
ψ	= Angle between assumed failure plane and vertical
α	= Angle of inclination from horizontal of tieback
L_b	= Bonded length of tieback
L_u	= Unbonded length of tieback
	[1]Expanded definition of terms on following pages

<u>SINGLE-TIER TIEDBACK SYSTEM</u> Refer to Figure 9-1

<u>Soil Pressure Values for Cohesionless Soil:</u>

Sheet Pile Shoring:

$P_A = K_a \gamma H \cos\delta$ = Maximum active pressure at the bottom of the excavation.

$P_E = [K_p \gamma D - K_a(D + H)]\cos\delta$ = Maximum passive resisting pressure.

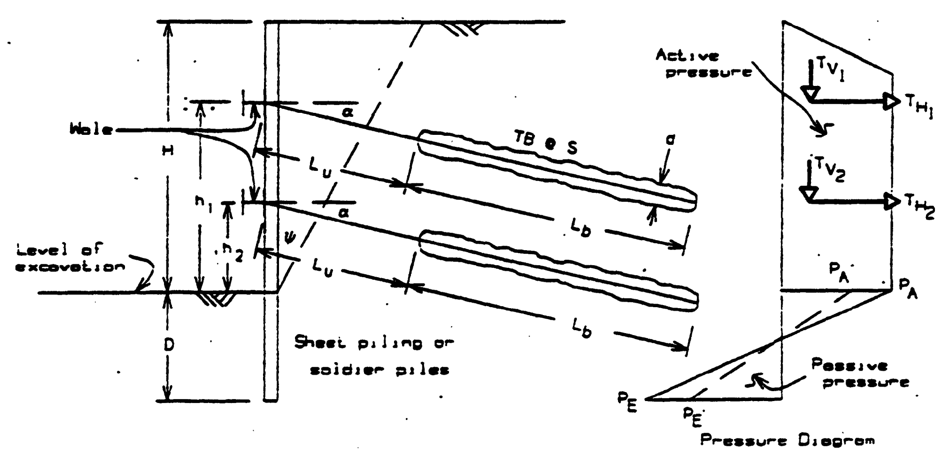
Soldier Pile Shoring:

The active pressure and passive resisting pressure acting on soldier piles below the depth of excavation elevation are reduced by the arching factor (f), defined in Chapter 10, "Soldier Piles," to permit sheet pile type analysis.

$P_A' = fP_A$ and $P_E' = fP_E$

Note: Alternatively a trapezoidal shaped distribution of the design active pressure acting over the height "H" (similar to that used for multi-tier tiedback systems) may be used in lieu of the triangular shaped diagram shown. This method produces a larger tieback force and less embedment depth. The most conservative practice is to check tiebacks using the trapezoidal distribution and embedment depth using the triangular distribution of pressure.

<u>MULTI-TIER TIEDBACK SYSTEM</u>



h_1, h_2 = Vertical location of tieback

FIGURE 9-2

MULTI-TIER TIEDBACK SYSTEM Refer to Figure 9-2

Soil Pressure Values for Cohesionless Soil

Sheet Pile Shoring:

$$P_A = 0.65K_A\gamma H\cos\delta$$

$$P_E = (K_P - K_A)\gamma D\cos\delta - P_A$$

Soldier Pile Shoring:

$$P_A' = fP_A$$

$$P_E' = fP_E$$

General Nomenclature

The embedded portion of the piling below level of excavation. The embedment depth and the horizontal component of the tieback design force required are determined by analyzing the active, passive, and surcharge pressures acting on the piling. A factor of safety is achieved by increasing the calculated embedment depth an additional 20% to 40%. The higher percentage should be used when soil properties are derived from log of test borings or other soil information and not determined from laboratory or in-situ tests used specifically to determine soil strength.

ψ Angle is $\approx 45° - \phi/2$ (for level surface). Values of ψ commonly vary between 20° to 35° depending upon the type of soil. For design of temporary shoring systems, it is normally acceptable to consider the failure plane to start at the elevation of the bottom of the excavation and extend upward at an angle ψ from vertical. For sloping or irregular surfaces, a wedge failure or similar type analysis may be necessary to predict the location of the failure plane (see example in Appendix H, "Memos").

α Angle of inclination of tieback from horizontal. Normally 10° - 15° is used for the angle α to facilitate the placement of grout or concrete. Angles up to 45° may be used to reduce the tieback length, reach stronger soil layers, or to avoid obstacles.

L_b Bonded length of the tieback, which is also referred to as the anchor length of the tieback. The required bonded length depends on the soil or rock properties, the anchor type, and the required anchor capacity.

L_u Unbonded length of tieback. Unbonded length is normally specified to start at some minimum distance past the failure plane to ensure that no portion of the bonded length falls within the failure wedge. Accurate determination of this length depends on how well-known the soil properties are and how accurately the location of the failure plane can be predicted. To ensure that the bonded length falls beyond the failure-plane it is common practice to extend the unbonded length about 5 feet beyond the assumed failure plane. The minimum recommended unbonded length is 15 feet.

CONSTRUCTION SEOUENCE

The construction sequence for an anchored sheet-pile or soldier pile system must be considered when making an engineering analysis. Different loads are imposed on the system before and after the completion of a level of tieback anchors. An analysis should be included for each stage of construction and an analysis may be needed for each stage of anchor removal during backfilling operations.

TIEDBACK ANCHOR SYSTEMS

There are many variations or configurations of tieback anchor systems. The tension element of a tieback may be either prestressing strands or bars using either single or multiple elements. Tiebacks may be anchored against wales, piles, or anchorblocks which are placed directly on the soil. The example problems in this chapter illustrate the use of tiebacks with several different types of shoring systems.

Figure. 9-3 illustrates a typical temporary tieback anchor. In this diagram, a bar tendon system is shown; strand systems are similar.

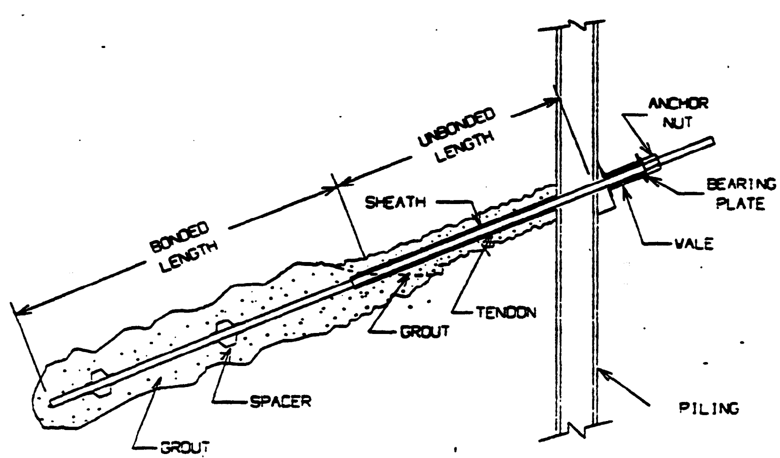


FIGURE 9-3

The more common components,. criteria, and materials used in conjunction with tiedback shoring systems are listed below;

Piling, Sheet piling and soldierpiles. See Chapter 12 "Construction," for common materials and allowable stresses.

Wale - These components transfer the resultant of the earth pressure from the piling to the tieback anchor. A design overstress of 33% is permitted for wales when proof testing the tieback anchor. Anchors for temporary work are often anchored directly against the soldier piling through holes or slots made in the flanges, eliminating the need for wales. Bearing stiffeners and flange cover plates are generally added to the pile section to compensate for the loss of section. A structural analysis of this cut section should always be required.

Tendon - Tieback-tendons are generally the same high strength bars or strands used in prestressing structural concrete.

The anchorage of the tieback tendons at the shoring members consists of bearing plates and anchor nuts for bar tendons and bearing plates, anchor head and strand wedges for strand tendons. The details of the anchorage must accommodate the inclination of the tieback relative to the face of the shoring members. Items that may be used to accomplish this are shims or wedge plates placed between the bearing plate and soldier pile or between the wale and sheet piling or soldier piles. Also for bar tendons spherical anchor nuts with special bearing washers plus wedge washers if needed or specially machined anchor plates may be used.

The tendon should be centered within the drilled hole within its bonded length. This is accomplished by the use of centralizers (spacers) adequately spaced to prevent the tendon from contacting the sides of the drilled hole or by installation with the use of a hollow stem auger.

Stress - Allowable tensile stress values are-based on a percentage of the minimum tensile strength (F_{pu}) of the tendons as indicated below: l

Bars: F_{pu} = 150 to 160 ksi
Strand: F_{pu} = 270 ksi

(Check manufacturers data for actual ultimate strength)

Allowable tensile stresses:

At design load $f_t \leq 0.6\ F_{pu}$
At proof load $f_t \leq 0.8\ F_{pu}$

(Both conditions must be checked)

Grout - A flowable portland cement mixture of grout or concrete which encapsulates the tendon and fills the drilled hole within the bonded length. Generally a neat cement grout is used in drilled holes of diameters up to 8 inches. A sand-cement mixture is used for hole diameters greater than 8 inches. An aggregate concrete mix is comnonly used in very large holes. Type I or II cement is commonly recommended for tiebacks. Type III cement may be used when high early strength is desired. Grout, with very few exceptions, should always be injected at the bottom of the drilled hole. This method ensures complete grouting and will displace any water that has accumulated in the hole.

<u>Tieback anchor</u>

There are several different types of tieback anchors. Their capacity depends on a number of interrelated factors:

Location - amount of overburden above the tieback
Drilling method and drilled hole configuration
Strength and type of the soil
Relative density of the soil
Grouting method
Tendon type, size, and shape

Typical shapes of drilled holes for tieback anchors are depicted in Figure 9-4.

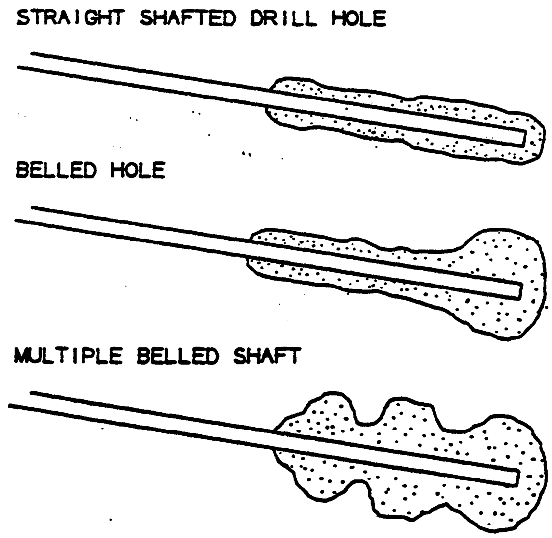


FIGURE 9-4

This is the simplest type and the one encountered most often.

In this case the resistance is a combination of perimeter bond and bearing against the soil.

Similar to above, this type of anchor is also referred to as under-reamed. It is used in stiff cohesive soil. The soil must be stiff enough to prevent collapse of the under-reams or drill hole in the anchor length.

The presence of water either introduced during drilling or existing ground water can cause significant reduction in anchor capacity when using a rotary drilling method in some cohesive soils (generally the softer clays).

High pressure grouting of 150 psi or greater in granular soils can result in significantly greater tieback capacity then by tremie or low pressure grouting methods. High pressure grouting is seldom used for temporary tieback systems.

Regrouting of tieback anchors has been used successfully to increase the capacity of an anchor. This method involves the placing of high pressure grout in a previously formed anchor. Regrouting breaks up the previously placed anchor grout and disperses new grout into the anchor zone; compressing the soil and forming an enlarged bulb of grout thereby increasing the anchor capacity. Regrouting is done through a separate grout tube installed with the anchor tendon. The separate grout tube will generally have sealed ports uniformly spaced along its length which open under pressure allowing the grout to exit into the previously formed anchor.

Due to the many factors involved, the determination of anchor capacity can vary quite widely. Proof tests or performance tests of the tiebacks are needed to confirm the anchor capacity. A Federal publication, the FHWA/RD-82/047 report on tiebacks, provides considerable information for estimating tieback capacities for the various types of tieback anchors. Also see "Supplemental Tieback Information" in Appendix E.

Bond capacity is the resistance to pull out of the tieback which is developed by the interaction of the anchor grout (or concrete) surface with the soil along the bonded length.

Determining or estimating the bond (resisting) capacity is a prime element in the design of a tieback anchor.

Included with some shoring designs there may be a Soils Laboratory report which will contain recommended value for the bond capacity to be used for tieback anchor design. The appropriateness of the value of the bond capacity will only be proven during tieback testing.

For most of the temporary shoring work normally encountered, the tieback anchors will be straight shafted with low pressure grout placement. For these conditions the following criteria can generally be used for estimating the tieback anchor capacity.

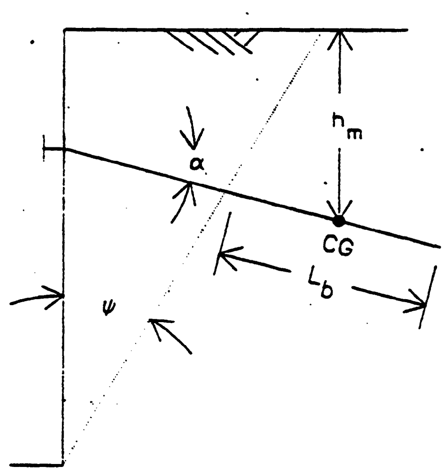


FIGURE 9-5

The FHWA formula for bond is defined as follows:

$$P_{ult} = \pi d L_b \gamma h_m (\tan\phi)$$

Where:

- d = Diameter of drill hole
- L_b = Bonded length of the tieback
- γ = Unit weight of soil
- ϕ = Angle of internal friction of the soil
- CG = Center of L_b = $(L_b/2)$
- h_m = Vertical distance from the ground line to the center of L_b
- ψ = Angle between assumed failure plane and vertical

<u>Forces On The Vertical Members</u>

Tiebacks are generally inclined, therefore the vertical component of the tieback force must be resisted by the vertical member through skin friction on the embedded length of the piling in contact with the soil and by end bearing. Problems with tiedback walls have occurred because of excessive downward wall movement. The pile capacity should always be checked to ensure that it can resist the vertical component of the tieback force. The sheet pile sample problem demonstrates one method to account for the vertical load on the piling.

Ultimate values (without safety factors) for friction and end bearing of piling follow:

For Driven Piling:

Skin Friction

= N/50 tsf for concrete piles

= N/lOO tsf for WF sections

(based on a rectangular perimeter equal to two times the width of the flange added to two times the depth of the section).

End Bearing:

Cohesionless Soil: = 4N tsf

Cohesive Soil:. = $9S_u$ or = 4.5_{qu}

(based on a rectangular perimeter equal to two times the width of the flange added to two times 'the depth of the section).

Special Note: For sheet piling use N/100 for skin friction for depth D on both faces, but do not use end bearing.

For Drilled Piling:

Skin Friction = N/l00 tsf

End Bearing

Cohesionless Soil:= 2N tsf

Cohesive Soil: = $9s_u$ or = 4.5_{qu} (based on the, gross area).

Where N = SPT (Standard Penetration Test) value

Overall (global) System Stability

To ensure overall stability of an an anchored system slope stability analysis may be required in addition. to the general (local) system analysis except when the horizontal component of the anchor is greater than total height of the vertical member. Figure 9-6 depicts the foregoing.

$$a/(H + D) > 1.0$$

Where:

a = The horizontal component of the tieback anchor length

H + D = The vertical member's total length.

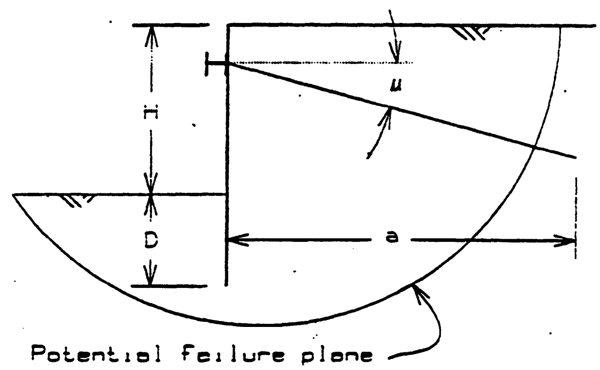


FIGURE 9-6

TESTING TIEBACK ANCHORS

The Contractor is responsible for providing a reasonable test method for verifying the capacity of the tieback anchors after installation. Anchors are tested to assure that they can sustain the design load over time without excessive movement. The need to test anchors is more important when the system will support, or be adjacent to existing structures, and when the system will be in place for an extended period of time.

The number of tiebacks tested; the duration of the test, and the allowable movement, or load loss, specified in the contractor's test methods should take into account the degree of risk to the adjacent surroundings. High risk situations would be cases where settlement or other damage would be experienced by adjacent facilities. See Table 9-1 for a list of minimum recommended criteria for testing temporary tieback anchors.

TIEBACK PROOF TEST CRITERIA

Test Load	Load Hold Duration	Percentage of tiebacks to be load tested
Cohesionless Soils		
Normal Risk		
1.2 to 1.3 Design Load	10 Minutes	10% for each soil type encountered
High Risk		
1.3 Design Load	10 Minutes	20% to 100%
Cohesive Soils		
Normal Risk		
1.2 to 1.3 Design Load	30 Minutes	10%
High Risk		
1.3 Design Load	60 minutes	30% to 100%*

*Use 100% when in soft clay or when ground water is encountered. Use load hold of 60 minutes for 10% and load hold of 10 minutes for remaining 90% of tiebacks.

TABLE 9-1

Generally the shoring plans should include tieback load testing criteria which should minimally consist of proof load test values; frequency of testing (number of anchors to be tested), test load duration, and allowable movement or loss of load permissible during the testing time frame and the anticipated life of the shoring system. The shoring plans should also include the measures that are to be taken when, or if, test anchors fail to meet the specified criteria.

Pressure gages or load cells used for determining test loads should have been recently calibrated by a certified lab, they should be clean and not abused, and they should be in good working order. The calibration dates should be determined and recorded.

Tiebacks which do not satisfy the testing criteria may still have some value. Often an auxiliary tieback may make up for the reduced value of adjacent tiebacks; or additional reduced value tiebacks may be installed to supplement the initial low value tiebacks.

Proof Testing

Proof testing of tiebacks anchors is normally accomplished by applying a sustained proof load to a tieback anchor and measuring anchor movement over a specified period of time. Proof testing may begin after the grout has achieved the desired strength. A specified number of the tieback anchors will be proof tested by the method specified on the Contractor's approved plans (see Table 9-l).

Generally, the unbonded length of a tieback is left ungrouted prior to and during testing (see Figure 9-7). This ensures that only the bonded length is carrying the proof load during testing. It is not desirable to have loads transferred to the soil through grout (or concrete) in the unbonded region since this length is considered to be within the zone of the failure wedge.

As an alternative, for small diameter drilled holes (6 inches or less) a plastic sheathing may be used over the unbonded length of the tendon to separate the tendon from the grout (see Figure 9-3). The sheathing permits the tendon to be grouted full length before proof testing. A void must be left between the top of the grout and the soldier pile to allow for movement of the grout column during testing.

Research has shown that small diameter tiebacks develop most of their capacity in the bonded length despite the additional grout in the unbonded length zone. This phenomenon is not true for larger diameter tieback anchors.

Generally the Contractor will specify an alignment load of 5 to 10% of the design load which is initially applied to the tendon to secure the jack against the anchor head and stabilize the setup. The load is then increased until the proof load is achieved. Generally a maximum amount of time is specified to reach proof load. Once the proof load is attained, the load hold period begins. Movement of the tieback anchor is normally measured by using a dial indicator gage mounted on a tripod independent of the tieback and shoring and positioned in a manner similar to that shown in Figure 9-7.

The tip of the dial indicator gage is positioned against a flat surface perpendicular to the centerline of the tendon (This can be a plate secured to the tendon). The piston of the jack may be used in lieu of a plate if the jack is not going to have to be cycled during the test. As long as the dial indicator gage is mounted independently of the shoring system, only movement of the anchor due to the proof load will be measured. Continuous jacking to maintain the specified proof load during the load hold period is essential to offset losses resulting from anchor creep or movement of the shoring into the supporting soil.

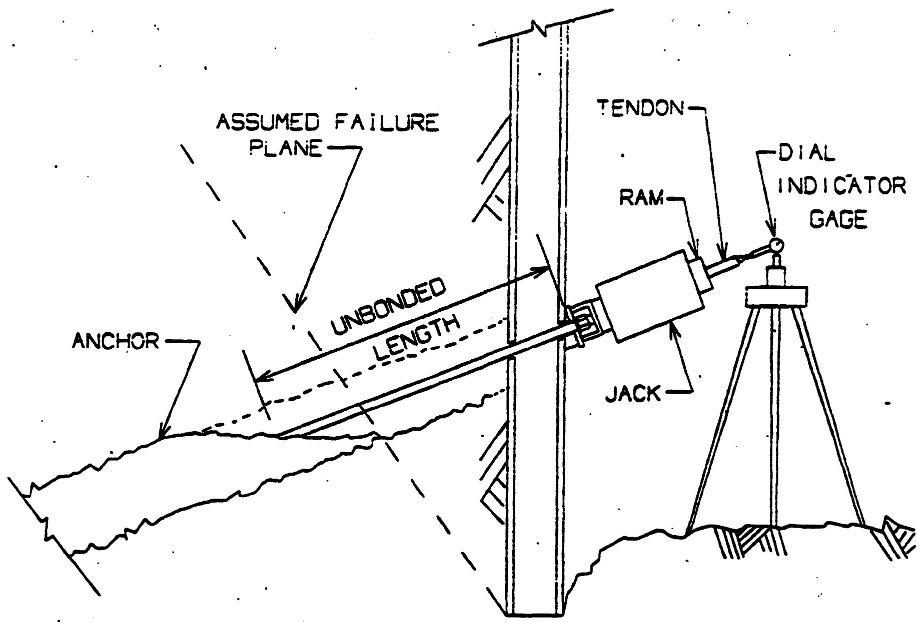


FIGURE 9-7

Measurements from the dial indicator gage are taken periodically during the load hold period. The total movement measured during the load hold period of time is compared to the allowable value indicated on the approved shoring plans to determine the acceptability of the anchor.

It is important that the proof load be reached quickly. When excessive time is taken to reach the proof load, or the proof load is held for an excessive amount of time before beginning. the measurement of creep movement, the creep rate indicated will not be representative. For the proof test to be accurate, the starting time must begin when the proof load is first reached.

As an alternative to measuring movement with a dial indicator gage, the contractor may propose a "lift-off test". A "lift-off test" compares the force on the tieback at seating to the force required to lift the anchor head off of the bearing plate. The comparison should be made over a specified period of time. The lost force can be converted into creep movement to provide an estimate of the amount of creep over-the life of the shoring system.

Use of the "lift-off test" may not accurately predict overall anchor movement. During the time period between lock-off and lift-off, the tieback may creep and the wall may move into the soil. These two components cannot be separated. If the test is done accurately, results are likely to be a conservative measure of anchor movement. The Office of Structure Construction recommends the use of a dial indicator gage to monitor creep rather than lift-off tests.

Evaluation of Creep Movement

Long-term tieback creep can be estimated from measurements taken during initial short term proof testing: In effect, measurements made at the time of proof testing can be extrapolated to determine anticipated total creep over the period the shoring system is in use if it is assumed that the anchor creep is roughly modeled by a curve described by the "log" of time.

The general formula listed below for the determination of the anticipated long term creep is only an estimate of the potential anchor creep and should be used in conjunction with periodic monitoring of the wall movement. This formula will not accurately predict anchor creep for soft cohesive soils.

Based on the assumed creep behavior, the following formula can be utilized to evaluate the long-term effects of creep:

General formula:

$$\Delta_{2\text{-}3} = C[\log_{10}(T_3/T_2)]$$

Where:

$C = \Delta_{1\text{-}2}/[\log_{10}(T_2/T_1)]$

Δ = Creep movement (inches) specified on the plans for times T_1, T_2, or T_3 (or measured in the field)

T_1 = Time of first movement measurement during load hold period (usually 1 minute after proof load is applied)

T_2 = Time of last movement measurement during load hold period

T_3 = Time the shoring system will be in use

If using a 'lift off test' to estimate the creep movement, the following approximation needs to be made for substitution into the above equation:

$$\Delta_{1\text{-}2} \approx (P_1 - P_2)L_u/AE$$

Where:

P_1 = Force at seating $\qquad$ P_2 = Force at lift off

L = L_u + 0 to 5 feet of the bonded length necessary to develop the tendon

A = Area of strand or bar in anchor

E = Modulus of elasticity of the strand or bar in anchor

Sample problem 9-1 demonstrates the calculation of long term creep.

Wall Movement and Settlement

As a rule of thumb, the settlement of the soil behind a tiedback wall, where the tiebacks are locked-off at a high percentage of the design force, can be approximated as equal to the movement at the top of the wall caused by anchor creep and deflection of the piling; Reference is made to the Section titled "Settlement and Deflection" near the end of Chapter 5.

If a shoring system is to be in close proximity to an existing structure where settlement might be-detrimental, significant deflection and creep of the shoring system would not be acceptable: If a shoring system will not affect permanent structures; or when the shoring might support something like a haul road, reasonable lateral movement and settlement can be tolerated.

Performance Testing

Performance testing is similar to, but more extensive, than proof testing. Performance testing is used to establish the movement behavior for a tieback anchor at a particular site. Performance testing is not normally specified for temporary shoring, but it can be utilized to identify the causes of anchor movement. Performance testing consists of incremental loading and unloading of a tieback anchor in conjunction with measuring movement.

Lock-Off Force

The lock-off force is thepercentage of the required design force that the anchor wedges or anchor nut is seated at after seating losses. A value of $O.8T_{DESIGN}$ is typically recommended as the lock-off force but lower or higher values are used to achieve specific design needs.

One method for obtaining the proper lock-off force for strand systems is to insert a shim plate under the anchor head equal to the elastic elongation of the tendon produced by a force equal to the proof load minus the lock-off load. A correction for seating of the wedges in the anchor head is often subtracted from the shim plate thickness. To determine the thickness of the shim plate you may use the following equation:

$$t_{shim} = \frac{(P_{Proof} - P_{lockoff})L}{AE} - \Delta L$$

where:

t_{shim} = thickness of shim

P_{Proof} = Proof load

$P_{lockoff}$ = Lock-off load

A = Area of tendon steel (bar or strands)

E = Modulus of Elasticity of strand or bar

ΔL = seating loss

$L \approx$ Elastic length of tendon (usually the unbonded length + 3 to 5 feet of the bonded length necessary to develop the tendon)

Seating loss can vary between 3/8" to 5/8" for strand systems. The seating loss should be determined by the designer of the system and verified during *installation. Often times, wedges are mechanically seated minimizing seating loss resulting in the use of a lesser value for the seating loss. For thread bar systems, seating loss is much less than that for strand systems and can vary between 0" to l/16".

After seating the wedges in the anchor head at the proof load, the tendon is loaded, the shim is removed and the whole anchor head assembly is seated against the bearing plate.

CORROSION PROTECTION

The contractor% submittal must address potential corrosion of the tendon after it has been stressed. For very short-term installations in non-corrosive sites corrosion protection may not be necessary. The exposed steel may not be affected by a small amount of corrosion that occurs during its life.

For longer term installations grouting of the bonded and unbonded length-is generally adequate-to minimize corrosion in most non-corrosive sites. Encapsulating *or* coating any ungrouted portions (anchor head, bearing plate, wedges, strand, etc.) of the tieback system may be necessary to guard against corrosion.

For long-term installations or installations in corrosive sites, more elaborate corrosion protection schemes may be necessary (Grease is often used as a corrosion inhibitor). Figure 9-8 depicts tendonsencapsulated in pregreased and pregrouted plastic sheaths generally used for permanent installations.

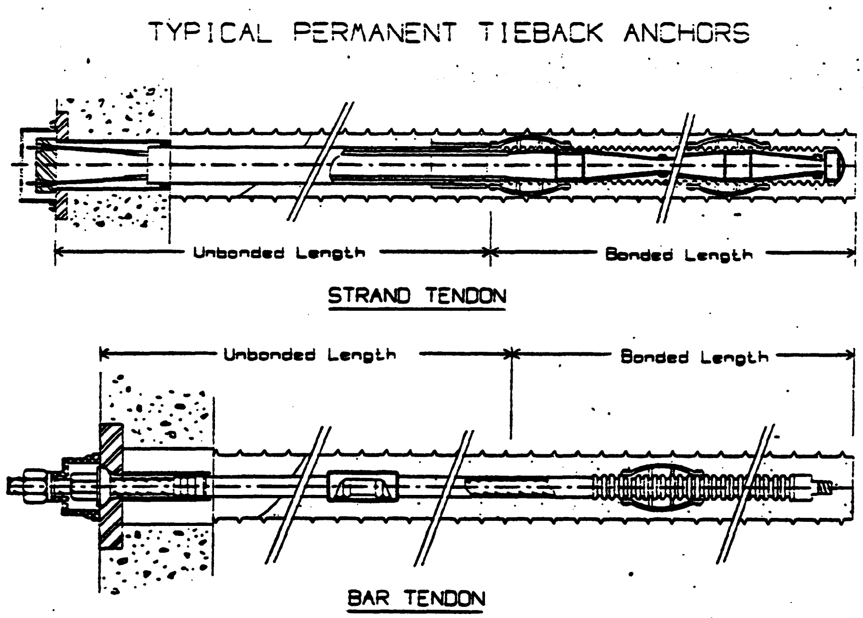


FIGURE 9-8

STEPS FOR CHECKING TIEDBACK SHORING SUBMITTAL

1. Review plan submittal for completeness.
2. Determine K_a and K_P
3. Develop pressure diagrams.
4. Determine forces.
5. Determine the moments around the top of the pile (or some other convenient location).
6. Solve for depth (D), for both lateral and vertical loads, and tieback force (T_H).
7. Check pile section.
8. Check anchor capacity.
9. Check miscellaneous details.
10. Check adequacy of tieback test procedure.
11. Review corrosion proposal,
12. General: Consider effects of wall deflection, and subsequent soil settlement on any surface feature behind the shoring wall.

SAMPLE PROBLEM 9-1 TIEBACK TESTING

Determine the long-term effects of creep.

Measurement and time method:

Given:

The shoring plans indicate that a proof had shall be applied in 2 minutes or less then the load shall be held for ten minutes. The test begins immediately upon reaching the proof load value. Measurements of movement. are to be taken at 1, 4, 6, 8 and 10 minutes. The proof load is to be 133% of the design load. The maximum permissible movement between 1 and 10 minutes of time will not exceed 0.1 inches. All tiebacks are to be tested. The system is anticipated to be in place for 1 year.

Solution:

$A = 0.1$ inches
$T_1 = 1$ minute
$T_2 = 10$ minutes
$T_3 = (1\ Y)(365\ D/Y)(24\ H/D)(60\ M/H) = 525{,}600$ minutes

$$C = \Delta_{1-2}/[\log_{10}(T_2/T_1)] = 0.1/[\log_{10}(10/1)] = 0.1$$

$$\text{Long term } \Delta_{2-3} = (C)\log_{10}(T_3/T_2) = (0.1)\log_{10}(525{,}600/10)$$
$$= 0.47 \text{ inches} = 1/2 \text{ inch}$$

The proof load, and duration of test are reasonable and exceed the minimums shown in Table 9-l. Applying the proof load in. a short period of time and beginning the test immediately upon reaching that load ensure the test results will be meaningful and can be compared to the calculated long term creep movement for the anchor.

If the shoring system was in close proximity to an existing structure that could not tolerate a l/2 inch of settlement the design would not be acceptable. If the shoring would not affect permanent structures or when the shoring might support something like a haul road, the anticipated movement would be tolerable.

Lift off load method:

Given:

Lift off test will be performed 24 hours after wedges are seated (1 minute). The force at seating the wedges will be 83,000 pounds and the lift off force will be no less than 67,900 pounds.

$L \approx 20$ ft which is the unbonded length of 15′ + 5′
$A = 0.647$ in^2
$E = 28x10^6$ psi
T_2 = 1 minute, this is the time the wedges are seated.

$\Delta_{1-2} \approx ((P_1 - P_2)L)/AE$
$\approx ((83{,}000 - 67{,}900)(20)(12))/((0.647)(28x10^6))$
≈ 0.2 in

$C \approx 0.2/[\log_{10}(1440/1)]$
≈ 0.06

Long term $\Delta_{2-3} \approx (C)\log_{10}(T_3/T_2) = (0.06)\log_{10}(525{,}600/1)$
≈ 0.34 inches $\approx 5/16$ inch

SAMPLE PROBLEM 9-2 SINGLE-TIER TIEBACK SHORING WALL

This example problem illustrates the analysis for a single tier tieback sheet pile wall next to a haul road and demonstrates the 'following principles:

- The use of Teng's "Free Earth Support Method" of sheet pile analysis with Rowe's "Moment Reduction Theory" to determine the required depth of embedment (D), the required sheet pile section modulus ($S_{REQUIRED}$), and the design tieback force(T).
- Low pressure grouted anchor tieback analysis.
- Review of proof loading and lock-off loading.

The Contractor% shoring submittal outlined below using PSX32 steel sheet pile *is* to be reviewed for adequacy.

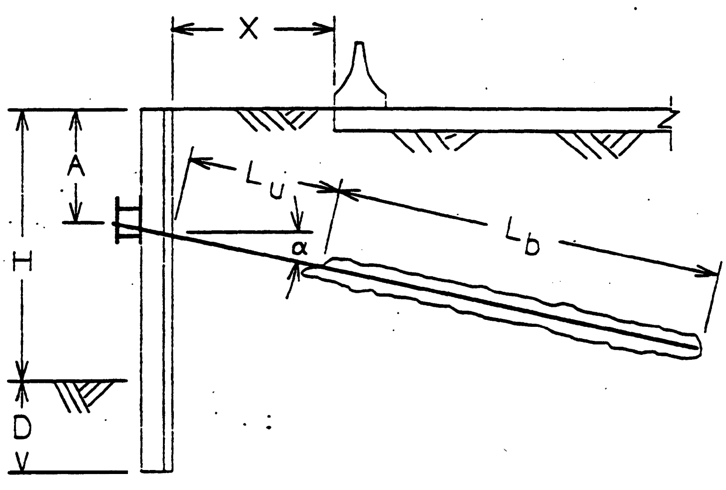


FIGURE 9-9

Soil Properties:

γ = 115 pcf

ϕ = 35°

Dimensions:

H = 15 feet   X = 10 feet   A = 3'-6"
L_U= 15 feet L_b= 25 feet   D = 6'-6"
Tieback angle α = 15°   Tieback spacing = 8'-O"

Anchor Details:

5/8" Dywidag bars at 8' 0" center spacing centered in 6" diameter (d) drilled holes which are to be grouted with low pressure grout.

T_{DESIGN} = 25 Kips
T_{proof} = $(1.3)T_{DESIGN}$

Proof Testing of Tiebacks: (Notes on the shoring plans)

Alternate anchors will be proof tested to T_{PROOF} after the anchor grout has obtained adequate strength.

The exposed end of the anchor rod shall not show movement of more that 2 inches while jacking up to the proof load value.

The proof load (T_{PROOF}) shall be attained and held for 15 minutes. Anchor movement shall not exceed 0.1 inches between 1 and 15 minute, Readings shall be taken at 1, 5, 10 and 15 minutes. The system will be in place approximately 6 months.

Anchors failing the -test criteria shall be replaced.

Analysis:

Top failure wedge width = 15' tan(45° - ϕ/2) = 7.8' < 10'+ 2'.

Since light haul road traffic is to be beyond the active failure wedge limits, the use of minimal friction on the sheet piling for the active condition may be permitted.

For simplified analysis, use the alternate loading of 100 psf for traffic surcharge.

Pressure Diagram:

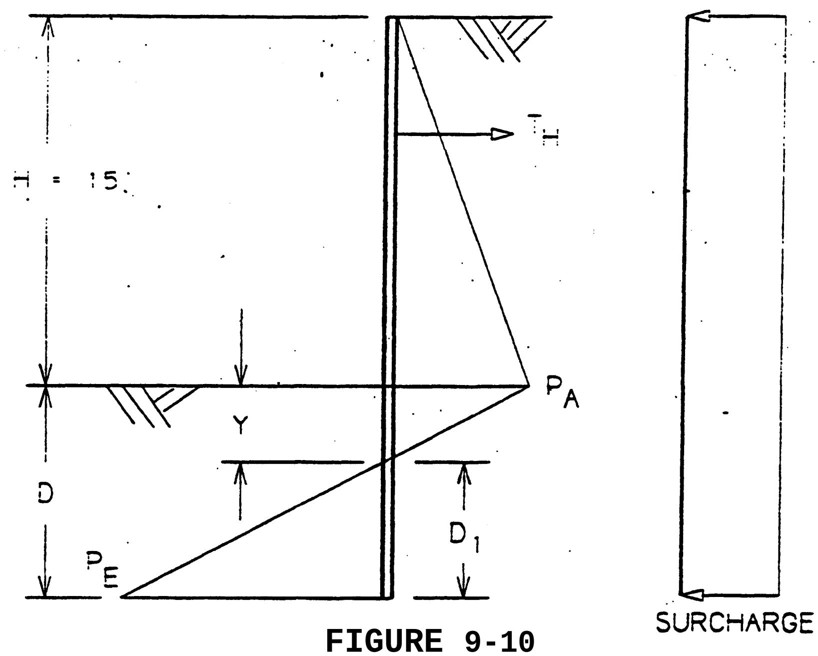


FIGURE 9-10

Use friction angle $\delta = \phi/2 = 17.5°$

From Log-Spiral: $K_a = 0.27$

From Log-Spiral: $K_p = 10.5(0.362) = 3.8$
(It is recommended that friction not be used for the passive condition; especially under conditions of dynamic loading).

$P_A = (\gamma)(H)(K_a)\cos\delta = (115)(15)(0.27)\cos 17.5° = 444.0$ psf

$P_E = \gamma(K_p - K_a)D_1 = 115(3.53)D_1 = 406D_1$ psf

$Y = P_A/\gamma(K_p - K_a) = 444/406 = 1.09$ feet

Horizontal Forces:

$P_1 = P_A(H)/2 = 444(15)/2 = 3{,}330$ Lb/LF

$P_2 = P_A(Y)/2 = 444(1.09)/2 = 242$

$P_{SC} = 100(H + Y + D_1) = 100(16.09 + D_1) = 1{,}609 + 100D_1$

$P_3 = P_E(D_1)/2 = 406D_1(D_1)/2 = 203(D_1)^2$

T_H = Unknown

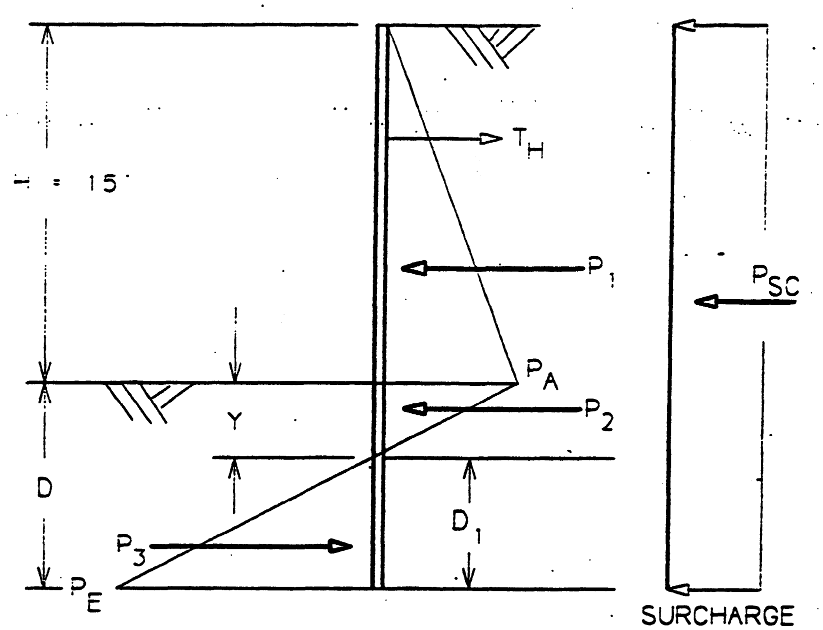


FIGURE 9-11

Sum Forces ($\Sigma F_H = 0$) → + :

$$T_H + P_3 - P_2 - P_1 - P_{SC} = 0$$

$$T_H + 203(D_1)^2 - 242 - 3{,}330 - (1{,}609 + 100D_1) = 0$$

EQUATION 1: $T_H = -203(D_1)^2 + 100D_1 + 5{,}181$

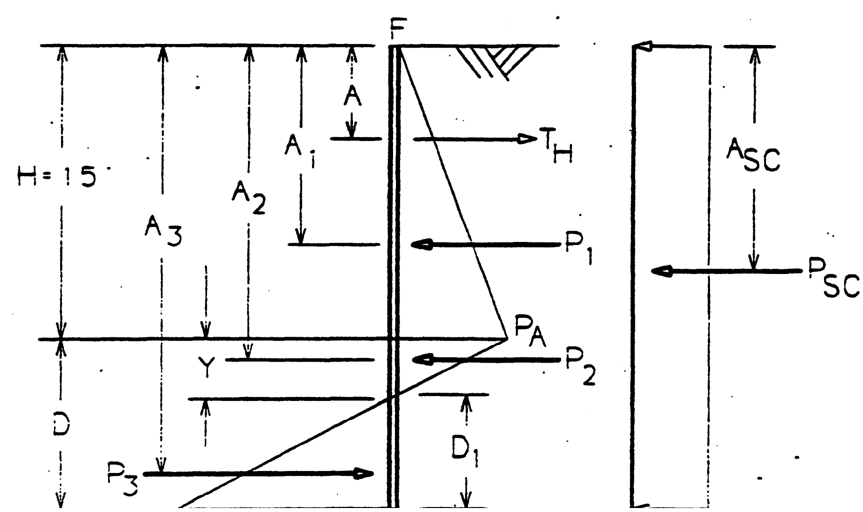


FIGURE 9-12

Moments About Top of Shoring:

$$
\begin{aligned}
P_1[2/3(H)] &\quad 3{,}330[10]\\
&= 33{,}300\\
P_2[15 + Y/3] &= 242[15.36]\\
&= 3{,}717\\
P_{sc}[(H + D)/2] &= P_{sc}[(H+Y+D_1)/2]\\
&= P_{sc}[(16.09+D_1)/2]\\
&= (1{,}609 + 100D_1)[(16.09 + D_1)]/2\\
&= 50(D_1)^2 + 1{,}609D_1 + 12{,}944\\
P_3[H + Y + 2/3D_1] &= 203(D_1)^2[16.09 + 2/3D_1]\\
&= 3{,}266(D_1)^2 + 135(D_1)^3\\
T_H[3.5] &= 3.5T_H
\end{aligned}
$$

Sum Moments ($\sum M_P$) = 0: (Use clockwise moments negative)

$$3.5T_H + 135(D_1)^3 + 3{,}266(D_1)^2 - 50(D_1)^2 - 1{,}609D_1 - 12{,}944 -3{,}717 - 33{,}300 = 0$$

$$3.5T_H = -135(D_1)^3 - 3{,}216(D_1)^2 + 1{,}609D_1 + 49{,}961$$

EQUATION 2:

$$T_H = -38.6(D_1)^3 - 918.9(D_1)^2 + 459.7D_1 + 14{,}2174.6$$

Equate EQUATION 1 = EQUATION 2 and solve for D:

$$- 38.6(D_1)^3 - 715.9(D_1)^2 + 359.7D_1 + 9{,}093.6 = 0$$

$$(D_1)^3 + 18.5(D_1)^2 - 9.3D_1 - 235.6 = 0$$

From which $D_1 \approx 3.50'$

then $D \approx 3.50 + 1.09 = 4.6'$

Increase D for factor of safety: 1.4(4.6) = 6.4 < 6'-6"

Solve EQUATION 1 for T_H:

$T_H = -203(3.50)^2 + 100(3.50) + 5{,}181 = 3{,}044.2$ Lb/LF

With ties at 8'-0" spacing design $T_H = 8(3{,}044) = 24{,}352$

Compute Tieback Forces:

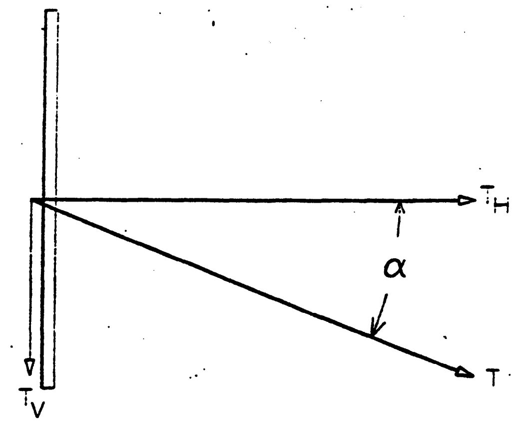


FIGURE 9-13

$T = T_H/\cos\alpha = 24{,}352/\cos 15° = 25{,}211$ Lb > 25K per plans

This difference is too small to be considered significant.

USE T = 25K

$T_H = 25{,}000 \cos 15° = 24{,}148$ Lb = $24{,}148/8 = 3{,}019$ Lb/LF

$T_V = 25{,}000 \sin 15° = 6{,}470$ Lb

Check Downward Force due to Prestressing:

Resistance to downward force is furnished by the skin. friction on both sides of the embedded sheet piling.

From TABLE 12 N = 27 for $\phi = 35°$

Friction resistance = N/100 tsf = 27/100 = 0.27 tsf

Resistance for 8'-0"
= 0.27(2,000 Lb/ton)(2 sides)(D)(Spacing)
= 0.27(2,000)(2)(6.5)(8)
= 56,160 Lb

Use safety factor of 2:

Resistance = 56,160/2 = 28,080 > T_V = 6,470 Lb OK

Locate Plane of Zero Shear for Sheet Piling:

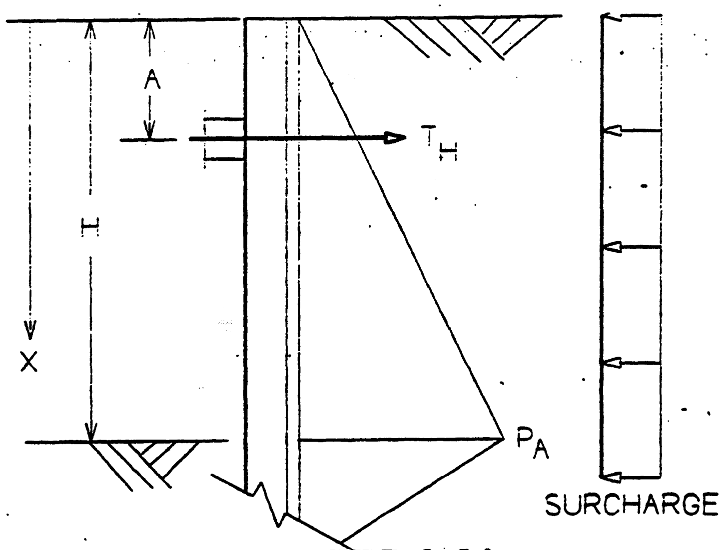


FIGURE 9-14

Assume $3.5' \leq X \leq 15'$

$$K_A\gamma(X^2/2)(\cos\delta) + 100X - T_H = 0$$

$$[0.27(115)(X^2/2)](\cos 17.5°) + 100X - 3{,}019 = 0$$

$$14.8X^2 + 100X - 3{,}019 = 0$$

$$X^2 + 6.8X - 204.0 = 0$$

The plane of zero shear is where $X = 11.3'$

Sheet Pile Moment and Section Modulus:

$$M_{MAX} = K_A\gamma(X^2)/2(\cos\delta)[X/3] + 100X[X/2] - T_H[X-3.5]$$

$$= 0.27(115)[(11.3)^2/2](0.95)[11.3/3] + 100(11.3)[11.3/2] -3{,}019[7.8]$$

$$= 7{,}094 + 6{,}385 - 23{,}548 = -10{,}069 \text{ Ft-Lb/LF}$$

$$S_{REQUIRED} = M/f = 10{,}069(12)/25{,}000 = 4.83 \text{ in}^3$$

For the PSX32 sheet pile section to be used $S = 2.4$ in^3/LF and $I = 3.7$ in^4/LF (see Table 19). Sheet pile sections this flexible are not generally used adjacent to traffic or other critical surcharge loads, but are being used here for illustrative purposes.

Since analysis is based on the Free Earth Support Method, Rowe's Moment Reduction Theory may be utilized.

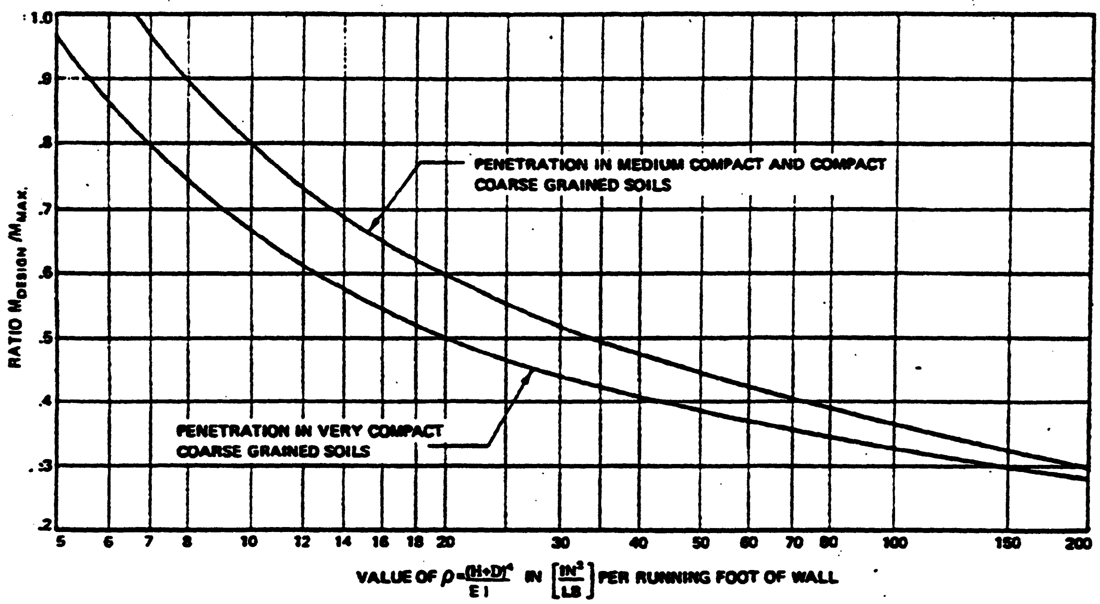


FIGURE 9-15

$\rho = (H + D)^4/EI = [(15 + 6.5)(12)]^4/(29x10^6)(3.7) = 41$

Use the curves in the preceding diagram for, 'PENETRATION IN MEDIUM COMPACT TO COMPACT COARSE GRAINED SOILS', to obtain the moment ratios:

Ratio $M_{DESIGN}/M_{MAX} = 0.47$

$M_{DESIGN} = 0.47(10,069) = 4,732$ Ft-Lb/LF

$f = M/S = 4,732(12)/2.4 = 23,660 < 25,000$ psi OK

<u>Check Anchor Tendon Capacity:</u>

Plan calls for 5/8" Dywidag bars spaced at 8'-0" centers:

$F_{ult} = 157$ ksi

$A_{bar} = 0.28$ in^2

<u>Allowable Bar Capacity:</u>

$T_{DESIGN} \leq 0.6F_{ult}A_{bar} = 0.6(157)(0.28) = 26.4$ K

$T_{PROOF} \leq 0.8F_{ult}A_{bar} = 0.8(157)(0.28) = 35.4$ K

Actual load on bars:

T_{DESIGN} = 25 K < 26.4 K OK
T_{PROOF} = 1.3(25) = 32.5 K < 35.4 K OK

H = 15'
A = 3.5'
L_u = 15'
L_b = 25'
ϕ = 35°
γ = 115 pcf

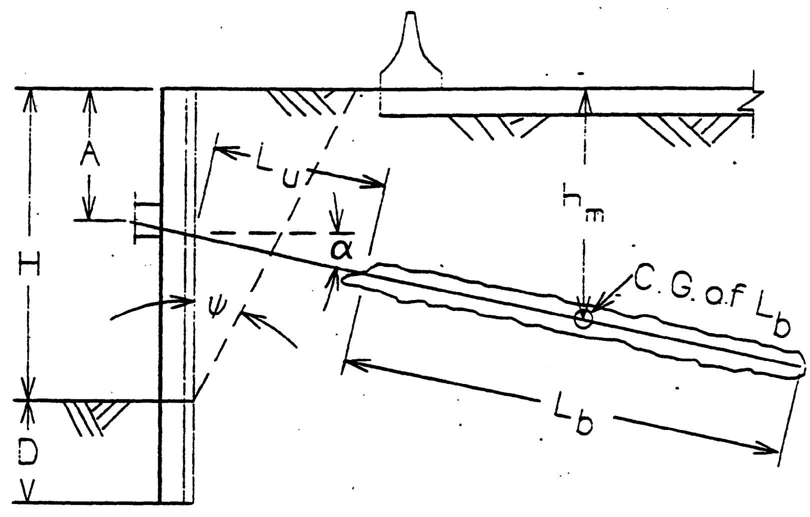


FIGURE 9-16

Assumed
failure wedge angle $\psi = 45° - \phi/2 = 27.5°$ $\alpha = 15°$

Check L_u minimum:

L_u minimum = $(H - 3.5')(\sin\psi)/\sin[180° - (90° - \alpha) - \psi)$

= $(15' - 3.5')\sin 27.5°/\sin 77.5°$

= 5.4' < 15' OK

Determine h_m:

$h_m = 3.5 + (L_u + L_b/2)\sin\alpha$

$= 3.5 + (15' + L_b/2)\sin 15°$

$= 7.4' + 0.13L_b$

Determine L_b using the FHWA formula:

$$P_{ult} = \pi(d)(L_b)(\gamma)(h_m)\tan\phi$$

$$= \pi(0.5)(L_b)(115)(7.4 + 0.13L_b)\tan 35°$$

$$= 936L_b + 16.4(L_b)^2 = 33{,}650 \text{ Lb}$$

$$P_{ult} = 33{,}650 \text{ Lb} > T_{PROOF} = 32{,}500 \text{ Lb} \quad \text{OK}$$

Proof testing will verify actual anchor capacities.

Simplified Wall Stability Check For Single Tier Tieback:

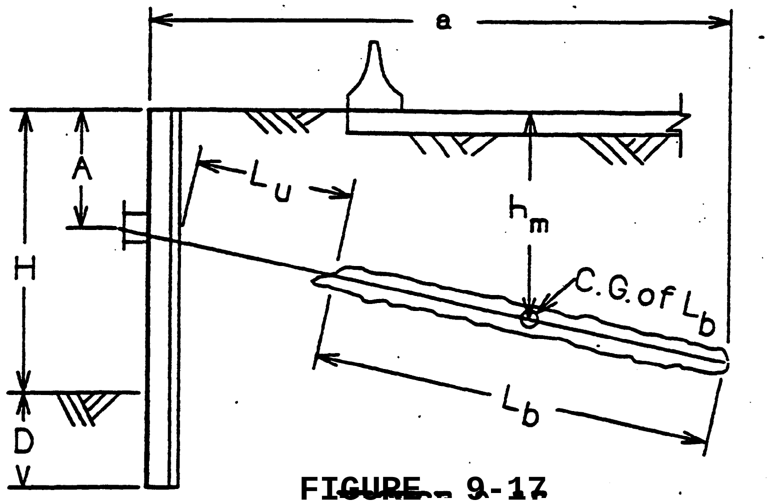


FIGURE 9-17

If a/(H + D) > 1, The wall may be considered stable.

where: a = horizontal component of the tie.

$= (L_u + L_b)\cos\alpha$

$= (15' + 25')\cos 15° = 38.6$ feet

$a/(H + D) = 38.6'/(15' + 6.5') = 1.8 > 1$ OK

Lock-Off Force:

A value of $0.8T_{DESIGN}$ is typically recommended as a minimum value for low to normal risk conditions. The use of $0.8T_{DESIGN}$ would be satisfactory for this case provided small settlements behind the wall will not be detrimental.

Check Proof Loading:

$T_{PROOF} = 1.3T_{DESIGN} = 1.3(25{,}000) = 32{,}500$ Lb

$\Delta = 0.1$ inch $T_1 = 1$ minute $T_2 = 15$ minutes

$T_3 = (1/2 \text{ Yr.})(365/2 \text{ D/Y})(24 \text{ H/D})(60 \text{ M/H}) = 262{,}800$ minutes

$$C = \Delta_{1\text{-}2}/[\log_{10}(T_2/T_1)] = 0.1/[\log_{10}(15/1)] = 0.085$$

$$\begin{aligned}\text{Long term } \Delta &= (C)\log_{10}(T_3/T_2)\\ &= (0.085)\log_{10}(262{,}800/15)\\ &= 0.36 \text{ in.}\end{aligned}$$

A long term movement of the wall can be approximated but if neither wall movement nor settlement behind the wall will. be detrimental then 0.36 inch would be acceptable.

SAMPLE PROBLEM 9-3 MULTIPLE-TIER TIEBACKS (PART 1)

This is a two part sample problem. The first part is a sample design using simplified criteria assuming the vertical member to be hinged at the depth of excavation. The second part is an analysis of the design and tiebacks using OSC criteria.

PART1

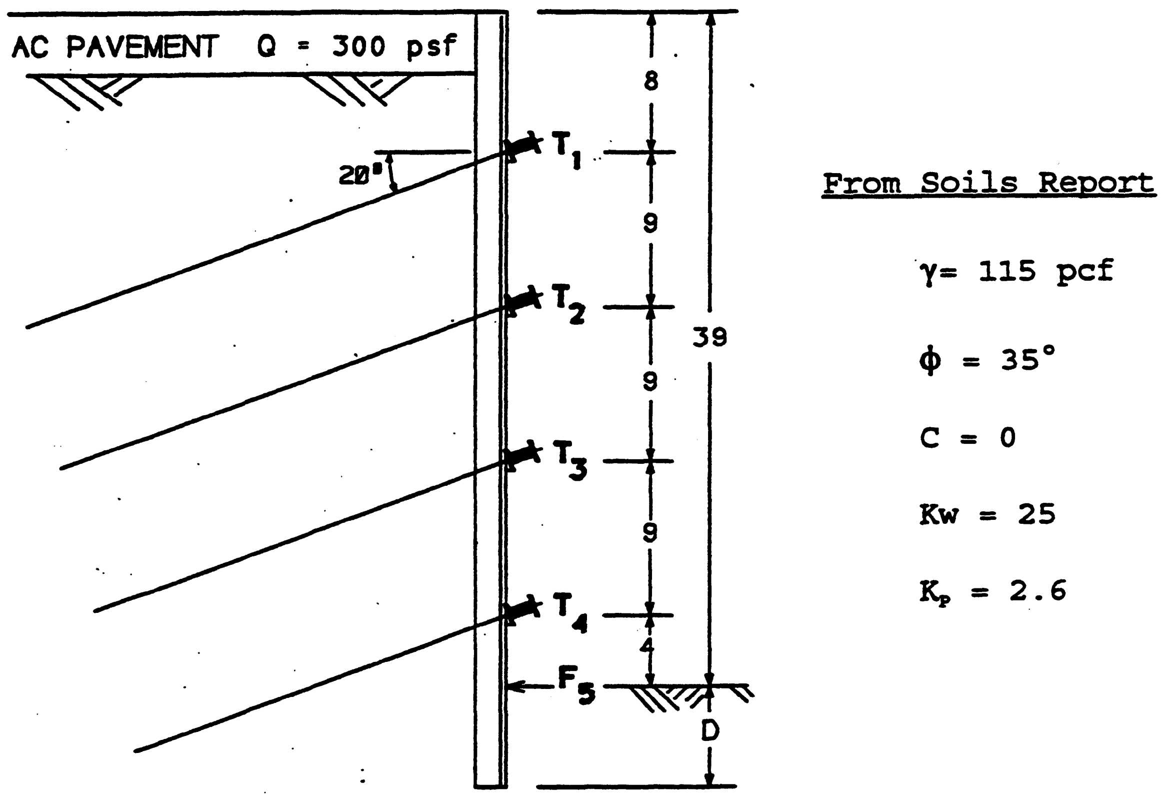


FIGURE 9-18

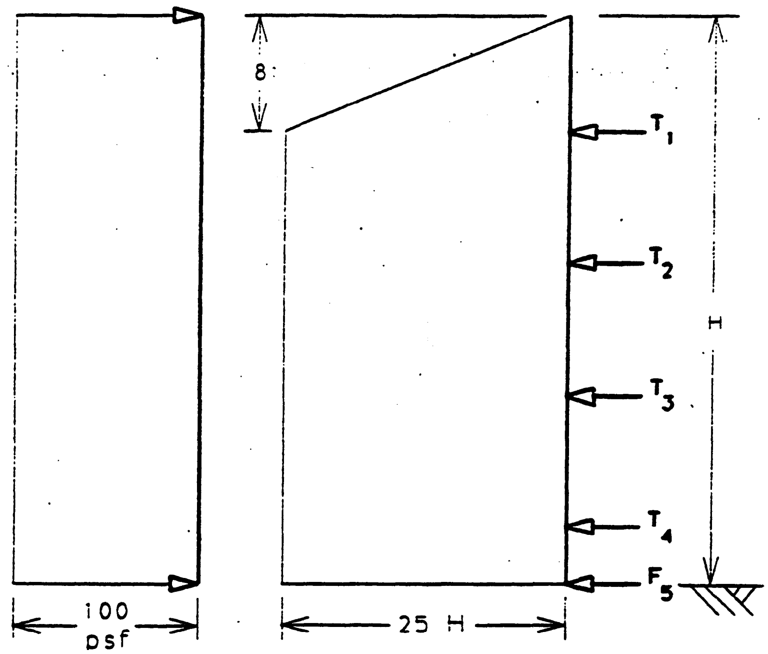
Driven New Steel Sheet Piling: Casteel CS60:

$S = 6.98\ in^3/Ft^2$ of wall

$I = 20.6\ in^4/LF$

$F_b = 25$ ksi

Tiebacks spaced at 7' -6" along with W16 x26 wales



SURCHARGE SOIL PRESSURE

FIGURE 9-19

Shear and moment at T_1 due to cantilever:

$V_c = 0.5(0.025)(H)(8) + 0.100(8) = 4.7$ Kip/LF

$M_c = 3.9(8/3) + 0.8(8/2) = 13.6$ K-Ft/LF

COMPUTER PROGRAM RESULTS:

at	Moment (Ft-Kips/LF)	Reaction (Kips/LF)
T_1	13.6	10.4
T_2	5.5	8.5
T_3	8.1	10.3
T_4	5.4	8.0
F_5	0	0.8

Approximate Maximum Positive Span Moments:

span 1 = 1.7 Ft-Kips/LF

2 = 4.1

3 = 4.1

4 = 0.3

Design Moment = Maximum Moment = 13.6 Ft-Kips/LF

Section Modulus Required 13.6(12)/25 = 6.52 < 6.98 in^3 OK

Tieback Forces:

$T_1 = 7.5(10.4) = 78$ K

T_1 Load = 78/cos 20° = 83 K

T_1 Test = 83/0.8 = 104 K

$T_2 = 7.5(8.5) = 63.75$ K

T_2 Load = 63.75/cos 20° = 68 K

T_2 Test = 86/0.8 = 108 K

$T_3 = 7.5(10.3) = 77.25$ K

T_3 Load = 77.25/cos 20° = 82 K

T_3 Test = 82/0.8 = 103 K

$T_4 = 7.5(8) = 60$ K

T_4 Load = 60/cos 20° = 64 K

T_4 Test = 64/0.8 = 80 K

Depth of Embedment:

Downward Force = (83 + 68 + 82 + 64)sin 20° = 101.58 K

Friction on sheet piling = (78 + 64 + 77 + 60)(0.4) = 111.6

111.6 > 101.58 K

Use passive resistance = 2.6 times the unit weight of the soil.

Passive resistance = 2.6(115) = 300 pcf

The primary portion of the downward force from the tieback load may be acting on one sheet pile, so prorate the load to the pile width of 27.6 inches (2.3 feet):

F_5(7.5/2.3) = 800(7.5/2.3) = 2,609 Lb/Sheetpile

This force needs to be resisted by a passive triangular load having a resistance of 300D for depth D so that:

$2{,}609 = 1/2(300)(2.3)D^2$ from which D = 2.75'

Use safety factor of 30%. Design D = 2.75(1.30) = 3.58'
Use D = 4.0 feet

Check Excavation Levels at 2'-0" Below Ties:

EXCAVATE 10'

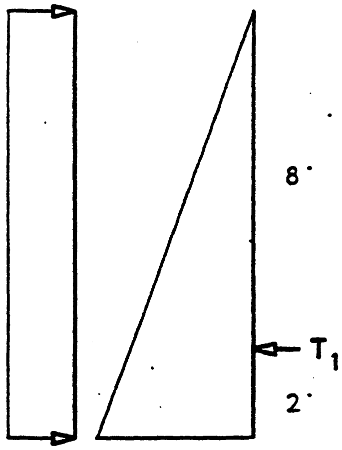


FIGURE 9-20

Before stressing T_1:

$M_{10'} = 25H(H/2)(H/3) + 100(H)(H/2)$
$= 25H^3/6 + 100H^2/2$
$= 9{,}167$ Ft-Lb/LF

After stressing T_1:

$M_{10'} = 9{,}167 - 10{,}400(2)$
$= -11{,}633$ Ft-Lb/LF

11,633 < 13,600 Ft-Lb/LF OK

EXCAVATE 19'

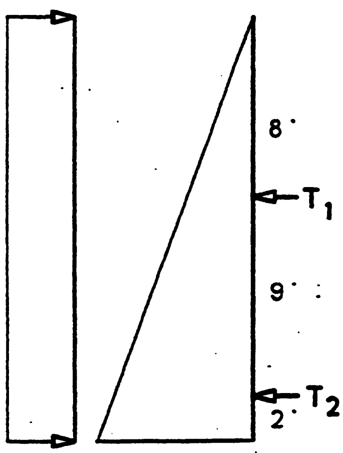


FIGURE 9-21

Before stressing T_2:

$M_{19'} = 25(19)^3/6 + 100(19)^2/2$
$- 10{,}400\ (11)$
$= -67{,}770$ Ft-Lb/LF > 13.6 K-Ft/LF

But which will be resisted the passive soil behind the piling ($K_p > K_a$).

And remainder OK by similar analysis.

MULTIPLE-TIER TIEBACKS (PART 2)

The traveled way pavement width of 15' - 2" ± starts 6 feet from the face of excavation. Surcharge loading for traffic is 300 psf acting vertically.

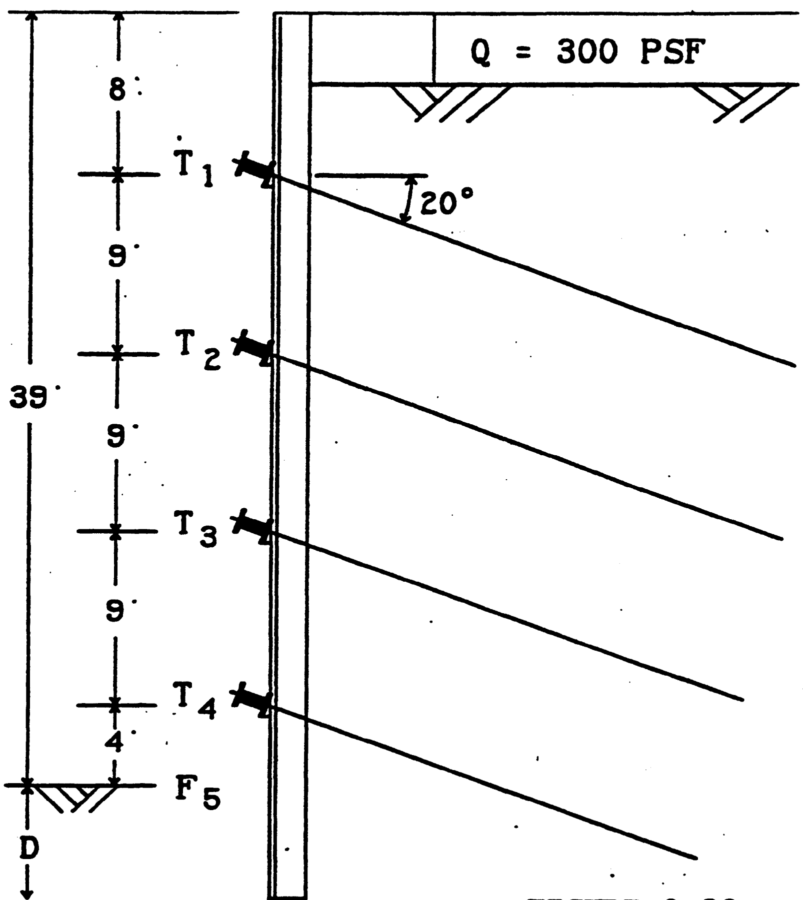


FIGURE 9-22

From Soils Report:

Soil unit weight γ = 115 pcf   Wall friction δ = 11°
Internal friction angle ϕ = 35° Cohesion C = 0
Equivalent unit weight Kw = 25 pcf

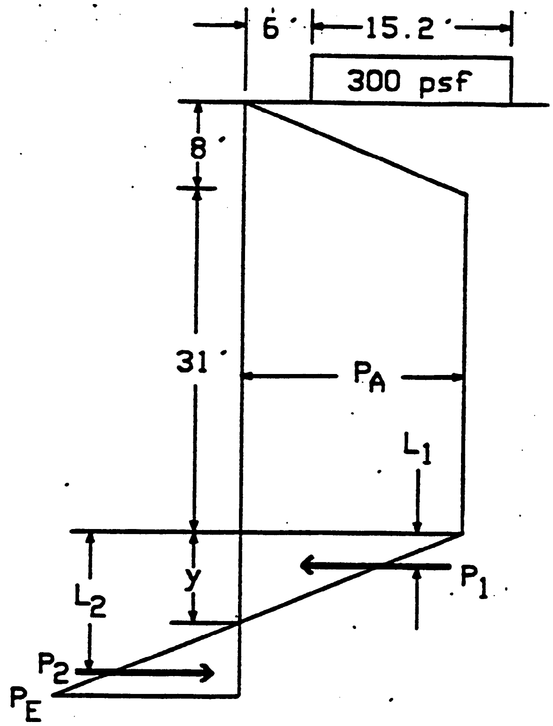
Shoring is driven new steel sheet piling:

Casteel CS60 for which: S = 6.98 in^3/Ft of wall
I = 20.6 in^4/LF
F_b = 25 ksi

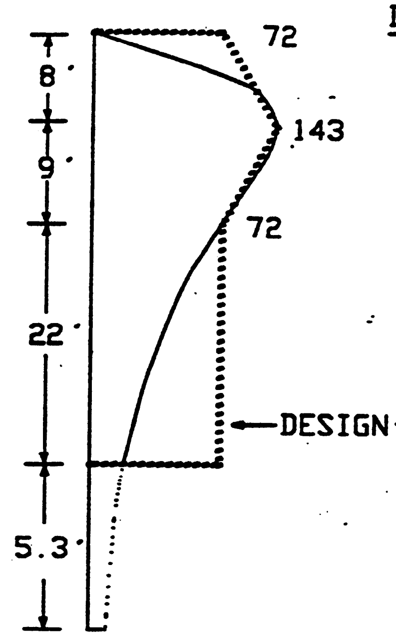
Tiebacks spaced at 7' - 6" along W16 x 26 wales

ANCHORED SHORING SYSTEMS

Use the Boussinesq loading for the surcharge and assume this loading-carries to the bottom of the excavation.



SOIL PRESSURE



DEPTH	PRESSURE
0	0 (72)
2	83
4	130
6	143
8	137
10	123
12	107
14	92
16	79
18	67(72)
↓	↓
44	10(0)

SURCHARGE PRESSURE

9-23

Use: $K_a = 0.28$, $K_p = (8.4)(0.593) = 4.98$

$P_A = 0.71K_a\gamma H(\cos\delta) = (0.71)(0.28)(115)(39)(\cos 11°) = 875$ psf

$P_E = \gamma D(K_p - K_a) - P_A = (115)(5.3)(4.70) - 875 = 1{,}990$ psf

$y = P_A/\gamma(K_p - K_a) = 875/(115)(4.70) = 1.62'$

$P_1 = (875)(1.62)/2 = 709$ Lb

$P_2 = (1{,}990)(5.3 - 1.62)/2 = 3{,}662$ Lb

$L_1 = y/3 = 1.62/3 = 0.54'$

$L_2 = y + (D - y)(2/3) = 1.62 + (5.3 - 1.62)(2/3) = 4.07'$

Two approaches may be considered for force P_2:*

1. Consider leg T_4-P_2 as a cantilever. <---Use option #1 for this analysis
2. Consider P_2 an auxiliary support.

* More recent thinking permits a hinge at the excavation line.

Moment Distribution Factors & Fixed End Moments:

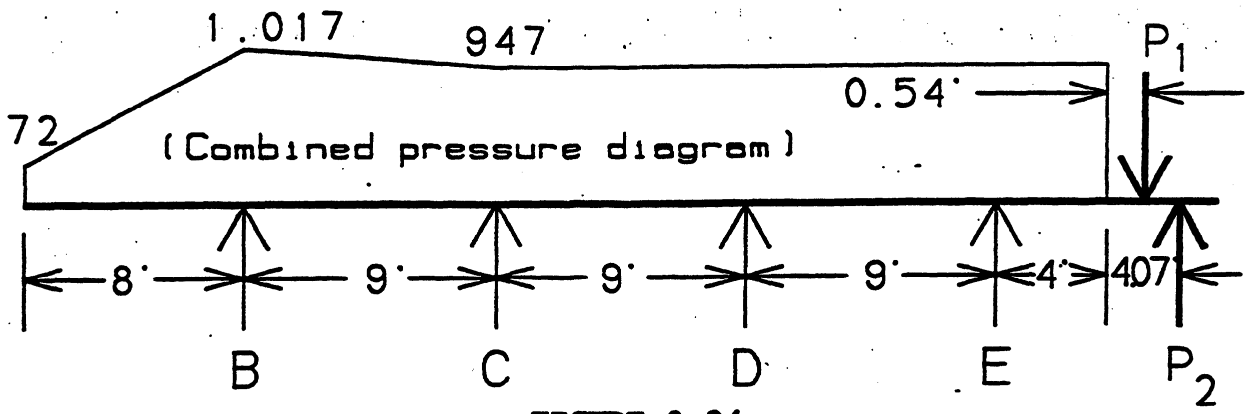


FIGURE 9-24

BC, CB, CD, DC, DE, ED = 4EI/L = 4/9 =0.444
(EI = constant)

Distribution Factors:

0.444/(0.444 + 0.444) = 0.5

BA = 0	CB = 0.5	DC = 0.5	ED = 1
BC = 1	CD = 0.5	DE = 0.5	EP_2 = 0

Rectangular Loads: $M = wL^2/12$

Triangular Loads: Light end, $M = wL^2/30$
Heavy end, $M = wL^2/20$

M_{BA} = (72)(8)[8/2] + {(1,017 - 72)(8)/2}[8/3] = 12,384

M_{BC} = (875 + 72)$(9)^2$/12 + (142 - 72)$(9)^2$/20 = 6,676

M_{CB} = (875 + 72)$(9)^2$/12 + (142 - 72)$(9)^2$/30 = 6,581

M_{CD} = M_{DC} = M_{DE} = M_{ED} = (875 + 72)$(9)^2$/12 = 6,392

M_{EP2} = (947)(4)[2] + 709[4.54] - 3,662[8.07] = -18,757

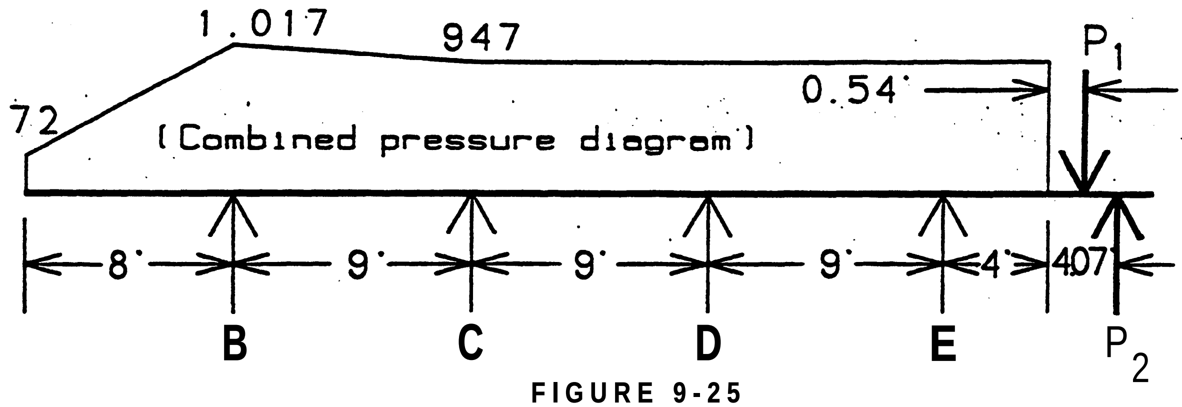


FIGURE 9-25

	0	1	0.5	0.5	0.5	0.5	1	0
	-12384	6676	-6581	6392	-6392	6392	-6392	-18757
		5708	95	94	0	0	25149	
		47	2854	0	47	12575	0	
		-47	-1427	-1427	-6311	-6311	0	
		-714	-24	-3156	-714	0	-3156	
		714	1590	1590	357	357	3156	
		795	357	178	795	1578	178	
		-795	-268	-268	-1187	-1187	-178	
		-134	-398	-593	-134	-89	-593	
		134	496	496	112	111	593	
		248	67	56	248	296	56	
		-248	-62	-61	-272	-272	-56	
		-31	-124	-136	-30	-28	-136	
		31	130	130	29	29	136	
		65	15	16	65	68	14	
		-65	-16	-15	-66	-66	-14	
M_{TOT}	-12384	12384	-3296	3296	-13453	13453	18757	-18757
V_M		1010	-1010	-1096	1096	3579	-3579	
V_{SB}	4356	4472	4367	4262	4262	4262	4262	835
V_{TOT}	4356	5482	3357	3166	5358	7841	683	835

Check Sheeting:

The maximum moment is M_{EP2} = 18,757 Ft-Lb/LF
S Required = M/f = (18,757) (12)/25,000 = 9.00 in^3

Analysis by the free earth support method permits the use of Rowe's Theory of Moment Reduction.

$p = (H + D)^4/EI = ((39 + 5.3)(12)\}^4/(30 \times 10^6)(20.6) = 129.2$
From-Rowe's moment reduction curves (see USS Steel Sheet

Piling Design Manual, page 32):

RATIO M_{DESIGN}/M_{MAX} = 0.34

$\therefore M_{DESIGN}$ = (0.34)(18,757) = 6,377 Ft-Lb/LF

S Required = (6,377)(12)/25,000 = 3.06 < 6.98 $\therefore$ O.K.

Check Wales:

$M \approx wL^2/10 \approx (13,199)(7.5)^2/10 \approx 74,244$ Ft-Lb
S Req'd = (74,244(12)/22,000 = 40.5 in^3
S furnished (W16 x 26) = 38.4 < 40.5 $\therefore$ Not satisfactory

Recommend using W14 x 30; S = 42.0

Determine Tieback Loads:

B: (T_1) = 7.5(9,838) = 73,785 Lb
C: (T_2) = 7.5(6,523) = 48,923 Lb
D: (T_3) = 7.5(13,199) = 98,993 Lb
E: (T_4) = 7.5(1,518) = 11,385 Lb

Tieback 1 (installed in 8" diameter drilled hole)

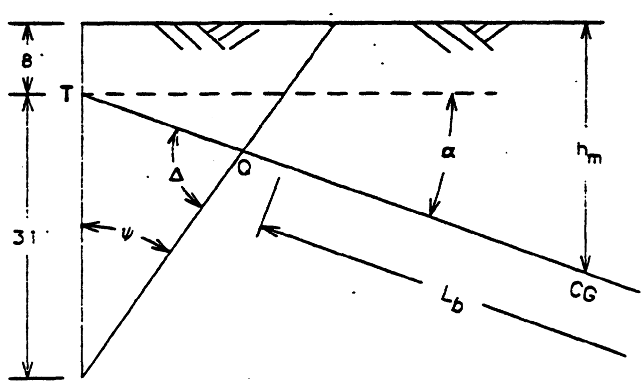


FIGURE 9-26

T_1Q = Unbonded length

L_b = Bonded Length

T_1 = 73,785 Lb

P = 73,785/cos 20°

P = 78,520 LB

ψ = 35°

$\Delta = 180° - (90-\alpha) - \psi$ = 75°

T_1Q = 31(sin ψ)/sin Δ = 31(0.5736)/0.9659 = 18.4' > 15'

h_m = Distance from ground surface to center of length L_b.

$$h_m = 8 + (T_1Q + L_b/2)\sin\alpha$$
$$h_m = 8 + (18.4 + L_b/2)(0.342) = 14.29 + 0.171L_b$$

Proof load = $P_{ULT} = P_{DESIGN}/0.8 = 98{,}150$ Lb

Use the FHWA formula to verify L_b

$$8{,}150 = \pi d\gamma h_m(\tan\phi)L_b$$
$$98{,}150 = \pi(0.667)(115)(14.29 + 0.171L_b)(\tan 35°)L_b$$
$$98{,}150 = 2{,}411.6L_b + 28.85L_b^2$$

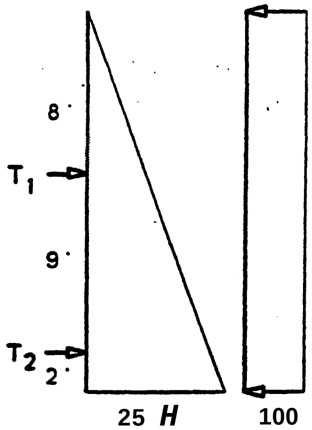
$$L_b^2 + 83.58\,L_b - 3{,}402.08 = 0 \qquad L_b = 30'$$

And similar computations can be made for the other ties.

<u>Check Excavation Levels at 2' -0" Below Tie to be Installed:</u>

<u>**EXCAVATE 10'**</u>

EXCAVATION 19'



Before stressing T_2:

$$M_{19'} = [25(19)^3/6 + 100(19)^2/2] - 9{,}838(11)$$

$$= -78{,}689 \text{ Ft-Lb/LF}$$

Indicating a larger sheet pile section is needed.

FIGURE 9-28

The remainder of the ties need to be checked. A similar analysis might be required for backfill operations because the elevations used for stopping for installation of ties may not the same as that used for removal of ties.

Check Sheet Pile Penetration:

Use skin friction on the sides of the piling in contact with the soil; and-use skin friction working load value = 50% of ultimate value.

Skin friction = N/100 = Ultimate value

N from Table 12 = 20

N/100 = 20/100 = 0.2 tsf = 400 psf

Working load value = 50% of the ultimate, ∴ use 400/2 = 200 psf

Downward load from the T forces = $\sum T(\tan 20°)$ = 31,078 (0.364)

= 11,312 Lb/LF

11,312(7.5) = (39 + 2D)(7.5)(200) from which D = 8.8'

Use Safety Factor = 20% (Shape of the sheet piling was neglected)

Minimum D = 1.20(8.8) = 10.6 feet > 5.6 feet shown on plan.

ANCHORED SHORING SYSTEMS

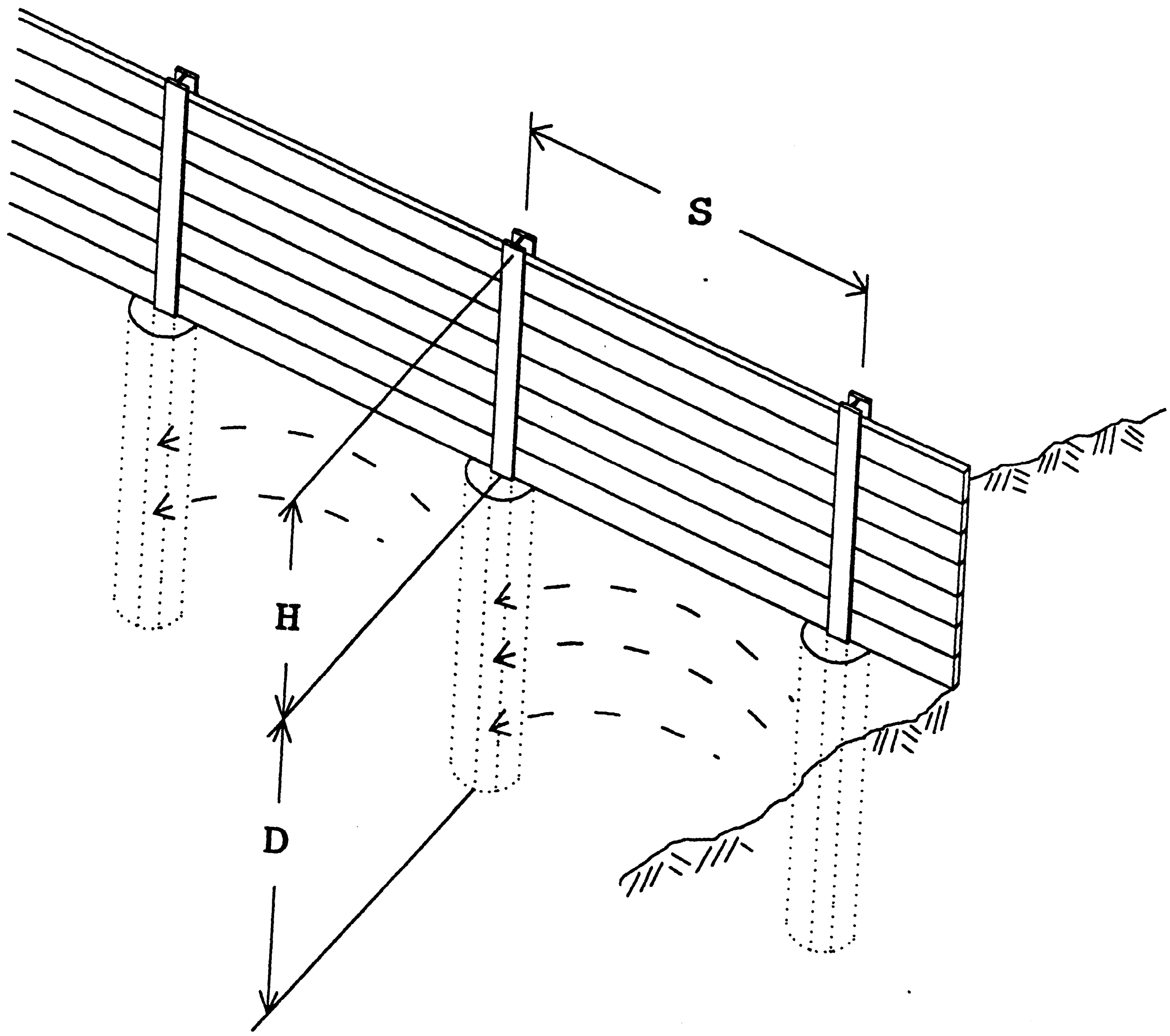
SUMMARY

Multiple tieback systems approximate a multiple strutted system. The soil pressure diagram for either system should more appropriately approximate a trapezoid rather than a triangle. This would be especially true for soft to medium clays.

A long bond length is required at the elevation of the upper tier primarily because of the low h_m value. The tiebacks of the upper tier would have been better designed by reducing the center to center tie spacing to achieve a shorter required bond length. Another way to reduce the bonded length would be to locate the upper ties tiff-center with respect to the second tier ties and to increase the tie slope angle in order to increase the h_m value. The most practical way to decrease the length requirement of the upper tier tie would be to increase the diameter of the drilled hole to 16ⁿ or to 18". This would. substantially increase 'the effective bond per linear foot of tie.

Three tiers of tiebacks properly spaced should have been adequate for the soil conditions and design parameters used in this case.

SOLDIER PILE SYSTEMS



SOLDIER PILES

SOLDIER PILE SYSTEMS

Soldier piles of varying materials and sections are used, often in conjunction with some form of lagging to support soils as a continuous wall above the depth of excavation. Soldier piling elements may consist of HP or wide flange sections, sheet pile sections, or CIDH piles. Lagging may consist of horizontally placed wood members, steel plates, or concrete sections.

Soil loads are transferred to soldier piles partially through the lagging and partially through soil arching. A semi-circular section of soil immediately behind the lagging may represent all the load that gets transmitted to the lagging. When the soil between soldier piles is capable of self support the soil loads will transfer to the adjacent soldier piles, and no lagging will be needed. This soil load transfer is referred to as soil arching. Compact or cohesive soils will demonstrate a greater ability for soil arching than will loose and cohesionless soils. However, the looser soils will tend to load the lagging more.

Stiff soils exhibit an ability to stand unsupported for some height for some period of time. This is evident by comparison to relatively small square or rectangular excavations where no shoring is used. The soil behind and along the cut faces transmit the lateral forces to the vertical corners through soil arching. Soldier piles act in the same manner as the vertical corners.

The general design procedure for soldier pile walls is to assume one half the pile spacing either side of the pile acts as a panel loaded with active soil pressures and surcharge loadings above the depth of excavation. The portion of soldier pile below the depth of excavation is likewise loaded with both active soil pressure and surcharge loadings.

Resistance to lateral movement or overturning (about any point) of the soldier pile is furnished by the passive resistance of the soil below the depth of excavation. The depth of pile penetration must be sufficient to prevent lateral movement or tip over (about the base) of the soldier pile system. To account for soil disturbance at the excavation elevation AASHTO recommends that any passive resistance be ignored or discounted for a distance equal to 1.5 times the effective pile diameter immediately below the depth of the excavation.

Soldier piles may be driven or they may be installed in drilled holes. Drilled holes may be backfilled with concrete, slurry, sand, pea-gravel or similar material after the soldier pile has been installed in the hole. Some soldier pile drilled holes are backfilled with concrete to the depth of excavation and then the remainder of the hole is filled with slurry to ground level. Slurry is generally considered to be a sand-cement mix placed wet enough to fill all voids. Backfill materials other than concrete are used when it is desirable to extract the soldier piles. When materials other than concrete are used to backfill drilled soldier pile holes some vibratory methods and/or jetting procedures need to be used. The backfill should be as compact as the native soil into which the pile is set.

The use of pea gravel backfill in lieu of concrete substantially decreases the passive resistance on all sides of the soldier pile. The pea gravel does not permit the soldier pile to act as a unit until sufficient soldier pile deflection compacts the pea gravel against the soil. Similar reasoning is true for compacted sand backfill.

Effective Width:

The effective width of a soldier pile is considered to be the dimension of the soldier pile taken parallel to the line of the wall for piles either driven, or placed in drilled holes backfilled with materials other than concrete. When hard rock concrete is used for the backfill of drilled holes the effective width of the soldier pile is the diameter of the drilled hole. Structural concrete is generally considered to be a 4 sack or better concrete mix. Properly placed lean concrete can also be effective. However, lean concrete must be sufficiently strong to prevent collapse of the hole, yet weak enough to be excavated easily. A lean concrete mix is normally about 1 to 2 sacks of cement per cubic yard with a minimum specified strength of 2,400 psi.

Experimentation (1970's and later) has determined that the passive resistance of cohesionless soils acts over a width greater than the effective width of the soldier pile. The pressure exerted by the laterally pulled soldier pile produces what amounts to a wedge shaped resisting soil configuration. This soil failure configuration offers a resistance similar to the resistance that would act over something wider than the effective pile width.

Because of the apparent increase in passive resistance previously mentioned, the effective widths of soldier piles installed in cohesionless soils may be increased by an adjustment factor (passive arching capability) of 0.08 times the internal friction angle of the soil (0.08ϕ), but not to exceed a value of 3.00.

> This means for example, that the final adjusted design width for a soldier pile with a flange width of 14" installed in soil which has a ϕ angle of 38^{o}, not installed in hard rock concrete, would be equivalent to [(0.08)(38)](14/12) = [3.00 max] (14/12) = 3.5 feet. The full value of 3.5 feet per soldier pile could be used provided the pile spacing is greater than 3' - 6". Care will have to be exercised to be sure that no more than 1 pile spacing is used for the final adjusted widths so that none of the widths overlap.

SOLDIER PILES

Adjustments:

Adjustment Factor = Arching Capability = 0.08ϕ ($\leq$ 3.00)

Adjusted Pile Width = (Effective Width)(0.08ϕ) $\leq$ 1 Pile Spacing

For cohesive soils the adjustment factor for increasing the effective soldier pile width ranges between 1 to 2. Permissible adjustment factors are listed in Table 104.

For excavations adjacent to railroad tracks the AREA recommendations specify that ϕ and C values be reduced 15% for the effects of dynamic loading when these values are determined by a qualified soils analysis laboratory. See the railroad requirements in the appendix.

Below the excavation depth the final adjusted width may be used for the passively loaded side of the pile. The same final adjusted width may be used for the active and surcharge loading also. Some consultants have used only the effective width of the piling for the active and surcharge loaded width (not advisable).

Soldier Piles As Sheet Piling:

Soldier piling can be analyzed in the same manner as sheet piling when the active loaded width below the depth of excavation is assumed to be the same as the passive loaded width.

When soldier piling is analyzed in the same manner as sheet piling either the loaded panel width (the pile spacing) or the effective soldier pile width must be adjusted. Proportioning the soldier pile effective width to the pile spacing permits analysis on a per foot basis of wall as is done for sheet pile analysis. For example, a soldier pile spaced on 8'-0" centers having a final effective pile width of 2'- 0" has an equivalent sheet pile width of 1'-0" above the excavation line and 2'/8' = 0.25 foot width below that line. Completion of the soldier pile computations using sheet pile analysis is accomplished by increasing all answers for moments and shears by a factor equal to the soldier pile spacing, in this case 8 feet.

An easy method for converting from soldier pile to sheet pile analysis involves determining an Arching Factor (f). The value of f is determined by multiplying the adjustment factor (Passive Arching Capability listed in Table 10-l), by the effective pile width then dividing that result by the soldier pile spacing.

f = Arching Factor

$$f = \frac{\text{(Passive Arching Capability) (Effective Pile Width)}}{\text{Soldier Pile Spacing}}$$

Where: $f \leq 1.0$

The value f must be equal to or less than 1.0 to prevent overlap of the passive resisting lengths.

Assume the same values previously cited where the soldier pile spacing is 8'- 0", the pile is not encased in hard rock concrete, the pile flange width is 14", and the internal friction angle ϕ of the cohesionless soil is 38^{0}.

$$f = \frac{[0.08(38)](14/12)}{8.0} = 0.438$$

A sheet pile analysis could then be made for the soldier piling as long as all equations used below the excavation line are factored by 0.438, and the final answers multiplied by 8 which equals the pile spacing of 8'- O".

AASHTO Methodology:

The 1992 publication, Standard Specifications for Highway Bridges by AASHTO contains a simplified method for designing cantilever soldier piling in cohesionless soils. The methodology along with a sample problem is included near the end of this chapter. The AASHTO method permits the inclusion of surcharges. This design method requires that no passive resistance be counted within 1.5 times the effective pile width below the depth of excavation, The method also provides that the computed pile depth (D) be increased by 30% for temporary work.

The AASHTO method indicates that the adjusted pile width may be up to 3 times the effective pile width provided that the soldier pile spacing is equal to or greater than 5 times the effective pile width. Structures policy will be to use an adjusted pile width of 0.08ϕ (≤ 3.00) times the effective pile width provided this width does not exceed the soldier pile spacing.

GUIDELINES FOR REVIEW OF SOLDIER PILE PASSIVE ARCHING CAPABILITIES

GRANULAR SOILS

COMPACTNESS	VERY LOOSE		LOOSE		MEDIUM		DENSE		VERY DENSE
Relative Density, D_d		15%		35%		65%		85%	
Standard Penetration Resistance, N = Blows/ft		4		10		30		50	
Angle of Internal Friction, ϕ		28		30		36		41	
Unit Weight (PCF)									
Moist	100		95-125		110-130		110-140		130+
Submerged	60		55-65		60-70		65-85		75+
Arching Capability	0.08ϕ		0.08ϕ		0.08ϕ		0.08ϕ		0.08ϕ

COHESIVE SOILS

CONSISTENCY	VERY SOFT		SOFT		MEDIUM		STIFF		VERY STIFF		HARD
q_u = unconfined comp. strength (PSF)		500		1000		2000		4000		8000	
Standard Penetration Resistance, N = Blows/Ft		2		4		8		16		32	
Unit Weight (PCF) Saturated		100-120				110-130			120-140		130+
Arching Capability		1 to 2		1 to 2		2		2		2	

TABLE 10 - 1 (TABLE 21)

LAGGING

Wood Jagging is commonly installed in front of, or behind the front flange of wide flange beam soldier piles. The procedure of installing lagging behind the back flange of the soldier piling is not recommended because the potential arching action of the supported soil will be destroyed. Lagging placed behind the front flange may be wedged back to provide tight soil, to lagging contact. Voids behind lagging should be filled with compacted material. Lagging may be installed with a maximum spacing up to 1 l/2" to permit seepage of moisture through the wall system. Movement of soil through the lagging spaces an be prevented by packing straw or hay in the spaces.

Construction grade lumber is the most common material used for lagging. Treated lumber is used when it is expected that the lagging will remain in place for a longer period of time or permanently.

Soil arching behind lagging is induced by lateral soil movement within the failure wedge. This soil movement causes the lagging to flex outward. The arching process induces a redistribution of soil pressure away from the center of the lagging toward the much stiffer soldier pile support. Because of this, the design load on the lagging may be taken as 0.6 times the theoretical or calculated pressure based on a simple span. Studies have shown that a maximum lagging pressure of 400 psf should be expected when surcharges are not affecting the system. Without soil arching, the pressure redistribution would not occur and reduced lagging loads should not be considered. For the arching effect to occur the back side of the soldier pile must bear against the soil.

- **Lagging design load = 0.6(shoring design load)**
- **Maximum lagging load may be 400 psf without surcharges**

Table 10-2 lists FHWA recommended minimum timber thickness for construction grade douglas fir lagging for a variety of soil classifications.

- <u>Competent Soils:</u> **These soils include high internal friction angle sand or granular material or stiff to very stiff clays.**
- <u>Difficult Soils:</u> **These soils consist of loose or low friction angle cohesionless material, silty sands, and over consolidated clays which may expand laterally, especially in deep excavations.**
- <u>Potentially Dangerous Soils:</u> **The use of lagging with potentially dangerous soils is questionable.**

The tabular values may be used for lagging where soil arching behind the lagging can develop. Tabular values should not be used for excavations adjacent to existing facilities including railroads. Lagging used in conjunction with surcharges should be analyzed separately.

RECOMMENDED THICKNESS OF WOOD LAGGING

WHEN SOIL ARCHING WILL BE DEVELOPED

(FOR LOCATIONS WITHOUT SURCHARGE LOADINGS)

Soil Description Classification	Unified	Depth	Recommended Thickness of Lagging (rough cut) for Clear Spans of: 5'	6'	7'	8'	9'	10'
COMPETENT SOILS								
Silts or fine sand and silt above water table	ML, SM - ML							
Sands and gravels (medium dense to dense).	GW, GP, GM, GC, SW, SP, SM	0' to 25'	2"	3"	3"	3"	4"	4"
Clays (stiff to very stiff); non-fissured	CL, CH	25' to 60'	3"	3"	3"	4"	4"	5"
Clays, medium consistency and $\gamma H/C < 5$.	CL, CH							
DIFFICULT SOILS								
Sands and silty sands, (loose).	SW, SP, SM							
Clayey sands (medium dense to dense) below water table.	SC	0' to 25'	3"	3"	3"	4"	4"	5"
Clays, heavily over-consolidated fissured.	CL, CH	25' to 60'	3"	3"	4"	4"	5"	5"
Cohesionless silt or fine sand and silt below water table	ML; SM - ML							
POTENTIALLY DANGEROUS SOILS (appropriateness of lagging is questionable)								
Soft clays $\gamma H/C > 5$.	CL, CH	0' to 15'	3"	3"	4"	5"	-	-
Slightly plastic silts below water table.	ML	15' to 25'	3"	4"	5"	6"	-	-
Clayey sands (loose), below water table.	SC	25' to 35'	4"	5"	6"	-	-	-

*Adapted and revised from the April 1976 Federal Highway Administration Report No. FHWA-RD-75-130.

TABLE 10- 2 (TABLE 22)

CANTILEVER SOLDIER PILES - GRANULAR SOIL

Basic soil - no surcharge

f = Arching factor
(See page 10-3)

$P_{A1} = \gamma H K_a = KwH$

$P_{A'1} = fP_{A1}$

$P_{A2} = f\gamma D K_a + P_{A'1}$

$P_E = f\gamma D(K_p - K_a) - P_{A'1}$

$P_J = f\gamma D(K_p - K_a) + f\gamma H K_p$

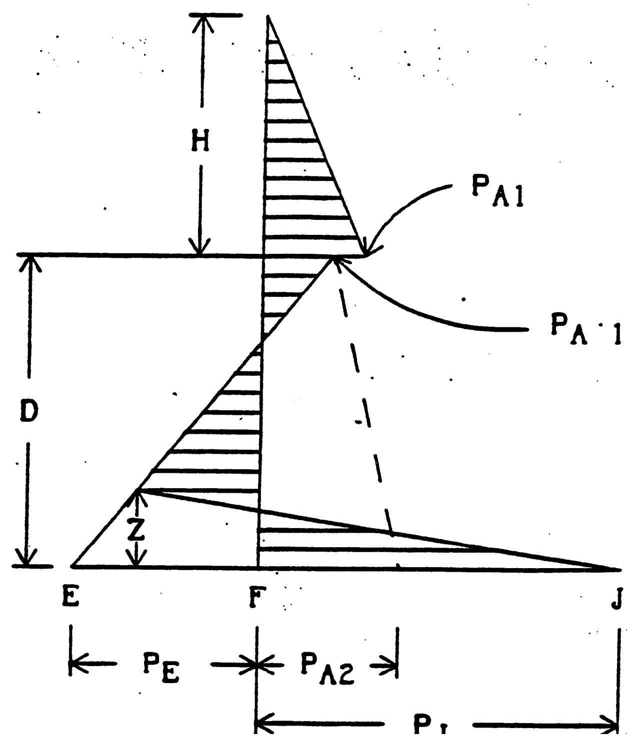


FIGURE 10-1

$$\Sigma F_H = 0 = (H)(P_{A1})/2 + (P_{A'1} + P_{A2})(D)/2 + (P_E + P_J)(Z)/2 - (P_E + P_{A2})(D)/2$$

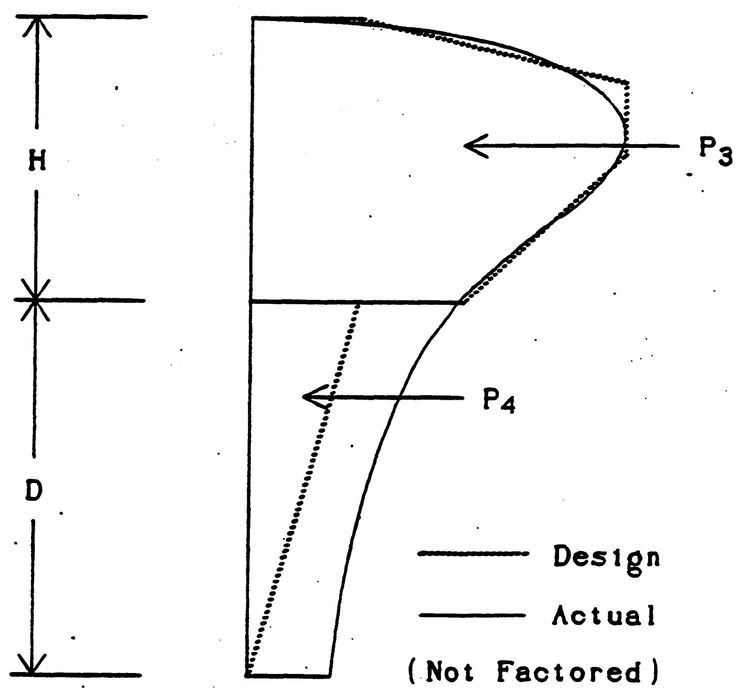
$$\therefore Z = \{(P_E + P_{A2})(D) - (H)(P_{A1}) - (P_{A'1} + P_{A2})(D)\}/(P_E + P_J)$$
$$= \{P_E - P_{A'1})(D) - (H)(P_{A1})\}/(P_E + P_J)$$

$$\Sigma M_F = 0 = \{(H)(P_{A1})/2\}[H/3 + D] + (P_{A'1}(D)[D/2\} + \{(P_{A2} - P_{A'1})(D)/2\}[D/3\} + \{(P_E + P_J)(Z)/2\}[Z/3] - \{(P_E + P_{A2})(D)/2\}[D/3]$$

<u>Surcharge Considerations:</u>

In most real situations there will be some sort of surcharge present. The following pages demonstrate how two types of surcharges may be handled. Simplifying the pressure diagrams (using sound engineering judgement) should not alter the results significantly and may make the problem much easier to solve. The surcharge pressures can be added directly to the soil pressure diagram, or may be sketched separately. It will often be convenient to convert a long uniform surcharge to an equivalent height of soil, increase H by that amount, and then analyze the shoring with one pressure diagram.

For Boussinesq surcharges, the pressure diagram above the excavation depth is simplified so as to match the area as close as possible, while still allowing for ease of computation. The surcharge pressure immediately below the depth of excavation is adjusted by the Passive Arching Capability factor and then may be tapered to zero (for small surcharge) at the bottom end of the soldier pile.



BOUSSINESQ SURCHARGE

FIGURE 10 - 2

P_3 = Area under dashed line above the excavation depth.

P_4 = Area under dashed line below the excavation depth.

The forces and moments are then added to the equations on the previous page to solve for the total horizontal forces and moments and to arrive at the required depth (D).

To allow for a Safety Factor and sufficient embedment increase D by 20% - 40% or initially adjust K_p by using $K_P/1.5$ to $K_P/1.75$.

SAMPLE PROBLEM 10-1: CANTILEVER SOLDIER PILE

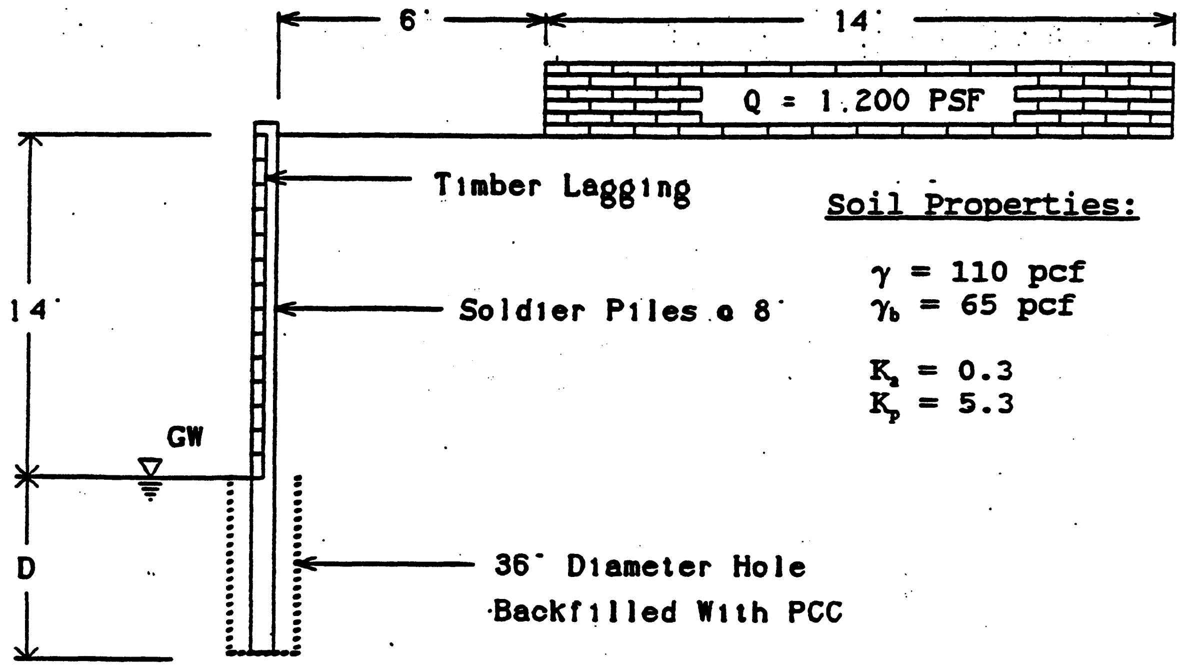


FIGURE 10 - 3

Determine Lateral Pressures: Soil parameters are arbitrary values chosen for simplicity

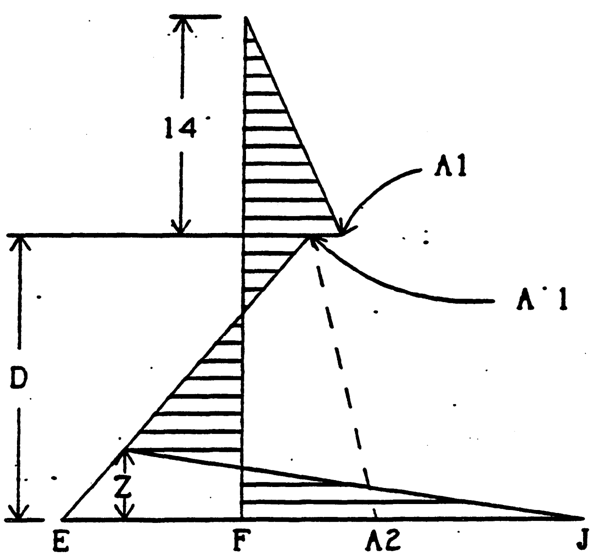


FIGURE 10 - 4

Soil pressure only.

Arching capability = 2

$f = (2)(36/12)/8 = 0.75$

$P_{A1} = \gamma H K_a = (110)(14)(0.3)$
$= 462$ psf

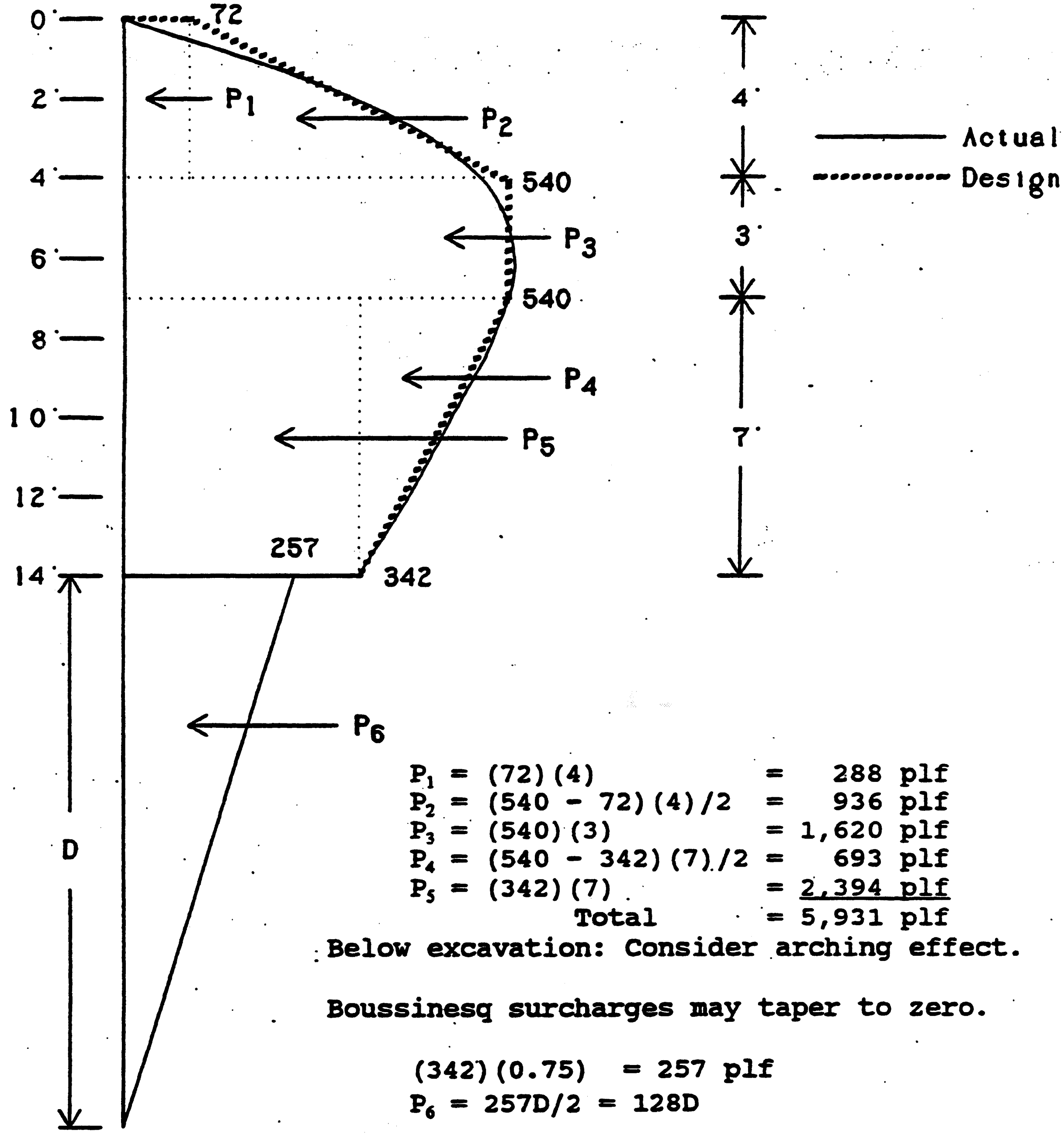
$P_{A'1} = fP_{A1} = (0.75)(462)$
$= 347$ psf

$P_{A2} = f\gamma_b D K_a + P_{A'1}$
$= (0.75)(65)(0.3)D + 347$
$= 15D + 347$

$P_E = f\gamma_b D(K_p - K_a) - P_{A'1} = (0.75)(65)(5.3 - 0.3)D - 347$
$= 244D - 347$

$P_J = f\gamma_b D(K_p - K_a) + f\gamma H K_p$
$= (0.75)(65)(D)(5.0) + (0.75)(110)(14)(5.3) = 244D + 6,122$

Surcharge Pressure:



$P_1 = (72)(4) = 288$ plf
$P_2 = (540 - 72)(4)/2 = 936$ plf
$P_3 = (540)(3) = 1,620$ plf
$P_4 = (540 - 342)(7)/2 = 693$ plf
$P_5 = (342)(7) = \underline{2,394}$ plf
Total $= 5,931$ plf

Below excavation: Consider arching effect.

Boussinesq surcharges may taper to zero.

$(342)(0.75) = 257$ plf
$P_6 = 257D/2 = 128D$

FIGURE 10 - 5

Determine D:

$\Sigma F_H = 0$

$$\begin{aligned} &= (14)(462)/2 + (347 + 15D + 347)(D)/2 \\ &\quad + (244D - 347 + 244D + 6,122)(Z)/2 + 5,931 + 128D \\ &\quad -(244D - 347 + 15D + 347)(D)/2 \\ &= -122D^2 + 475D + 244DZ + 2,888Z + 9,165 \end{aligned}$$

$\therefore Z = (D^2 - 3.9D - 75.1)/(2D + 23.7)$

$$
\begin{aligned}
\Sigma M_F &= 0 \\
&= \{(14)(462)/2\}[D + 14/3] + 347D[D/2] + \{(15D)(D)/2\}[D/3] \\
&\quad + \{(244D - 347 + 244D + 6{,}122)(Z)/2\}[Z/3] + 288[D + 12] \\
&\quad + 936[D + 11.33] + 1{,}620[D + 8.5] + 693[D + 4.67] \\
&\quad + 2{,}394[D + 3.5] + 128D[2D/3] \\
&\quad - \{(244D - 347 + 15D + 347)(D)/2\}[D/3] \\
&= -41D^3 + 259D^2 + 9{,}165D + 81DZ^2 + 963Z^2 + 54{,}538 = 0
\end{aligned}
$$

$$\therefore Z^2 = (D^3 - 6.3D^2 - 223.5D - 1{,}330.2)/(2.0D + 23.5)$$

By trial and error or other means D = 22.30′ & Z = 4.91′
Use a safety factor of 30%: Use D = 1.3(22.3) = 29.0′

Find Maximum Moment:

(Composite section properties ignored)

Locate plane of zero shear (B).

$y = P_{A'1} / f\gamma_b(K_p - K_a)$

$= 347/(0.75)(65)(5.3 - 0.3) = 1.42'$

Surcharge pressure at A:

257(22.30 - 1.42)/22.30 = 241 psf

Shear due to surcharge at A:

1.42(257 + 241)/2 + 5,931 = 6,285

Total shear at A:

14(462)/2 + 347(1.42)/2 + 6,285
= 9,765 Lb/LF

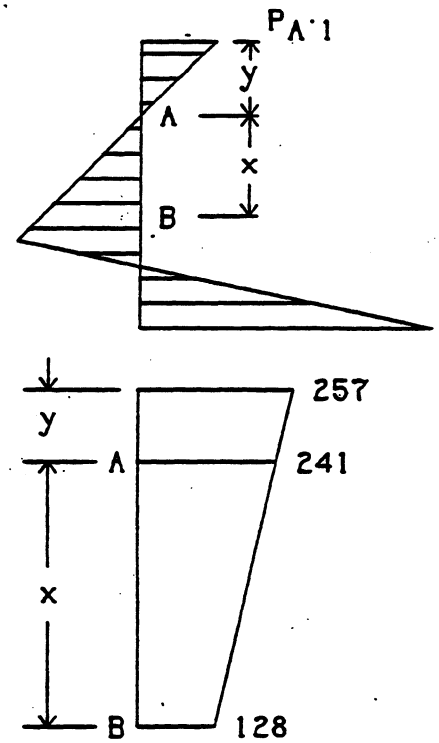


FIGURE 10 - 6

Shear for area between A & B = 9,765

$\{f\gamma_b(K_p - K_a)x^2/2\} - (\{241 + (257)(22.30 - 1.42 - x)/22.30\}/2)x$
$= 9{,}765$

Substituting: $0.75(65)(5.3 - 0.3)x^2 - 241x + 5.76x^2 = 9{,}765$

Simplifying: $127.6x^2 - 241x - 9{,}765 = 0$

Solving for x: x = 9.74′

Find Moment at B:

M due to soil pressure above A.

{14(462)/2}[15.83] + {347(1.42)/2}[10.69] = 53,828 Ft-Lb/LF

M due to soil pressure between A & B:

0.75(65)(5.3 - 0.3)(9.74)(9.74/2)[9.74/3] = 37,538 Ft-Lb/LF

M due to surcharge above excavation:

288[23.16] + 936[22.49] + 1,620[19.66] + 693[15.83] + 2,394[14.66] = 105,636 Ft-Lb/LF

Surcharge pressure at B:

257(22.30 - 1.42 - 9.74)/22.30 = 128 psf

M due to surcharge below excavation:

128(11.16)[11.16/2] + {(257 - 128)(11.16)/2}[11.16)(2)/3] = 13,326 Ft-Lb/LF

M (Total) = 8(53,828 - 37,538 + 105,636 + 13,326)
= 1,082,016 Ft-Lb

S Required = (1,082,016)(12)/22,000 = 590.2 in^3

Use W30 x 191, S = 598 in^3

Determine Lagging Needed:

By inspection, maximum load occurs at the depth of excavation.
$M_{max} = wL^2/8 = (342 \quad 462)(8)^2/8 = 6,432$ Ft-Lb
(With total soil arching $M_{max} = 400(8)^2/8 = 3,200$ Ft-Lb)

S Required = 6,432(12)(0.6)/(1,500)(1.0)* = 30.9 in^3
* 1.0 In lieu of 1.33 duration factor due to high risk building

Use 4 x 12's (rough lumber): S = 32 in^3

This answer does not agree with values in Table 22 because the table does not provide for surcharge loadings.

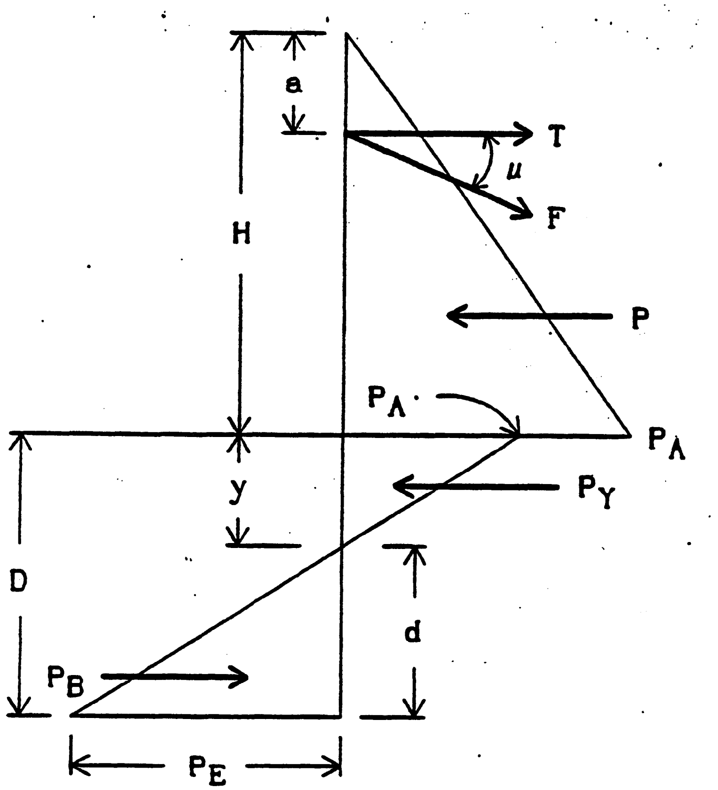
Summary:

Use W30 X 191's - minimum length of 42' 8", placed in 36" diameter holes and backfilled to bottom of excavation with concrete.

Use 4 x 12's (Rough lumber) for lagging.

The size of the wide flange beam and the diameter of the drilled hole indicate that cantilevered soldier piles would not be the correct type of shoring for the conditions given.

SOLDIER PILES W/SINGLE TIEBACK - GRANULAR SOIL



Soil only (No surcharge)
$a \approx 0.2H$ to $0.4H$

S = Soldier pile spacing

f = Arching factor

$P_A = \gamma H K_a = KwH$

$P = HP_A/2 = \gamma H^2 K_a/2$

$= KwH^2/2$

$P_{A'} = fP_A$

$y = P_{A'}/f\gamma(K_p - K_a)$

$P_Y = yP_{A'}/2$

$P_E = f\gamma d(K_p - K_a)$

$P_B = dP_E/2$

FIGURE 10 - 7

Determine D:

$\Sigma M_T = 0 = P[2H/3 - a] + P_Y[H-a + y/3] - P_B[H - a + y + 2d/3]$

Solve for d by trial and error (or other means). For the first approximation try $d = H/4.5$.

$D = y + d$. Adjust the value of D or K_p for a safety factor.

Determine T: $T = S(P + P_Y - P_B)$ and $F = T/\cos\mu$

Find the maximum moment. This will generally be either the cantilever section above T or somewhere between T and the excavation level. Determine this second point by locating the point of zero shear.

Determine soldier pile section modulus required, size of lagging needed, and tieback requirements.

Note: A surcharge will normally be present (or the minimum surcharge or 72 psf will be used) and should be added to the equations shown above.

SAMPLE PROBLEM 10-2: SOLDIER PILE WITH SINGLE TIEBACK

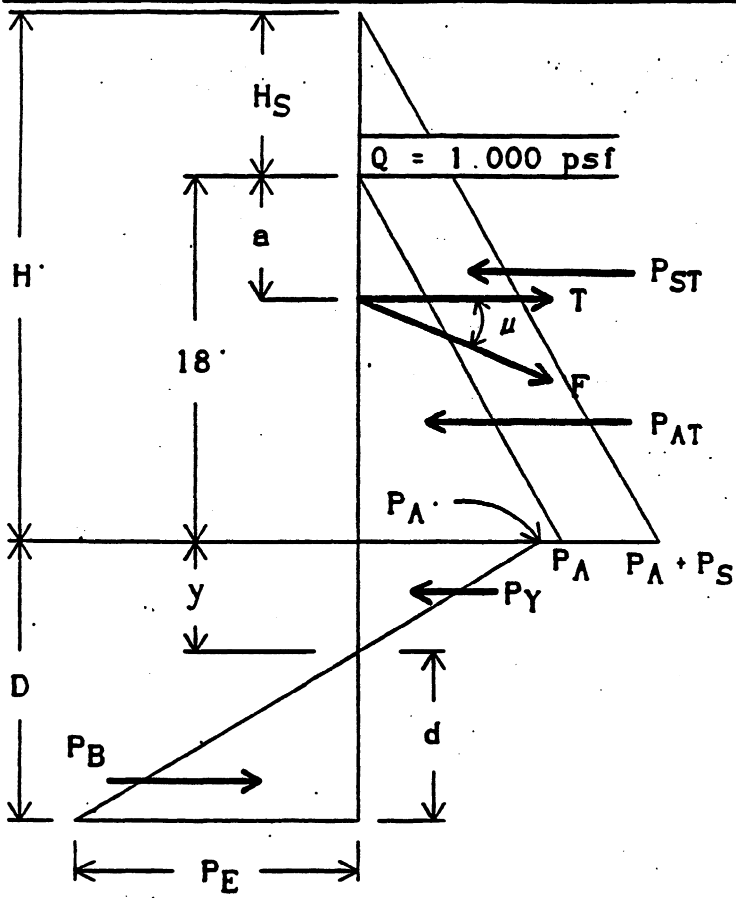


FIGURE 10 - 8

Given:

$Kw = 35$ pcf
$\gamma = 115$ pcf $\mu = 20°$

$K_a = Kw/\gamma = 0.30$
$K_p = \gamma/Kw = 3.29$

$H = 18'$ $a = 6'$

Solution:

Q = Uniform surcharge
$H_S = 1,000/115 = 8.7'$
$H' = 18 + 8.7 = 26.7'$

Pile spacing (S) = 6'
Pile Hole Diameter = 24"
Arching capability = 2.0
$f = (2)(24/12)/6 = 0.67$

$P_A = KwH = 35(18) = 630$

$P_A + P_S = KwH' = 35(26.7) = 935$ psf

$P_S = 935 - 630 = 305$ psf

$P_{AT} = 630(18/2) = 5670$ Lb/LF

$P_{ST} = 305(H) = 305(18) = 5490$ Lb/LF

$P_{A'} = fP_A = 0.67(630) = 422$ psf

$y = P_{A'}/f\gamma(K_p - K_a) = 422/[(0.67)(115)(3.29 - 0.30)] = 1.83$

$P_Y = yP_{A'}/2 = 1.83(422)/2 = 386$ Lb/LF

$P_E = f\gamma(d)(K_p - K_a) = 0.67(115)(d)(3.29 - 0.30) = 230d$

$P_B = dP_E/2 = d(230d)/2 = 115d^2$

Determine D:

$$\Sigma M_T = 0 = P_{AT}[2H/3 - a] + P_y[H - a + y/3] + P_{ST}[H/2] - P_B[H - a + y + 2d/3]$$

$$= 5,670[2(18.0)/3 - 6] + 386[18 - 6 + 1.83/3] + 5490[18/2 - 6] - (115d^2)[18 - 6 + 1.83 + 2d/3]$$
$$= 55,357.5 - 1590.5d^2 - 76.7d^3 = d^3 + 20.7d^2 - 721.7$$

By trial and error, or by other means, d = 5.27'

$D = d + y = 5.27 + 1.83 = 7.10'$
Increase D by 30% for safety factor: $D = 1.3(7.1) = 9.2'$

Determine T:

$P_B = 115(5.27)^2 = 3{,}194$ Lb/LF

$T = P_{AT} + P_{ST} + P_y - P_B = 5{,}670 + 5{,}490 + 386 - 3{,}194 = 8{,}352$ Lb/LF
$F = T/\cos\mu = 8{,}352/\cos 20° = 8{,}888$ Lb/LF
Total $F = 6(8{,}888) = 53{,}328$

Find Maximum Moment (See Note)

Locate point of zero shear (x).

$8{,}352 = P_A(x/18)(x/2) + P_S x$

$8{,}352 = 17.5x^2 + 305x$

$x = 14.8'$

$M = -(630)(14.8/18)(14.8/2)[14.8/3]$

$- (305)(14.8)[14.8/2]$

$+ (8{,}352)[14.8 - 6.0]$

$= 21{,}184$ Ft Lb/LF

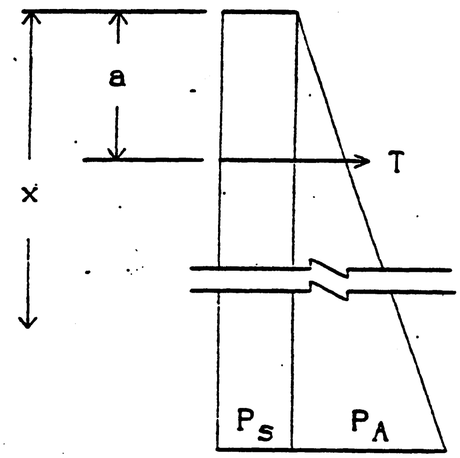


FIGURE 10 - 9

Check Cantilever:

$M = \{(630)(6.0/18)(6.0)/2\}[6.0/3] = 6{,}750$ Ft-Lb/LF
This does not control

S Required $= 21{,}184(6)(12)/22{,}000 = 69.3\ in^3$

Use W14 x 53, $S = 77.8\ in^3$

Determine lagging and tieback requirements.

See alternate analysis using AISC specifications at end of Chapter.

Note: When the soldier pile is encased in 4 sack or better concrete, the buried portion of the pile acts as a composite section which will have a large section modulus. When this is the case the moment at the excavation line may often be controlling.

SAMPLE PROBLEM 10-3: SOLDIER PILE WITH SINGLE TIEBACK

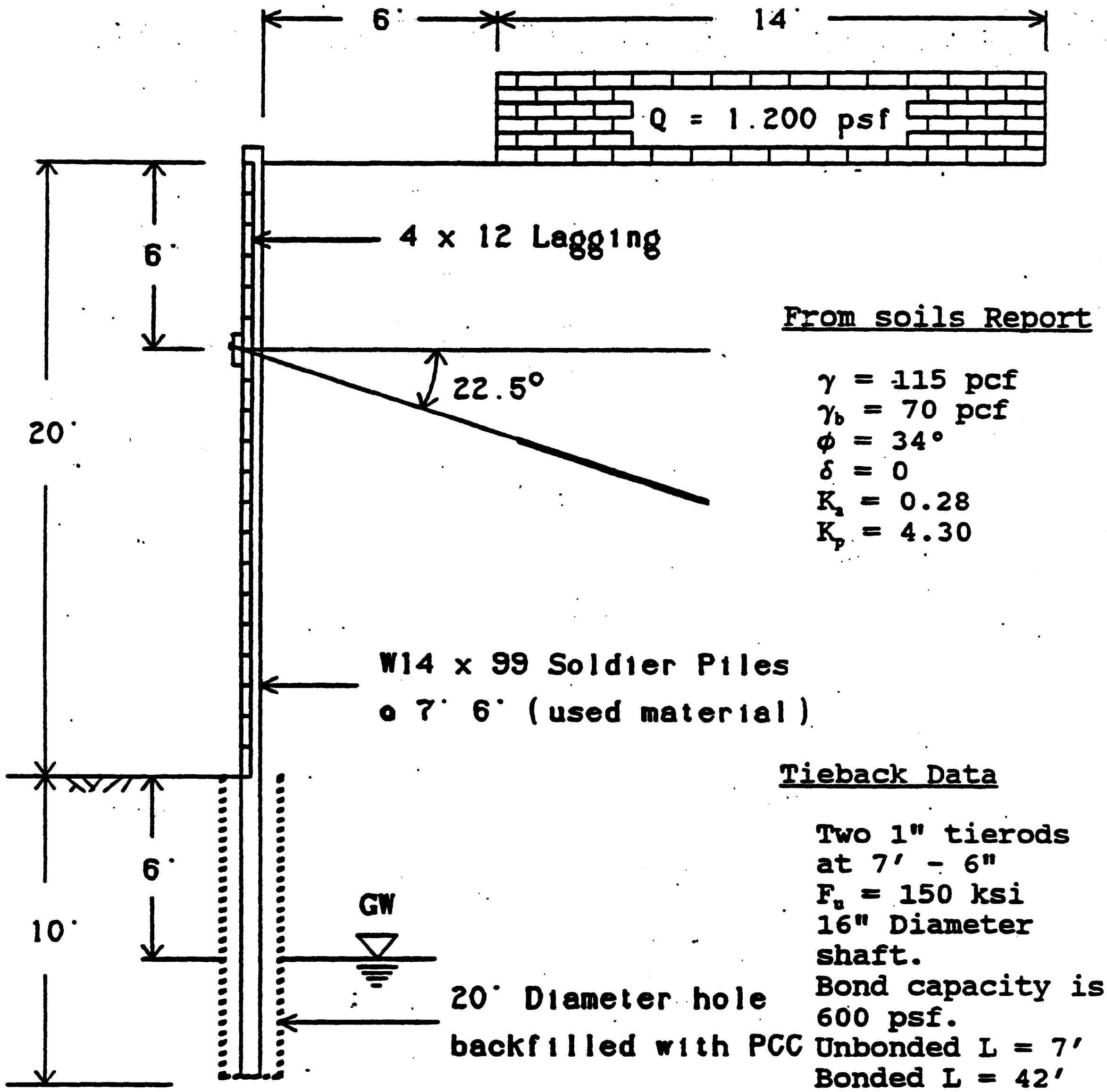


FIGURE 10 - 10

Determine Lateral Pressures:

FHWA recommends adjusting K_p by division with 1.5 in lieu of increasing D by 20% - 40%. (This is conservative)

Adjust K_p by 1.5. For this example K_p = 4.3/1.5 = 2.87

$K_p - K_a$ = 2.87 - 0.28 = 2.59

Arching capability = 0.08(34°) = 2.7 Use 1.5 due to wet condition (Need to be conservative next to building).
f = 1.5(20/12)/7.5 = 0.33

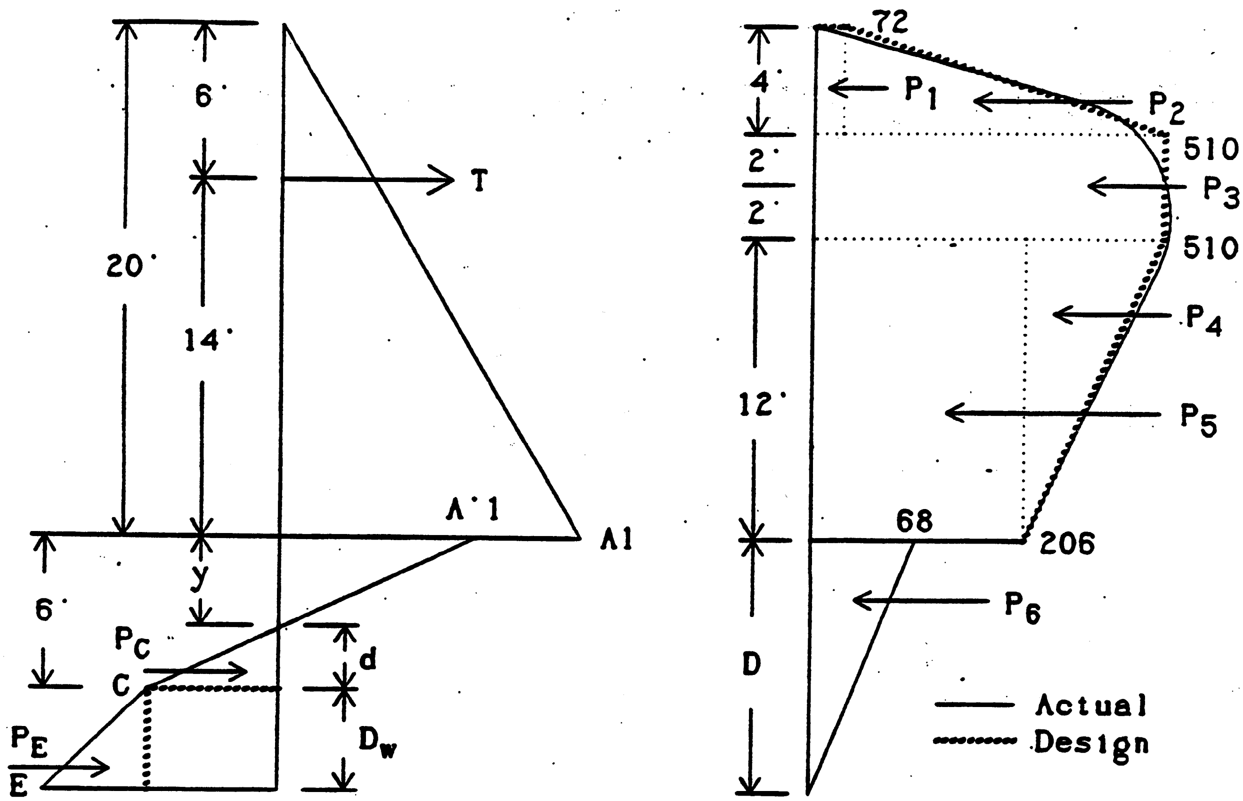


FIGURE 10 - 11

$P_{A1} = \gamma H K_a = 115(20)(0.28) = 644$ psf

$P_{A'1} = fP_{A1} = 0.33(644) = 213$ psf

$y = P_{A'1}/f\gamma(K_p - K_a) = 213/[(0.33)(115)(2.59)] = 2.17'$

$d = 6 - 2.17 = 3.83'$

$P_C = f\gamma d(K_p - K_a) = 0.33(115)(3.83)(2.59) = 376$ psf

$P_E = P_C + f\gamma_b D_w(K_p - K_a) = 376 + 0.33(70)(2.59)D_w = 376 + 60D_w$

$P_1 = 72(4) = 288$ psf
$P_2 = (510 - 72)(4)/2 = 876$ psf
$P_3 = 510(4) = 2,040$ psf
$P_4 = (510 - 206)(12)/2 = 1,824$ psf
$P_5 = 206(12) = 2,472$ psf

Below excavation: Consider arching effect. Boussinesq surcharges may be tapered to zero at the calculated depth D.

$206(0.33) = 68$ psf
$P_6 = 68(6 + D_w)/2 = 204 + 34D_w$

Determine D:

$\Sigma m_T = 0$

Moment due to surcharge:

$$(204 + 34D_w)[14 + (6 + D_w)/3] + 2{,}472[8] + 1{,}824[6] + 2{,}040[0] - 876[3.33] - 288[4]$$
$$= 11D_w^2 + 612D_w + 29{,}915$$

Moment due to soil:

$$\{644(20)/2\}[7.33] + \{213(2.17)/2\}[14.72] - \{376(3.83)/2\}[18.72] - 376(D_w)[20 + D_w/2] - \{60D_w)(D_w)/2\}[20 + (2/3)D_w]$$
$$= -20D_w^3 - 788D_w^2 - 7{,}520D_w + 37{,}128$$

Combined moment:

$$20D_w^3 + 777D_w^2 + 6{,}908D_w - 67{,}043 = 0$$
$$\text{or } D_w^3 + 39D_w^2 + 345D_w - 3{,}352 = 0$$

From which $D_w = 5.62'$

$D = D_w + 6 = 5.62 + 6 = 11.62'$ (Use D = 11'-8")

Determine T:

$$\Sigma F_H = 0$$
$$= 288 + 876 + 2{,}040 + 1{,}824 + 2{,}472 + \{204 + 34(5.62)\} + 644(20)/2 + 213(2.17)/2 - 376(3.83)/2 - 376(5.62) - 60(5.62)(5.62/2) - T$$

$\therefore T = 10{,}785$ Lb/LF

Total $T = 10{,}785(7.5) = 80{,}888$ Lb

Find Maximum Moment:

Check cantilever moment at T:

$$M = \{644(6/20)(6/2)\}[6/3] + 288[2 + 4/2] + 876[2 + 4/3] + 510(2)[2/2]$$
$$= 6{,}251 \text{ Ft-Lb/LF}$$

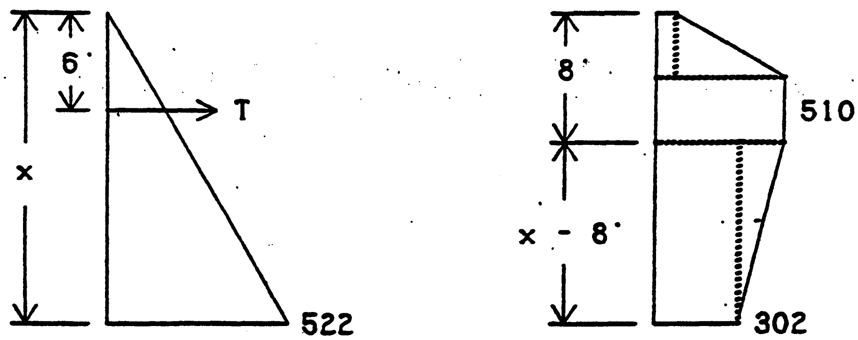


FIGURE 10 - 12

Locate point of zero shear:

$$10{,}785 - 288 - 876 - 2{,}040 - 644(x/20)(x/2)$$
$$- \{x - 8\}\{510 + [206 + (510 - 206)(20 - x)/12]\}/2 = 0$$

$$x^2 + 207.6x - 3{,}632.5 = 0$$
$$\therefore x = 16.23'$$

Pressure at point of zero shear due to soil:

$$644(16.2/20) = 522 \text{ psf}$$

Pressure at point of zero shear due to surcharge:

$$206 + (510 - 206)(20 - 16.2)/12 = 302 \text{ psf}$$

Moment due to tieback and soil:

$$10{,}785[16.2 - 6] - \{(522)(16.2/2)\}[16.2/3] = 87{,}175 \text{ Ft-Lb/LF}$$

Moment due to surcharge:

$$M = - 288[16.2 - 4/2] - 876[16.2 - 4(2/3)]$$
$$- 2{,}040[16.2 - 4 - 4/2] - 302(16.2 - 8)[(16.2 - 8)/2]$$
$$- \{(510 - 302)(16.2 - 8)/2\}[(16.2 - 8)(2/3)]$$
$$= - 51{,}568 \text{ Ft-Lb/LF}$$

Combined moment (assuming non-compact section):

87,175 - 51,568 = 35,607 Ft-Lb/LF   This controls.

S Required = 35,607(7.5)(12)/22,000 = 145.67 in^3

S furnished = 157 in^3 > 145.67 in^3 ∴ O.K.

See alternate analysis using AISC specifications on page 10-21.

Check Lagging:

Consider arching effect on lagging. Multiply all pressure results by 0.6. By inspection, maximum moment occurs at the depth of excavation.

$M_{max} = wL^2/8 = (644 + 206)(7.5)^2/8 = 5,977$ Ft-Lb

S Required $= 5,977(12))(0.6)/1,500(1.0)^* = 28.7$ in^3
*** Load duration factor due to high risk building.**

Use 4 x 12's (Rough lumber). $S = 32$ in^3

Check Shear:

$V = (7.5/2 - 0.33)(850)(0.6) = 1,744$ Lb
$v = 3V/2A = 3(1,744)/2(4)(12) = 54.5$ psi < 140 psi $\therefore$ O.K.

Tiebacks:

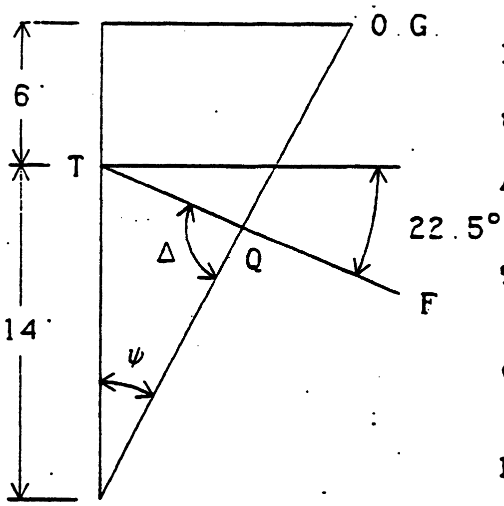


FIGURE 10 - 13

$F = 80,888/\cos 22.5° = 87,553$ Lb

$\psi \approx 45° - \phi/2 \approx 45° - 34°/2 \approx 28°$

$\Delta = 180° - (90° - 22.5°) - \psi = 84.5°$

TQ = Unbonded length
$= (14 \sin 28°)/\sin \Delta = 6.6'$

6.6' < 7.0' provided $\therefore$ O.K.

Bond capacity given as 600 psf

Bonded frictional resistance (per foot) $= \pi(16/12)(600)(L)$
$= 2,513(L)$ Lb/LF

Length needed = 87,553/2,513 = 34.8' < 42' $\therefore$ O.K.

Safety Factor = 42/34.8 = 1.21 = (21%)

Note: Vertical downward component of tie may be used in conjunction with wedge weights in stability analysis to counteract slip circle failure.

Vertical component $= 87,553 \sin 22.5° = 33,505$ Lb

SOLDIER PILE WITH SINGLE TIEBACK

Soldier piles in the two previous problems were not checked for compressive stress or for the combined stresses due to the vertical component of the tieback force. AISC criteria may be used to check combined stresses. For Sample Problems 15 and 16 assume maximum unbraced length is the cantilever or the length between the tie and the point where the passive soil resistance becomes effective.

SAMPLE PROBLEM 10 - 2 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 15.6\ in^2$ $r_x = 5.89$ in

Assume k = 1, then $KL/r = 15.8(12)/5.89 = 32.19$

From AISC: $M_r = 142.6$ Ft-k and $F_a = 19.79$ ksi

Downward tie force $= F \sin \omega = 53.3 \sin 20° = 18$ k

$M_{max} = 6(21.18) = 127.1$ Ft-k

$f_a/F_a + f_b/F_b = f_a/F_a + M_{max}/M_r \leq 1.0$

$= (18/15.6)/19.79 + 127.1/142.6$

$= 0.06 + 0.89 = 0.95 \leq 1.0$ OK

SAMPLE PROBLEM 10 - 3 - SOLDIER PILE: (ALTERNATE ANALYSIS)

Pile properties: $A = 29.1\ in^2$ $r_x = 6.14$ in

Assume k = 1; then $KL/r = 16.2(12)/6.14 = 31.7$

From AISC: $M_r = 288$ Ft-k and $F_a = 19.82$ ksi

Downward tie force $= F \sin \omega = 87.6 \sin 22.5° = 33.5$ k

$M_{max} = 7.5(35.6) = 267$ Ft-k

$f_a/F_a + f_b/F_b = f_a/F_a + M_{max}/M_r \leq 1.0$

$= (33.5/29.1)/19.82 + 267/288$

$= 0.06 + 0.93 = 0.99 \leq 1.0$ OK

SAMPLE PROBLEM 10-4: SOLDIER PILE WITH RAKER

Given:

From Soils Report:

γ = 115 pcf
ϕ = 35°
δ = 0

Allowable Soil Bearing = 2 ksf

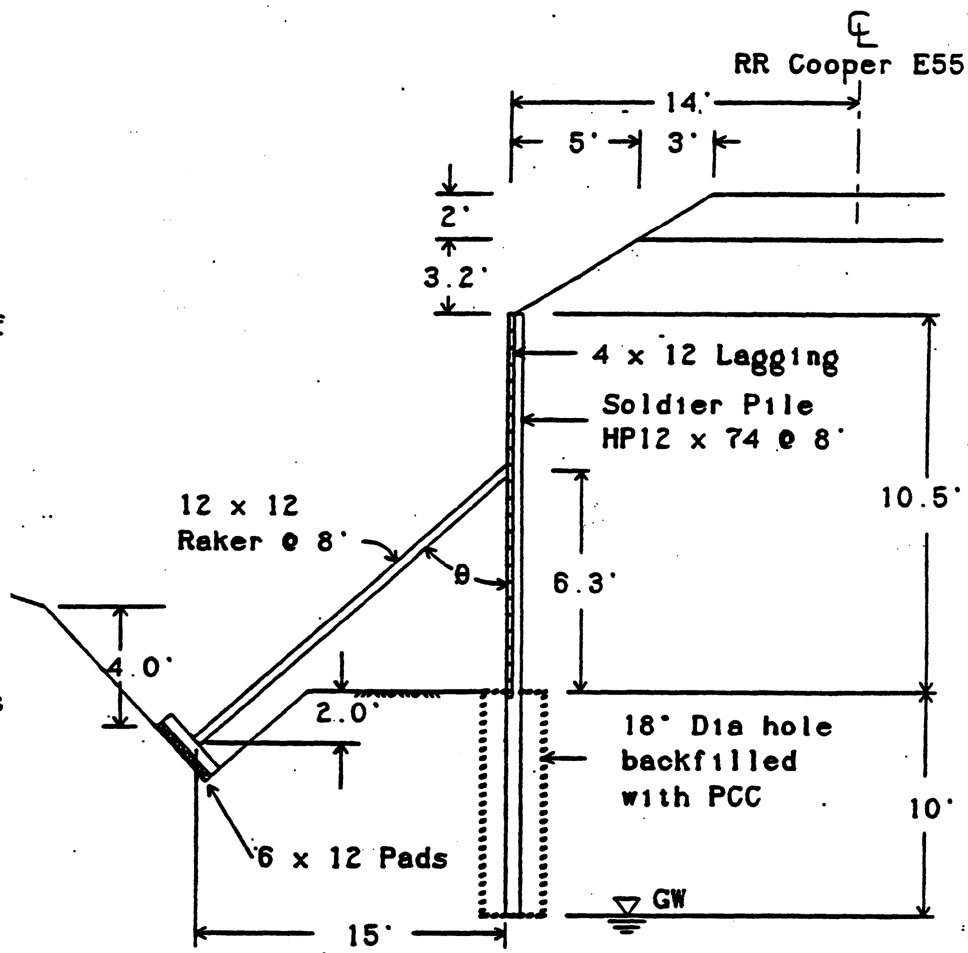
Loose Ballast:

γ = 115 pcf

Rough lumber is to be used

$\beta = \tan^{-1}(5.2/8)$
$= 33°$

$\beta/\phi = 33°/35°$
$= 0.94$



Solution:

FIGURE 10-14

NOTE: Assume that the back slope and ballast are one unit since their densities are equal and their angles are practically the same l If this was not the case, or if $\beta/\phi >$ then Trial Wedge or similar analysis would have to be used to solve the problem.

From Figure 8, $K_a = 0.65$; $K_p = \tan^2(45° + 35°/2) = 3.69$

$P_A = K_a \gamma H = 0.65(115)(10.5) = 785$ psf

Adjust K_p per FHWA recommendation in lieu of increasing D 20%-40%.

Adjust K_p by 1.5: $K_p = 3.69/1.5 = 2.46$

$K_p - K_a = 2.46 - 0.65 = 1.8$

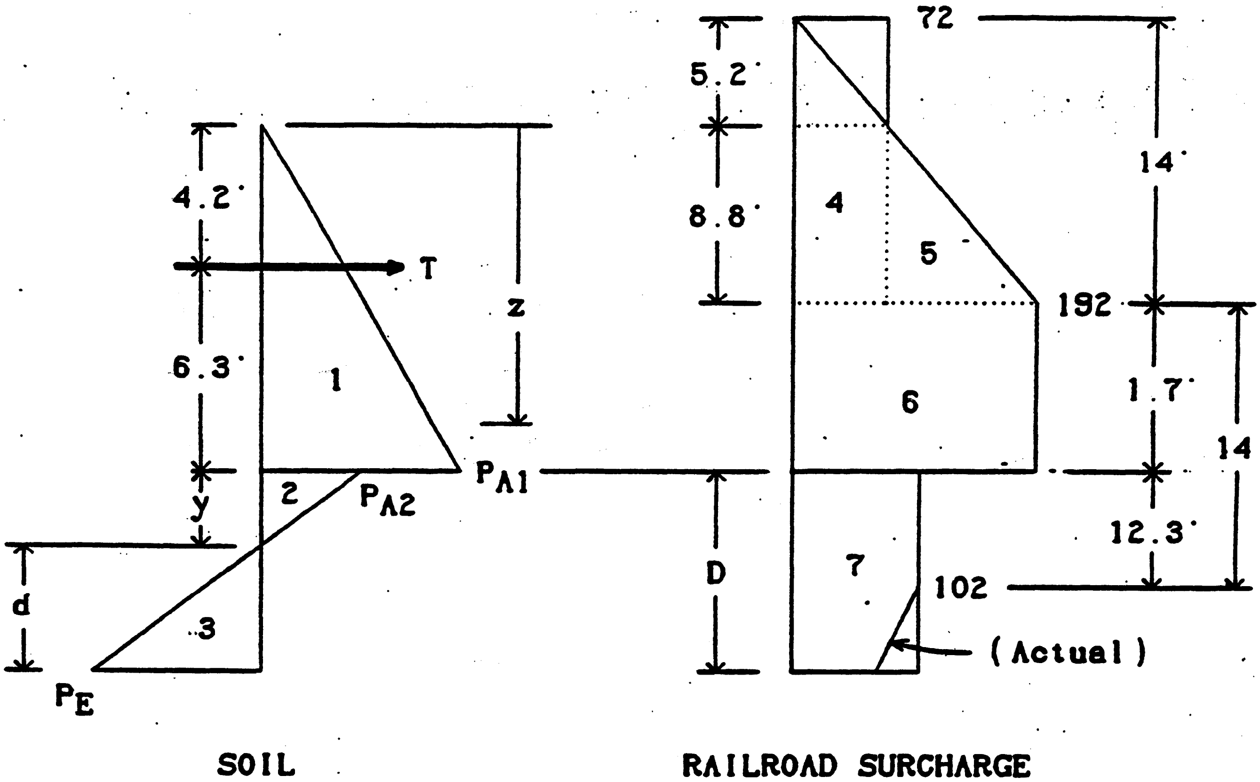


FIGURE 10 - 15

NOTE:- For ease of computation assume that the railroad surcharge has a rectangular shape below the excavation. The railroad sucharge is prorated from "CHART 3.6, LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD": 279(E55/E80) = 192 psf. Note that the top of the railroad surcharge diagram is always located at the elevation of the top of the rail.

Soil and surcharge act on adjusted width of soldier pile. Determine f and multiply a11 pressures below. the excavation line by this factor.

Passive arching capability = 0.08(35°) = 2.8
Diameter of drilled hole = 18/12 = 1.5'
Arching factor (f) = 2.8(1.5)/8 = 0.53

$P_{A2} = 0.53P_{A1} = 0.53(785) = 416$ psf
Surcharge at excavation line = 0.53(192) = 102 psf

$y = P_{A2}/f\gamma(K_p - K_a) = 416/[0.53(115)(1.8)] = 3.79'$
$P_E = f\gamma d(K_p - K_a) = 0.53(115)(1.8)d = 110d$

<u>Determine D:</u>

$D = d + y$

$\Sigma M_T = 0$

	AREA		ARM		MOMENT
1	785(10.5)/2	= 4,121	2/3(10.5) - 4.2	= 2.80	11,539
2	416(3.79)/2	= 788	6.3 + 3.79/3	= 7.56	5,957
4	72(8.8)	= 634	8.8/2 - 4.2	= 0.20	127
5	(192 -72)(8.8)/2	= 528	2/3(8.8) -4.2	= 1.66	876
6	192(1.7)	= 326	6.3 - 1.7/2	= 5.45	1,777
		6,397			20,276

3 AREA = $-110d(d/2) = -55d^2$
ARM = $6.3 + 3.79 + 2/3(d) = 10.09 + 0.67d$
MOMENT = $-554.95d^2 - 36.85d^3$

7 AREA = $102(3.79 + d) = 386.58 + 102d$
ARM = $6.3 + (3.79 + d)/2 = 8.20 + 0.5d$
MOMENT = $3{,}169.96 + 1{,}029.69d + 51d^2$

$\therefore 0 = 23{,}445.96 + 1{,}029.69d - 503.95d^2 - 36.85d^3$

Solve for d and D: $d = 6.37'$
$D = d + y = 6.37 + 3.79 = 10.16'$

Determine T:

$T = \Sigma$ Active areas $- \Sigma$ passive areas
$= 6{,}397 + 102(6.37 + 3.79) - (6.37)^2(110)/2 = 5{,}202$ Lb/LF

<u>Check Soldier Pile:</u> Find M_{max}

By inspection, the point of zero shear will be above the excavation.

$0 = 785z/10.5(z/2) + 634 + 528 + 192(z - 8.8) - 5{,}202$
$= 37.38z^2 - 4{,}040 + 192z - 1{,}689.60 = z^2 + 5.14z - 153.28$
$\therefore z = 10.08' \approx 10.1'$

$M_{max} = 37.38(10.1)^2[10.1/3] + 634[10.1 - 8.8/2]$
$+ 528[10.1 - 2/3(8.8)] + 192(10.1 - 8.8)[(10.1 - 8.8)/2]$
$- 5{,}202[10.1 - 4.2]$
$= 11{,}843$ Ft-Lb/LF

Total Moment = 8(11,843) = 94,744 Ft-Lb

S Required = M/F_b = 94,744(12)/22,0000 = 51.68 in^3
S Furnished = 93.8 in^3 > 51.68 in^3

Check Lagging:

Pressure is greatest at the bottom of the excavation:
785 + 192 = 977 psf
$M = wL^2/8 = 977(12)(8)^2/8 = 93{,}792$ In-Lb

S Required = 93,792(0.6)/(1,500)(1.0)* = 37.52 in^3
* No load duration factor used when adjacent to railroads.

S for a 4 x 12 = $12(4)^2/6 = 32\ in^2 < 37.52\ in^3$
S for a 6 x 12 = $12(6)^2/6 = 72\ in^3$ Use 6 x 12 lagging

$V = (8/2 - 0.33)(977)(0.6) = 2{,}151$ Lb

$f_v = 3V/2A = 3(2{,}151)/[2(6)(12)] = 44.8\ psi < 140\ psi \quad \therefore$ OK

Check Raker:

T = 8(5,202) = 41,616 Lb

Angle $\theta = \tan^{-1} 15/8.3 = 61°$

$L = [(15)^2 + (8.3)^2]^{1/2} = 17.14$ Ft

Axial load = 41,616(17.14/15) = 47,553 Lb

P/A = 47,553/(12)(12) = 330 psi

Allowable $F_c = 480{,}000/(L/d)^2 = 480{,}000/[(17.14)(12)/12]^2 = 1{,}634$
$= 1{,}600$ psi max

$330 < 1{,}600 \quad \therefore$ OK

Check Pad:

Determine allowable soil pressure under 6 x 12 pads:

(Use NAVFAC inclined load on inclined footing - See Appendix B)

Angle of pad to horizontal = 61°

D/B = (4.0/5.0) = 0.8 $\quad N_{\gamma q}$ From graph = 20

$q_{ult} = CN_{cq} + 1/2(\gamma B)N_{\gamma q} = 0 + 1/2(115)(5)(20) = 5{,}750$ psf

$q_{Allowable}/FS = 5{,}750/2 = 2{,}875 > 2{,}000$ psf Use 2,000 psf

Pad bearing area needed = 47,553/2,000 = 23.78 Ft^2

Pad length needed = 23.78/5.00 = 4.76 Ft

Pad cantiliver length = (4.76 - 1.00)/2 = 1.88 Ft

M (for 1 pad) = $wL^2/2 = 2,000(1.88)^2/2 = 3,534$ Ft-Lb

$f_b = M/S = 3,534(12)/[12(6)^2/6] = 589 < 1,500$ psi

Shear V = 2,000(1.88 - 0.5) = 2,760 Lb

$f_v = 3V/2A = 3(2,760)/2(6 \times 12) = 58 < 140$ psi

Check Corbel:

Raker to corbel crushing = 47,553/12 x 12 = 330 < 450 psi

Length for flexure = (5.00 - 1.00)/2 = 2.00

Load per foot of corbel = 2,000(4.76) = 9,520 Lb/Et

$M = wL^2/2 = 9,520(2)^2/2 = 19,040$ Ft-Lb

$f_b = M/S = 19,040(12)/[12(12)^2/6] = 793 < 1,800$ psi

Length for shear = 5.00/2 - 1.00/2 - 1.00 = 1.00 Ft

Shear V = 2,000(4.76)(1.00) = 9,520 Lb

$f_v = 3(9,520)/2(12 \times 12) = 99 < 144$ psi

Summary:

Use HP12 x 74 (equivalent or larger) soldier pile (D = 10'- 3").
Use 6 x 12's for lagging (may use 4 x 12 for upper half).
Use 12 X 12 raker.
Provide at least 5 x 5 pad.

SAMPLE PROBLEM 10-5: PREVIOUS PROBLEM WITH NO RAKER

Given:
Analyze previous problem using an H of 8 feet and no raker.

Solution:
Instead of adjusting K_p per FHWA recommendation, use a safety factor of 30% for D.

$K_a = 0.65$ $K_p = 3.69$ (Arbitrary values)

$K_p - K_a = 3.69 - 0.65 = 3.04$

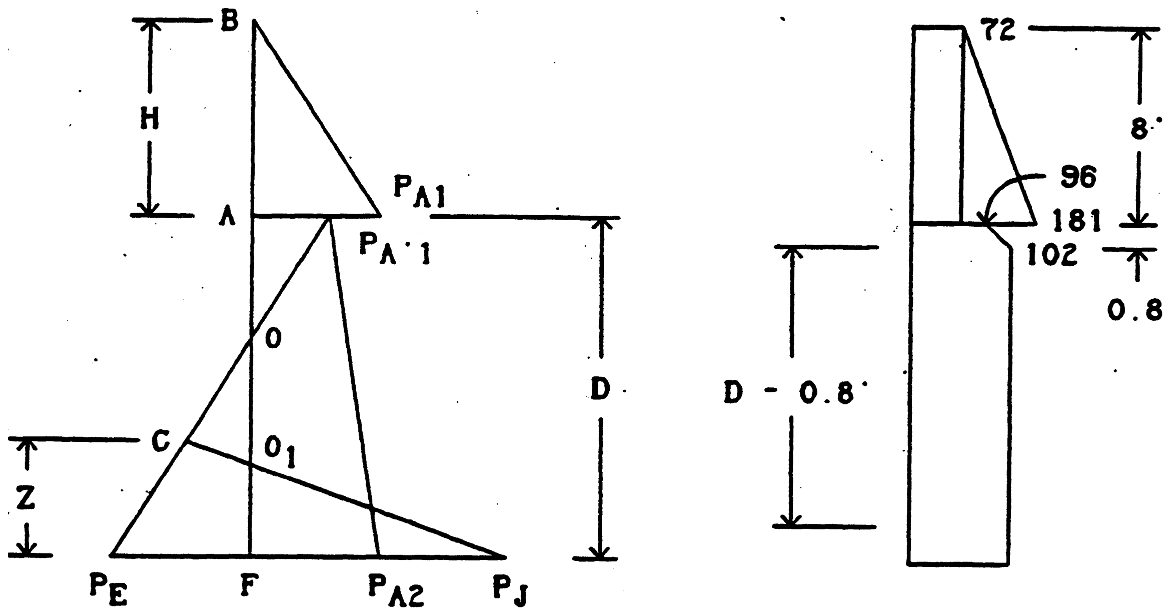


FIGURE 10 - 16

$$P_{A1} = K_a\gamma H = (0.65)(115)(8) = 598 \text{ psf}$$

$$P_{A'1} = (0.53)(598) = 317 \text{ psf}$$

$$P_{A2} = P_{A'1} + f\gamma DK_a = 317 + (0.53)(115)(0.65)D - 317 = 317 + 40D$$

$$P_E = f\gamma D(K_p - K_a) - P_{A'1} = (0.53)(115)(3.04)D - 317 = 185D - 317$$

$$P_J = f\gamma D(K_p - K_a) + f\gamma HK_p = (0.53)(115)(3.04)D + (0.53)(115)(8)(3.69) = 1{,}799 + 185D$$

Areas:

$ABA_1 \quad 598(8)/2 = 2,392$

$AA'_1A_2F = (317 + 317 + 40D)D/2 = 317D + 20D^2$

$ECJ = (185D - 317 + 1,779 + 185D)Z/2 = 185DZ + 731Z$

$EA'_1A_2 = -(185D - 317 + 317 + 40D)D/2 = -113D^2$ (negative area)

Surcharge area (for ease of computation assume surcharge uniform below excavation):

$72(8) + (181 - 72)(8)/2 + (102)D = 1,012 + 102D$

$\Sigma F_H = 0 \quad \therefore ABA_1 + AA'_1A_2f + ECJ - EA'_1A_2 + \text{surcharge} = 0$

$2,392 + 317D + 20D^2 + 185DZ + 731Z - 113D^2 + 1,012 + 102D = 0$

$z = (D^2 - 4.51D - 36.60)/(1.99D + 7.86)$

$\Sigma M_F = 0$

$$= ABA_1[D + 8/3] + (P_{A'1})(D)[D/2] + (P_{A2} - P_{A'1})(D/2)[D/3] + ECJ[Z/3] - EA'_1A_2[D/3] + 576[8/2 + D] + 436[8/3 + D] + 102D[D/2]$$

$$2,392[D + 2.7] + 158D^2 + 6.67D^3 + (185DZ + 731Z)[Z/3] - 37.67D^3 + (2,304 + 576D) + (1,163 + 436D) + 51D^2 = 0$$

$Z^2 = (D^3 - 6.74D^2 - 109.81D - 320.17)/(1.99D + 7.86)$

By trial and error, or by other means: Z = 4.15' and D = 16.88'
Increase D by 30% $\quad$ D = 1.30(16.88) = 21.9 Ft
Use D = 22 Ft

Check Soldier Pile: Find $_{max}$

Locate point of zero shear.
Assume point between 0 & G).

$AO/P_{A'1} = D/(P_{A'1} + P_E)$
$AO = 1.71'$

$OG = D - AO - z$
$= 16.88 - 1.71 - 4.15 = 11.02'$

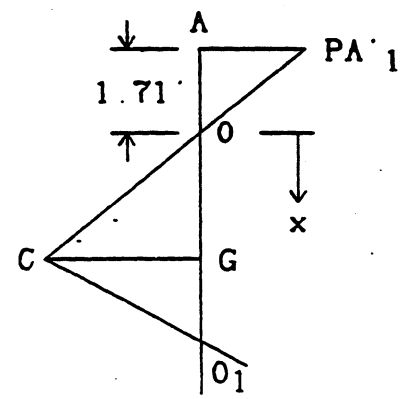


FIGURE 10 - 17

Shear at point 0 $= 2,392 + 576 + 436 + 317(1.71)/2 + (102)(1.71)$

$= 3,849$ Lb/LF

$3,849 + 102x = 185x(x/2)$

$92.5x^2 - 102x - 3,849 = 0$

$x^2 - 1.10x - 41.61 = 0 \quad \therefore x = 7.02'$ (and assumption is correct)

$AO + x = 1.71 + 7.02 = 8.73'$

$$M_{max} = 2,392[8/3 + 8.73] + 576[8/2 + 8.73] + 436[8/3 + 8.73]$$
$$271.0[(1.71)(2/3) + 7.02] + 102(8.73)[8.73/2]$$
$$-(185)(7.02)(7.02/2)[7.02/3]$$
$$= 34,994 \text{ Ft-Lb/LF}$$

Total soldier pile moment = moment per foot times pile spacing:

$= 8(34,994) = 279,952$ Ft-Lb

S required $= M/F_b = 279,952(12)/22,000 = 152.7$ in^3

S furnished with HP12 x 84 = 106 in^3 < 152.7 in^3'

Use HP14 x 117, S = 172 in^3

A second point of zero shear occurs near the computed depth, but this point is not normally used for maximum moment.

Compare the moment computed above to the moment at the depth of excavation:

$M = 2,392[8/3] + 576[8/2] + 436[8/3] = 9,846 < 34,994$ Ft-Lb/Lf

The portion of piling encased in sound concrete (generally, four sack or better) comprises a composite section usually having a large section modulus. If this is the case, the moment at or above the excavation elevation may be controlling to determine the critical section modulus.

CANTILEVER SOLDIER PILE - COHESIVE SOIL (ϕ = 0 METHOD)

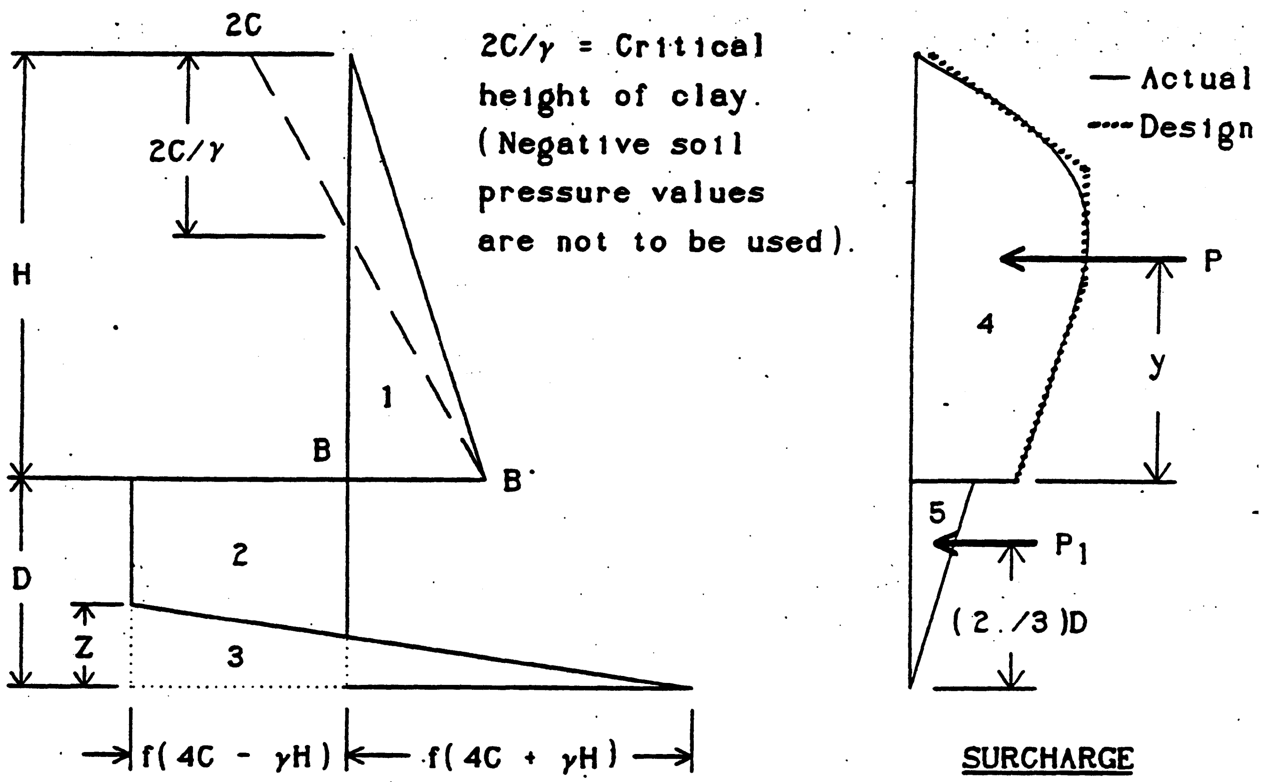


FIGURE 10-38

Use a safety factor of 50% - 70% with the clay or increase D by 20% - 40%. Critical height of wall = $H_c = 4C/\gamma$. Theoretically the wall will fail if $\gamma H_c > 4$.

BB′ = $\gamma H - 2C \geq 0$. (If not, see note below)
f = Arching factor

1 = BB′(H/2)
2 = f(4C − γH)D
3 = {f(4C − γH) + f(4C + γH)}{Z/2} = 4fCZ
4 = P = Area under dashed line above the excavation depth.
5 = P_1 = Area below the excavation depth (when used).

1) $\Sigma F_H = 0 = 1 - 2 + 3 + 4 + 5$ and $Z = (2 - 1 - 4 - 5)/4fC$

2) $\Sigma M_{base} = 0 = 1[H + H/3] - 2[D/2] + 3[Z/3] + 4[D + y] + 5[2D/3]$

Solve equations 1) and 2) simultaneously for D and Z.
Determine maximum moment and section modulus required.
Determine. lagging requirements.

*Note : **If ϕ does not = 0, or if BB′ < 0, see next page.**

CANTILEVER SOLDIER PILE - COHESIVE SOIL (ALTERNATE METHOD)

This approach should be used only when $\phi \neq 0$, or when $BB' \leq 0$.

If $\phi \neq 0$, then $BB' = \gamma K_a H$ where $K_a = \tan^2(45° - \phi/2)$

If $BB' < 0$, then assume $C = 0$ and $\phi = 20°$ to $30°$.
$BB' = \gamma H K_a$ where $K_a = \tan^2(45° - \phi/2)$

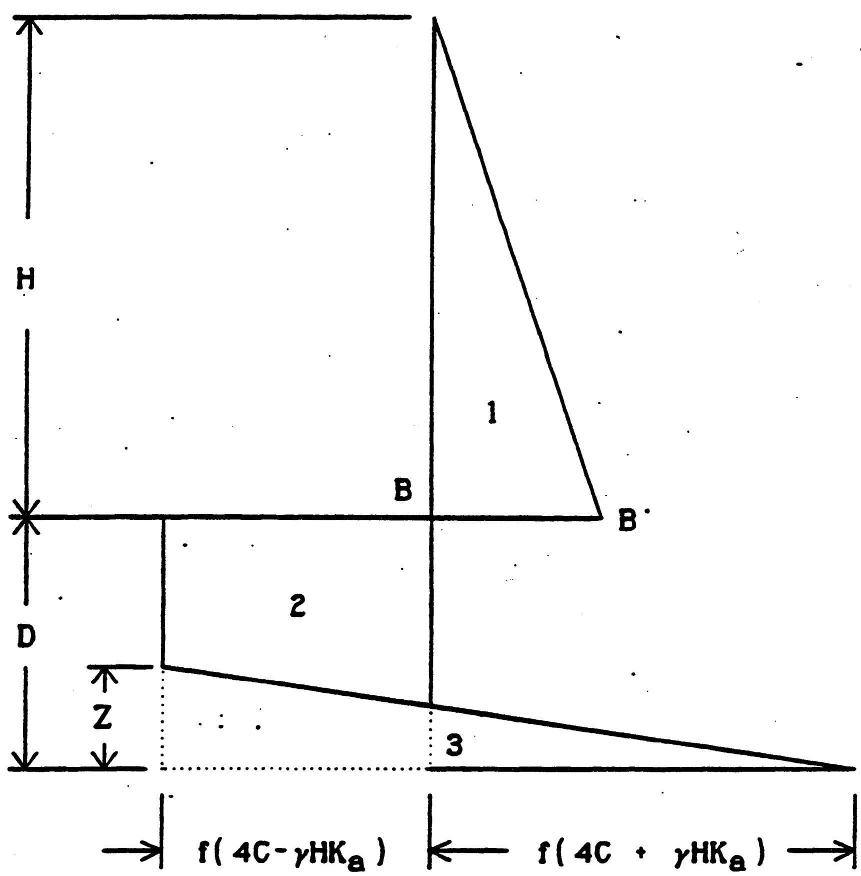
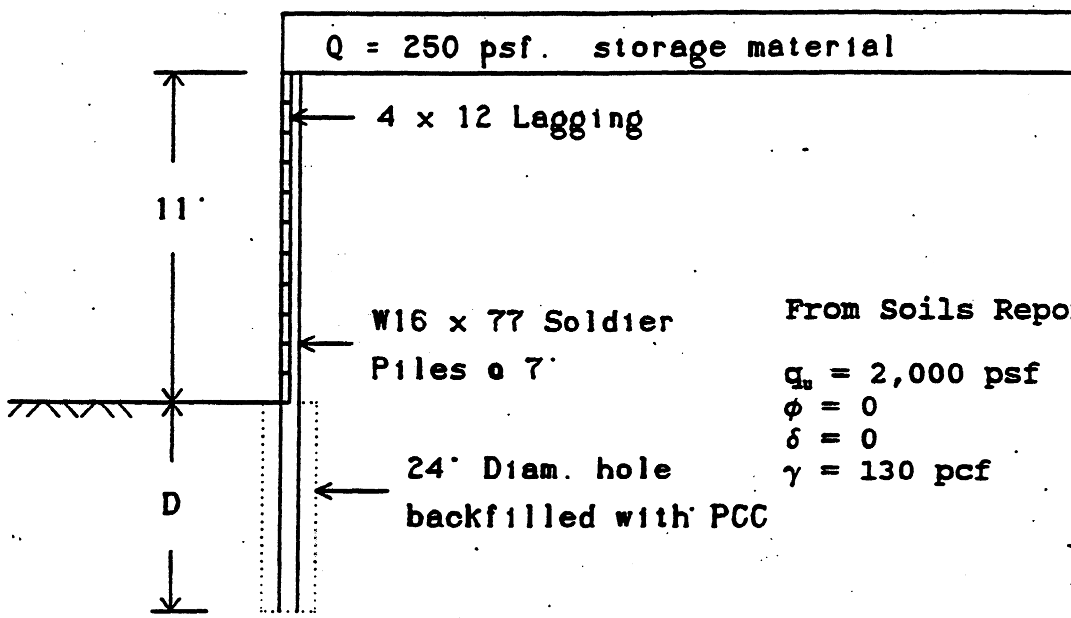


FIGURE 10 - 19

The procedure from this point on, including the addition of any surcharges, is identical to the "$\phi = 0$ method" outlined on the previous page.

SAMPLE PROBLEM 10-6: CANTILEVER SOLDIER PILE: COHESIVE SOIL

Given:



From Soils Report:

q_u = 2,000 psf
ϕ = 0
δ = 0
γ = 130 pcf

FIGURE 10 - 20

Solution:

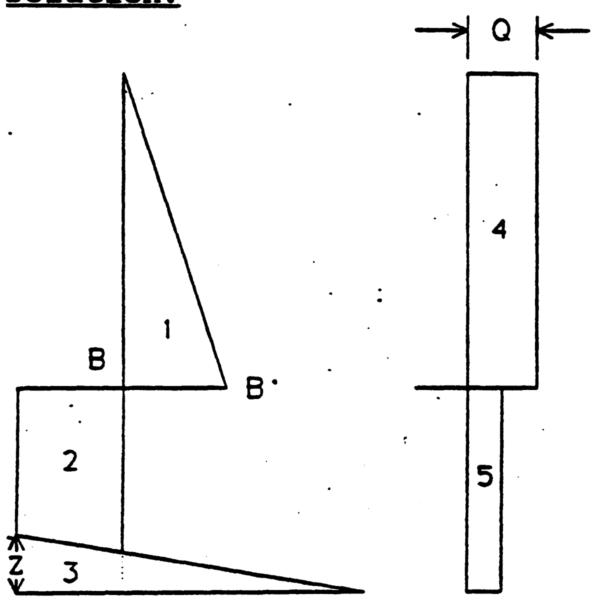


FIGURE 10 - 21

Use a safety factor of 1.6 for clay.

$q_u/1.6 = 2{,}000/1.6 = 1{,}250$ psf
$C = q_u/2 = 1{,}250/2 = 625$ psf

Arching capability = 2.0
$f = 2(24/12)/7 = 0.57$

Check critical height of wall:
$H_c = 4C/\gamma$
$= 4(625)/130 = 19.2' > 11'$

$BB' = \gamma H - 2C$
$= 130(11) - 2(625)$
$= 180$ psf

AREAS:
1 = $bb'h/2 = 180(11)/2 = 990$
2 = $F(4c - \gamma h)d = [(0.57)(4)(625) - 130(11)]D = 610D$
3 = $4fCZ = 4(0.57)(625)Z = 1{,}425Z$
4 = $QH = 250(11) = 2{,}750$
5 = $fQD \doteq 0.57(250)D = 143D$

Determine D:

$\Sigma F_H = 0$

$= 990 - 610D + 1,425Z + 2,750 + 143D = 0$

$3,740 + 1,425Z - 467D = 0$

1) $Z = 0.33D - 2.62$

$\Sigma M_{base} = 0$
$= 990[D + 11/3] - 610D[D/2] + 1,425Z[Z/3] + 2,750[D + 11/2]$
$+143D[D/2] = 3,740D + 18,755 + 475Z^2 - 233D^2$

2) $Z^2 = 0.49D^2 - 7.87D - 39.48$

By trial and error, or by other means, solve equations 1) and 2):

There is no need to increase D since a safety factor has already been applied to the clay.

<u>Find Maximum Moment:</u> (Assuming non-composite section):

Locate point of zero shear.

Shear at excavation elevation.
990 + 250(11) = 3,740 psf/LF

Determine distance below excavation level to obtain shear. equality:

$610x - 143x = 467x$ psf/LF
$467x = 3,740 \quad \therefore x = 3,740/467$

$M_{max} = 990[8.01 + 11/3] + 250(11)[8.01 + 11/2]$
$+143(8.01)[8.01/2] - 610(8.01)[8.01/2]$
$= 33,731$ Ft-Lb/LF

S required = $33,731(7)(12)/22,000 = 128.8\ in^3$
S furnished = $134\ in^3 > 128.8\ in^3$ OK

Often, with sound concrete below the depth of excavation, the moment occurring at that elevation may be used to determine the critical section modulus.

Check Lagging:

Consider arching effect on lagging. Multiply all loads by 0.6. By inspection, maximum moment occurs at the depth of excavation.

$M_{max} = wL^2/8 = (180 + 250)(7)^2/8 = 2,634$ Ft-Lb

S required = $2,634(12)(0.6)/(1,500) = 12.43$ in^3.

S furnished (rough lumber) = 32 in^3 > 12.43 in^3 OK

$V = (7/2 - 0.33)(180 + 250)(0.6) = 818$ Lb

$v = 3V/2A = 3(818)/[2(4)(12)] = 25.6$ psi < 140 psi OK

EFFECT OF SURCHARGE BELOW DEPTH OF EXCAVATION

Sample problems 10-1, 10-3, 10-6 and 22 (Appendix F) were recomputed using no surcharge below the depth of excavation to demonstrate the negligible difference in answers. A comparison of answers follows:

SAMPLE PROBLEM 10-1	WITH SURCHARGE	WITHOUT SURCHARGE
D	22.3′	21.1′
130%(D)	29.0′	27.4′
Z	4.91′	4.7′
M_{total}	1,082,016 Ft-Lb	1,009,632 Ft-Lb
S Required	590.2 in^3	550.7 in^3

SAMPLE PROBLEM 10-3		
D_w	5.62′	5.2′
D	11.62′ Use 11′- 8"	11.2′ Use 11′-3"
T	10,785 Lb/LF	10,685 Lb/LF
Total T	80,888 Lb	80,138 Lb
Combined Moment	35,607 Ft-Lb/LF	34,587 Ft-Lb/Lf
S Required	145.67 in^3	141.5 in^3

SAMPLE PROBLEM 10-6		
Z	4.54′	5.1′
D	21.71′	18.0′
M_{max}	33,731 Ft-Lb/LF	30,220 Ft-Lb/LF
S Required	128.8 in^3	115.4 in^3

SAMPLE PROBLEM 22 (Appendix F)		
Y	20.45 ′	19.77′
D	24.26′	23.63′

ACCEPTABLE ALTERNATE DESIGN METHODS WHICH HAVE BEEN USED

Cantilever System or Single Tie (Or Strutted) System:

Surcharge (S) may be limited to a depth of 10 feet or more, or to the elevation of the upper tie or strut (depicted as force T).

Dimension A is designers choice.

The location of point M is the designers choice. M may be located anywhere between points L to N. Point N is used for cantilever sheetpile or continuous walls.

Forces above L-N represent active loads on the soldier pile.

Passive forces are based on the effective pile diameter which includes the unitless number 3 (divided by an appropriate safety factor), times the pile dimension or drilled hole diameter, times K_p, times the unit weight of the soil.

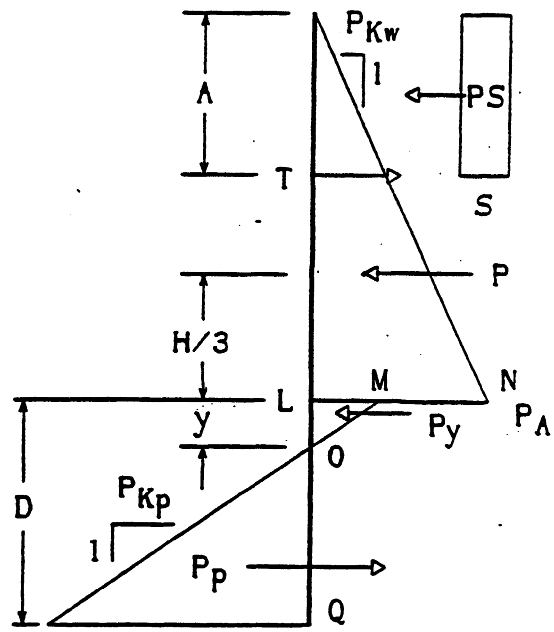


FIGURE 10 - 22

$y = LM/[3dK_p\gamma/S.F.]$ (May not include contribution of surcharge).

Cantilever Systems:

D is generally determined from moments taken about point Q. A safety factor against overturning should be included so that passive moments exceed active moments, all taken above point Q.

The section modulus of the soldier pile may be determined from moments taken about the point of zero shear.

Single Tie Or Strut System: (Only one method described below.)

T may be determined from moments of the forces above point 0.

D may be determined from moments of the forces about T.

Check stability against overturning by taking moments about Q.

The section modulus required for the soldier pile is determined from the larger of the cantilever moment for the forces above point T, or from moments taken about the plane of zero shear.

AASHTO Soldier Pile Method

The figure below represents the soil pressures that may be used for cantilever soldier piles cohesionless soil This figure (excluding surcharge) is an adaptation from the figure titled, Simplified Earth Pressure Distributions for Permanent Flexible Cantilevered Walls with Discrete Vertical Wall Elements, in AASHTO Standard Specifications For Highway Bridges.

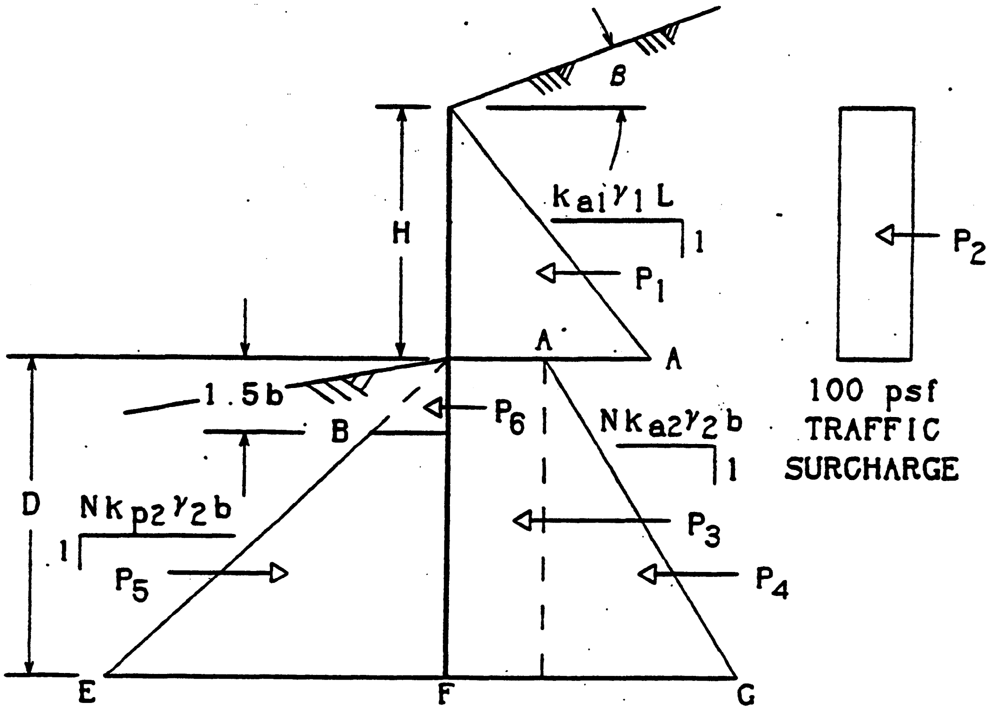


FIGURE 10 - 23

L = Soldier Pile Spacing
N = Increase in effective pile width. AASHTO uses N = 3, (Caltrans uses N = 0.08ϕ). The value of Nb cannot exceed the soldier pile spacing "L" (AASHTO uses $L \leq 5b$).
b = Effective pile width: pile width or width of drilled hole backfilled with hard rock concrete.

PROJECT USE:

A-minimum safety factor of 50% is to be added to the computed embedment depth (D) for permanent flexible cantilever walls, otherwise 30% should be added for temporary construction.

For temporary construction the forces P_3 and P_4 may be ignored.

GENERAL PRESSURE EQUATIONS:

$P_A = K_{a1}\gamma_1 H(L)$ $\quad$ $P_B = K_{p2}\gamma_2(1.5b)(Nb)$ $\quad$ Schrg = $P_S(L)$

$P_{A'} = K_{a2}\gamma_2 H(Nb)$ $\quad$ $P_G = K_{a2}\gamma_2 D(Nb)$ $\quad$ $P_E = K_{p2}\gamma_2 D(Nb)$

SAMPLE PROBLEM 10 - 7: USING THE AASHTO METHODOLOGY

$\phi = 32°$ $\beta = \phi$

$\gamma_1 = \gamma_2 = 110$ pcf

L = 7 Ft

b = 1.33 Ft

N = 0.08(32)
= 2.56

Nb = 2.56(1.33)
= 3.40

From Log-Spiral:
$K_a = 0.80$
$K_p = 8.0(0.425)$
$= 3.4$

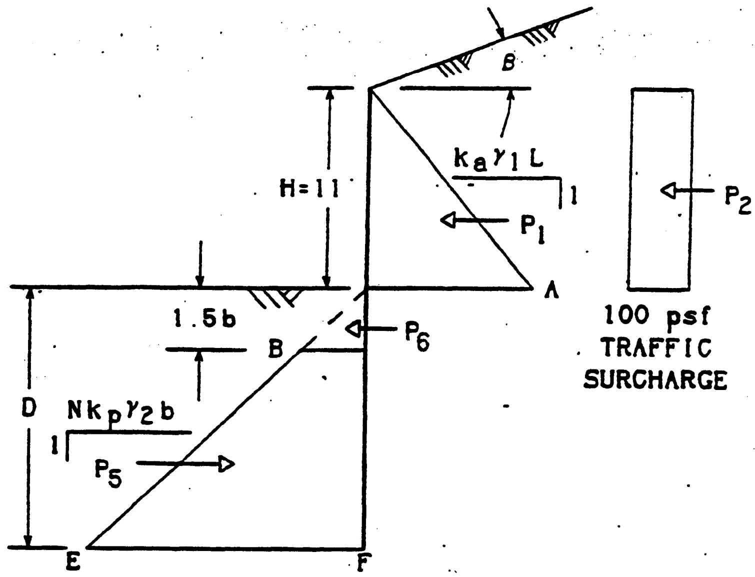


FIGURE 10-24

General Equations:

$P_A = K_a\gamma_1 H(L) = 0.8(110)(11)(7) =$	6,776 Lb/Ft
$P_S = 100(L) = 100(7) =$	700
$P_E = K_p\gamma_2 D(Nb) = 3.4(110(3.4)D =$	1,271.6D
$P_B = K_p\gamma_2(1.5b)(Nb) = 3.4(110)(1.5)(1.33)(3.40) =$	2,536.8
$P_1 = P_A(H/2) =$	37,268 Lb
$P_2 = P_S H =$	7,700
$P_6 = P_B(1.5b/2) =$	2,530.5
$P_5 = P_E(D/2) =$	$- 635.8D^2$

Determine D by Taking Moments About F:

$0 = P_1[D + H/3] + P_2[D + H/2] + P_6[D - 2/3(1.5b)] - P_5[D/3]$

$= 37{,}268D + 136{,}649.3 + 7{,}700D + 42{,}350 + 2{,}530.5D - 3{,}365.6 - 211.9D^3$

$= 211.9D^3 - 47{,}498.5D - 175{,}633.7$ From which D = 16.56'

Use D = 1.30(16.56) = 21.5 Ft

Locate Depth To Plane Of Zero Shear:

(Use x in lieu of D in Figure 10 - 25)

$P_1 + P_2 + P_6 = P_5$

$37{,}268 + 7{,}700 + 2{,}530.5 = 635.8(x^2)$

$x^2 = 74.71'$ $\qquad x = 8.64'$

Determine Moment At Plane Of Zero Shear:

$M = P_1[8.64 + H/3] + P_2[8.64 + H/2] + P_6[8.64 - 2/3(1.5b)]$
$- P_5[8.64/3]$

$= 37{,}268[12.31] + 7{,}700[14.14] + 2.530.5[7.31] - 635.8(8.64)^2[2.88]$

$= 458{,}769 + 108{,}878 + 18{,}498 - 136{,}691$

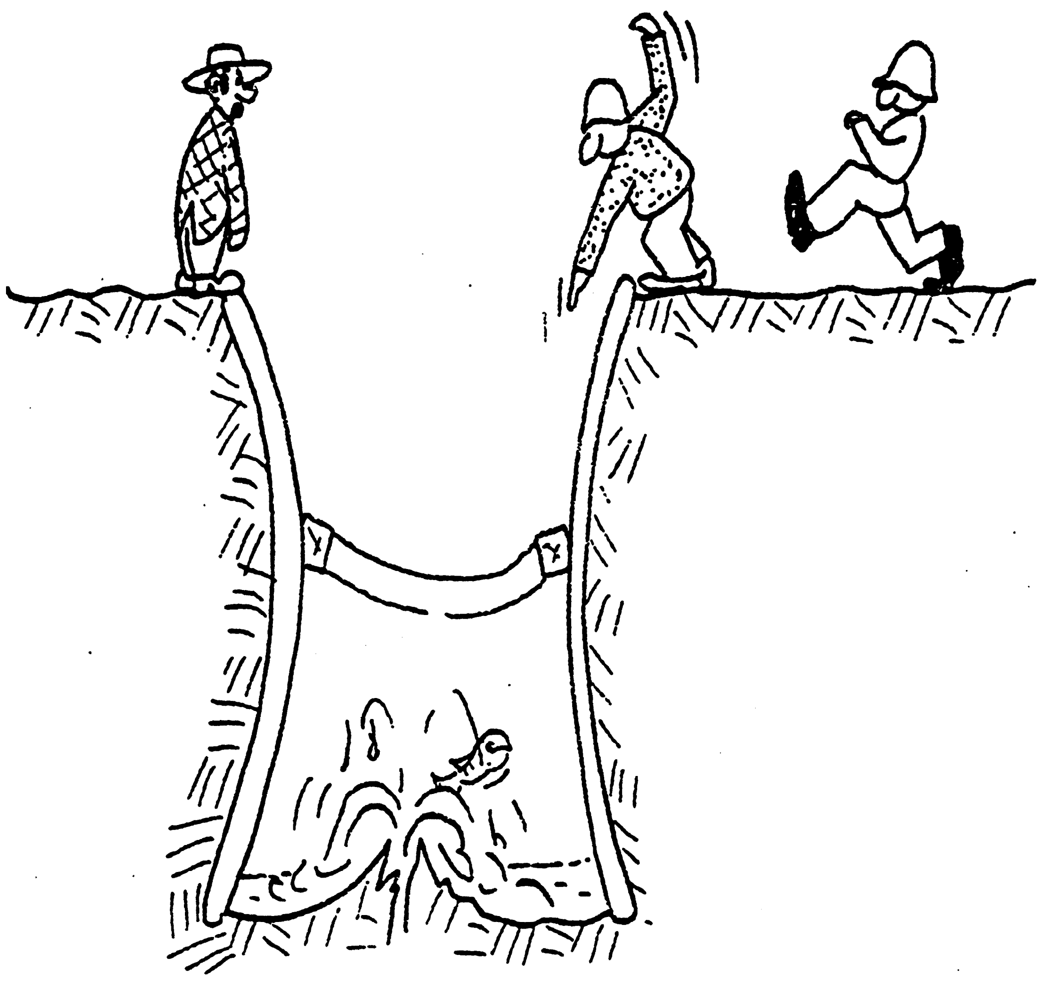
$= 449{,}454$ Ft-Lb

Determine Section Modulus Required:

S = M(12)/22,000 = 449,454(12)/22,000 = 245 in^3

W12 X 190 (S = 263 in^3) Could be used (providing deflection is not a consideration).

SPECIAL CONDITIONS



SPECIAL CONDITIONS

SPECIAL CONDITIONS

The best shoring system in the world would be of little value if the soil being supported does not act as contemplated by the designer. Adverse soil properties and changing conditions need to be considered.

Anchors placed within a soil failure wedge will exhibit little holding value when soil movement in the active zone occurs. The same reasoning holds for the anchors or piles in soils which decrease bonding or shear resistance due to changes in plasticity or cohesion. Additional information regarding anchors may be found in the USS Steel Sheet Piling Design Manual.

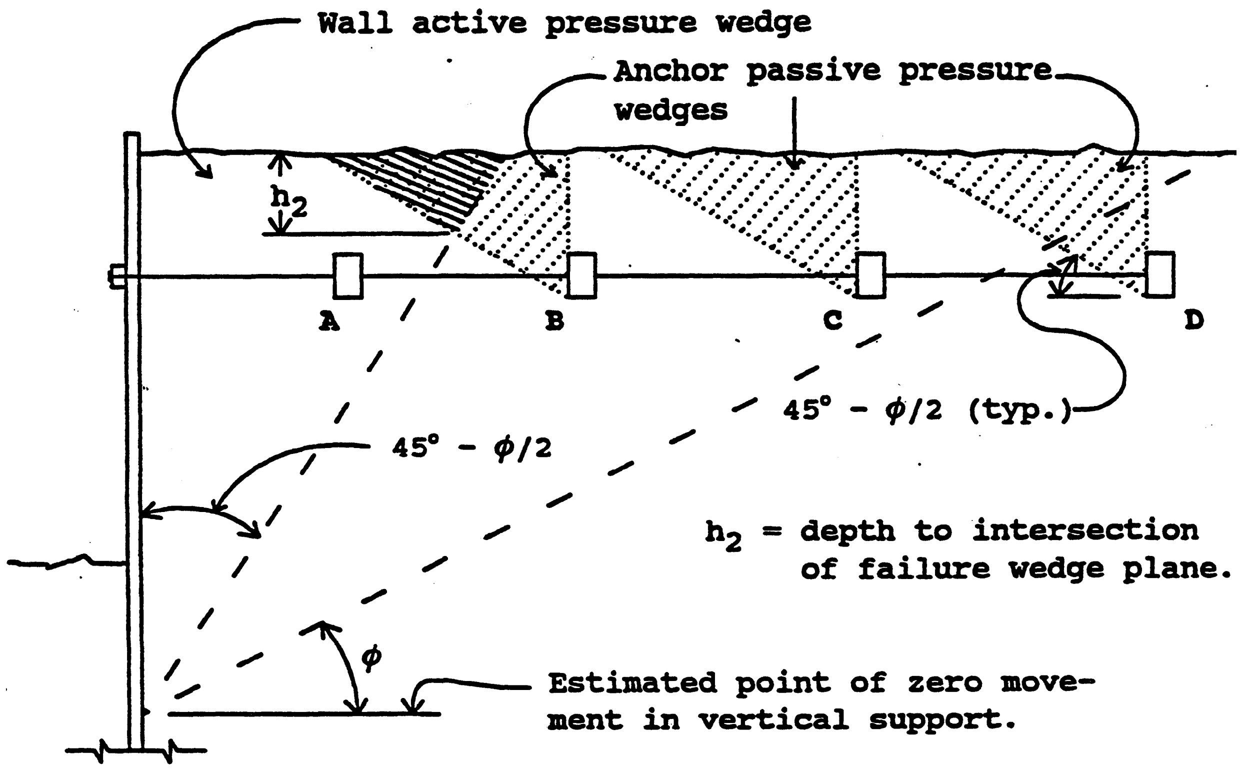
When cohesive soils tend to expand or are pushed upward in an excavation, the shoring wants to move laterally. Soil rising in an excavation indicates that somewhere else soil is settling. Water rising in an excavation can lead to quick conditions, while water moving horizontally can transport soil particles leaving unwanted voids at possibly critical locations.

Avery important consideration always present in all but a few types of shoring systems is the potential for a sudden failure due to slippage of the soil around the shoring system along a surface offering the least amount of resistance.

Sample situations of the above are included on the following pages.

DEADMEN

The size, shape, depth and location of an anchor block affects the resistance capacity developed by that anchor. The following diagram explains how the distance from the wall affects capacity.



Deadman A located inside active wedge and offers no resistance.
Deadman B resistance is reduced due to overlap of the active wedge (wall) and the passive wedge (anchor).

Anchor reduction: (Granular soils)

$$\Delta P_p = (1/2)(K_p - K_a)\gamma h_2^2$$

ΔP_p is transferred to the wall.

Deadman C develops full capacity but increases pressure on wall
Deadman D develops full capacity and has no effect on bulkhead.

Deadmen should be placed against firm natural soil and should not be allowed to settle.

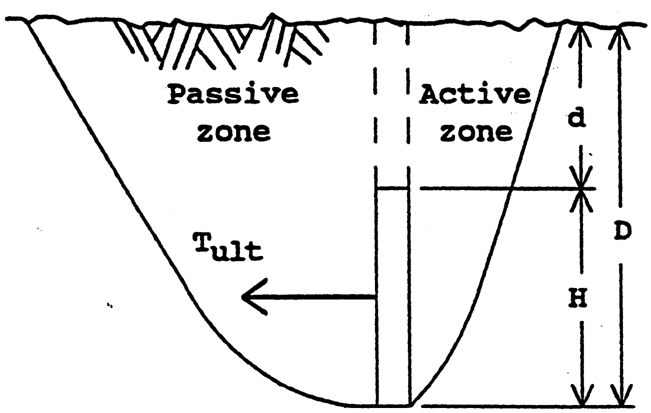
A safety factor of 2 is recommended for all anchors and deadmen.

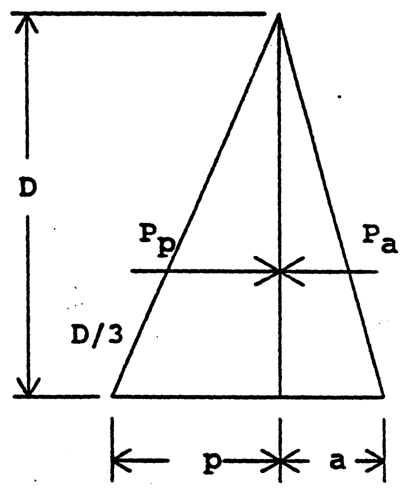
The following criteria is for anchors or deadman located entirely in the passive zone as indicated by Anchor D.

SPECIAL CONDITIONS

DEADMEN IN COHESIONLESS SOIL NEAR GROUND SURFACE :$d \le H/2$.

The forces acting on an anchor are shown in the following diagrams. For this case, $d \le H/2$, it is assumed that the anchor extends to the ground surface.





GRANULAR SOILS

where $p = \gamma D K_p$; $a = \gamma D K_a$

The capacity of a deadman also depends on whether it is continuous (long) or short. A deadman is considered continuous when its length greatly exceeds it height.

The basic equation is:

$$T_{ult} = L(P_p - P_a) \text{ where } L = \text{Length of Deadman.}$$

For continuous Deadmen

$$P_a = K_a \gamma D^2/2 \qquad P_p = K_p \gamma D^2/2$$

$$T_{ult} = \gamma D^2(K_p - K_a)L\}/2$$

For Short Deadmen ($L \le 3H$)

$$T_{ult} = L(P_p - P_a) + (\gamma K_o D^3 \tan\phi [K_p + K_a]^{1/2})/3$$

$K_o = 0.4$ is recommended.

DEADMEN IN COHESIONLESS SOIL where $1.5 \leq D/H \leq 5.5$

This chart is based on sand of medium density, (ϕ= 32.5). For other values of ϕ a linear correlation may be made from (ϕ /32.5). The chart is valid for ratios of depth to height of anchor (D/H) between 1.5. and 5.5.

For square deadman the value from the chart (K_p') is larger than the value for continuous deadman (KP). This is because the failure surface is larger than the actual dimensions of the deadman. In testing it is determined to be approximately twice the width.

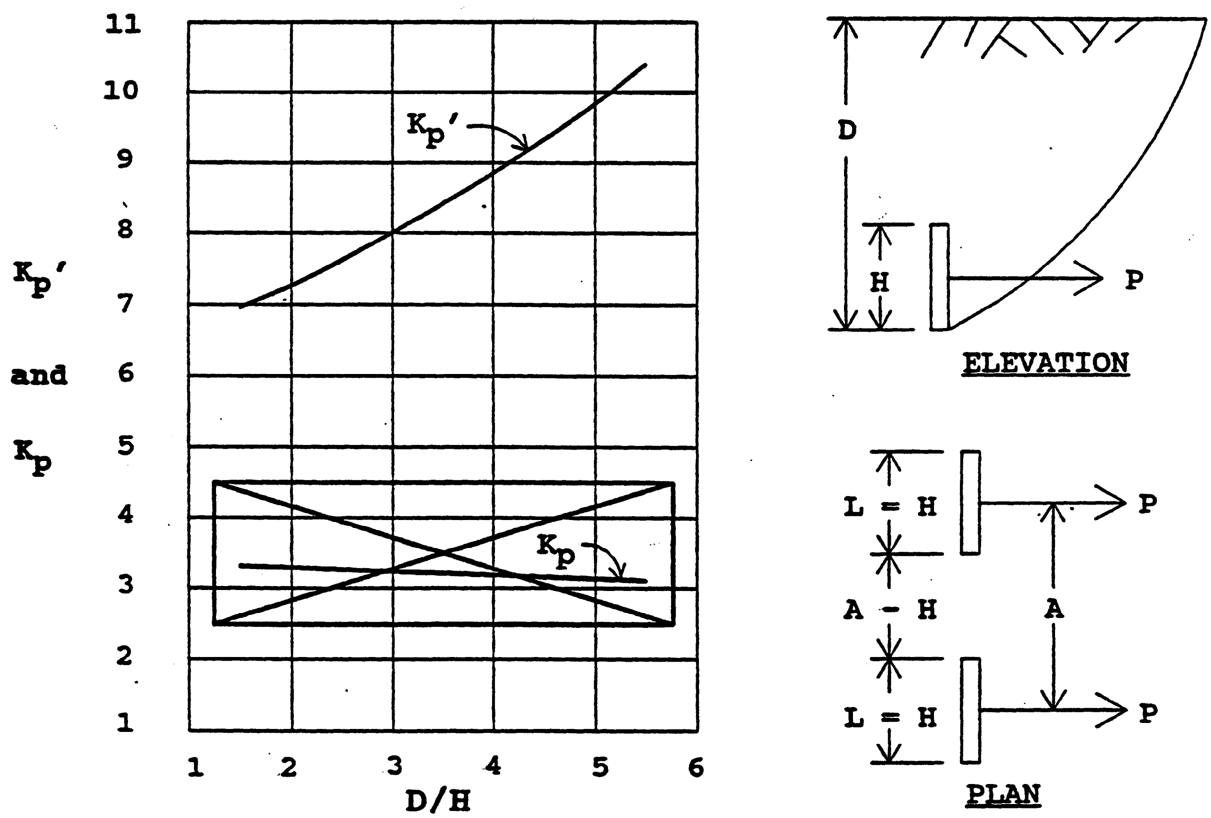


FIGURE 23

For Continuous Deadman

Use Ovesen's method as described in the USS Steel Sheet Piling Design Manual.

For Square (or Short) Deadmen, L = H

$$P_{ult} = (\gamma H^2 K_p' L)/2$$

It is recommended that a factor of safety of 2 be used.

<u>DEADMEN IN COHESIONLESS SOIL</u> **where D/H $\geq$ 5.5**

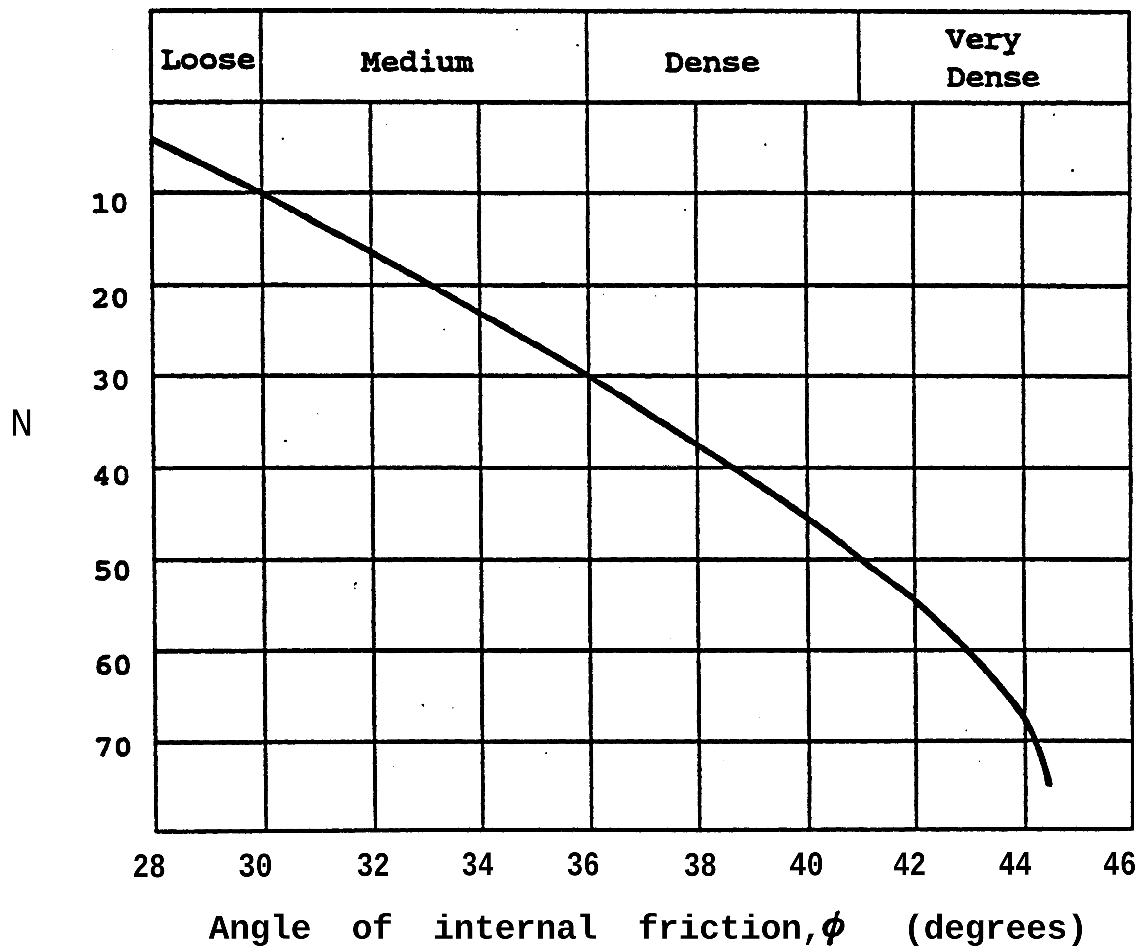
When deadmen are placed at great depth the resistance can be approximately calculated as the capacity of a footing at a depth equal to the center of the deadman. Resistance can also be estimated from the following equations and chart.

T_{ult} = Bearing capacity of a footing at a depth equal to D + H/2 (see page 11-3).

Where water is not a factor:

$$T_{ult} \text{ (Square Block)} = 2LN^2 + 6H(100 + N^2)(LH)$$

$$T_{ult} \text{ (Long Block)} = 3LN^2 + 5H(100 + N^2)(LH)$$

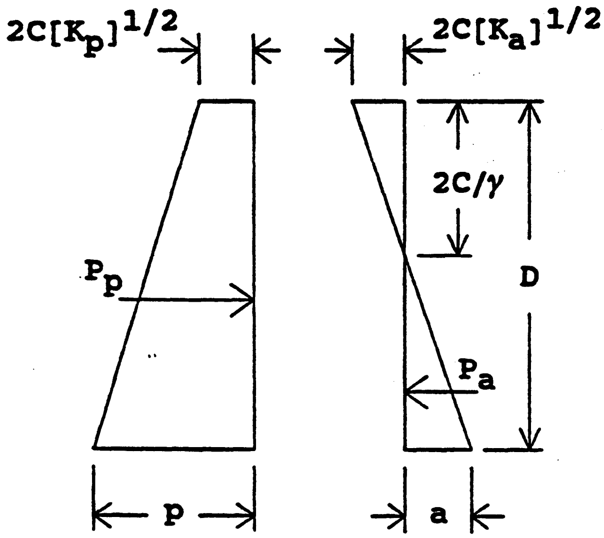


N = Standard Penetration Resistance (Number of blows per foot)

FIGURE 24

<u>DEADMEN IN COHESIVE SOIL NEAR THE GROUND SURFACE</u> $d \leq H/2$

The forces acting on an anchor are shown in the following diagrams. For this case, $d \leq H/2$ (see page 11-3), it is assumed that the anchor extends to the ground surface, Capacity of the anchor depends upon whether it is considered continuous or short.



COHESIVE SOIL

where $p = \gamma DK_p + 2C[K_p]^{1/2}$
$a = \gamma DK_a - 2C[K_a]^{1/2}$

The pressure diagram for cohesive soils assumes a short load duration. For a duration of a period of years it is likely that creep will change the pressure diagram. Therefore conservative assumptions should be used in the analysis, such as:

$$C = 0 \quad \text{and} \quad \phi = 27°$$

The basic equation is:

$$T_{ult} = L(P_p - P_a) \text{ where } L = \text{Length of Deadman.}$$

For Continuous Deadmen:

$$P_p = \gamma D^2 K_p/2 + 2CD[K_p]^{1/2}$$

$$P_a = (\gamma DK_a - 2C[K_a]^{1/2})(D - 2C/\gamma)/2$$

It is recommended that the tension zone be neglected.

$$T_{ult} = L(P_p - P_a$$

For Short Deadmen:

$$T_{ult} = L(P_p - P_a) + 2CD^2$$

DEADMEN IN COHESIVE SOIL where d $\geq$ H/2

A chart has been developed through testing for deadmen other than near the surface. This chart relates a dimensionless coefficient (R) to the ratios of depth to height of an anchor (D/H) to determine the capacity of the deadman.

This chart applies to continuous anchors only.

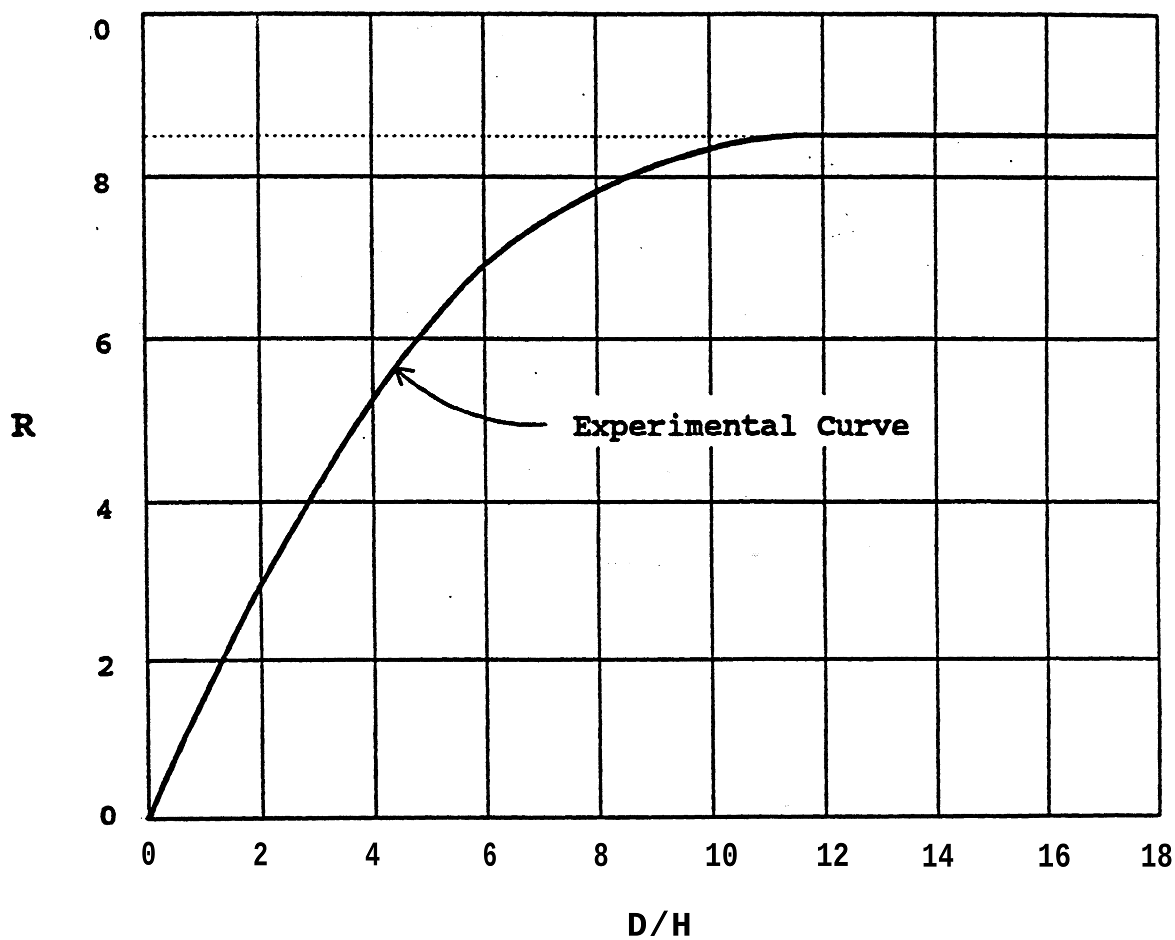


FIGURE 25

The above graph is from Strength of Deadmen Anchors in Clay, Thomas R. Mackenzie, Master's Thesis Princeton University, Princeton, New Jersey, 1955.

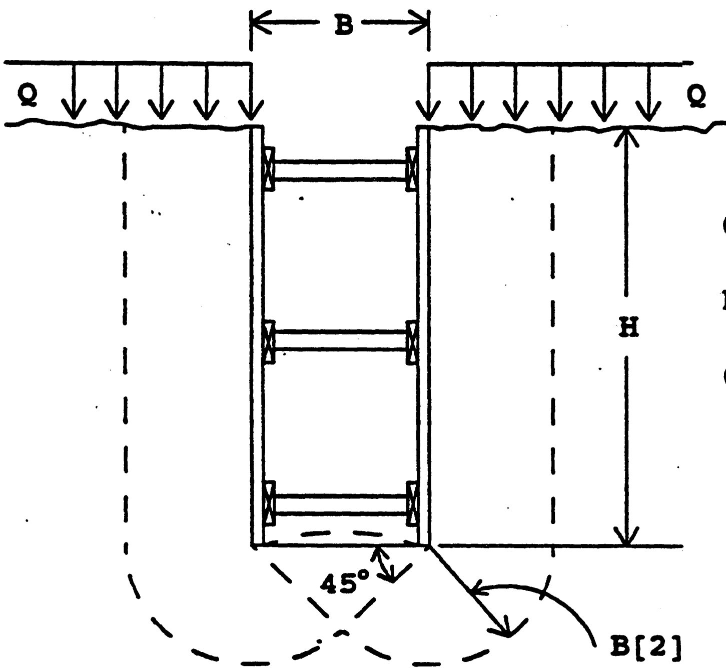
P_{ult} = RCHL with a maximum value of R = 8.5.

It is recommended that a factor of safety of 2 be used.

HEAVE

The condition of heave can occur in soft plastic clays when the depth of the excavation is sufficient to cause the surrounding clay soil to displace vertically with a corresponding upward movement of the material in the bottom of the excavation.

The possibility of heave and slip circle failure in soft clays, and in the underlying clay layers, should be checked when the Stability Number (N_o) exceeds 6 (Stability Number, $N_o = \gamma H/C$).



The relative layout of the excavation influences how heave may be checked. The two conditions depend on whether the sides of the excavation are in close proximity of each other as comparedto the depth.

For the condition of H < B (wide, shallow excavations)

Critical Height $H_C = (5.7C - Q)/\{\gamma - (C/B)[2]^{1/2}\}$ (Terzaghi)

For the condition of H > B (trench type excavations)

critical Height $H_C = (CN_c - Q)/\gamma$ (Skempton)

The Bearing capacity factor, N_C, is determined from FIGURE 26 on the following page.

It is recommended that a minimum safety factor of 1.5 be applied to we Unit Cohesion of soil (use C/1.5).

If heave is probable while using the minimum safety factor it could be prevented by extending the shoring system (sheeting) below the bottom of the excavation into a more stable layer, or for a distance of one-half the width of the excavation (typically valid for only excavations where H>B). Another solution would be overexcavating and constructing a counterweight or tremie seal.

NOTE- Strutting a wall near its bottom will not prevent heave but such strutting may prevent the wall from rotating into the excavation.

A procedure for calculating the critical height H_c, at which heaving could occur is outlined on the previous page.

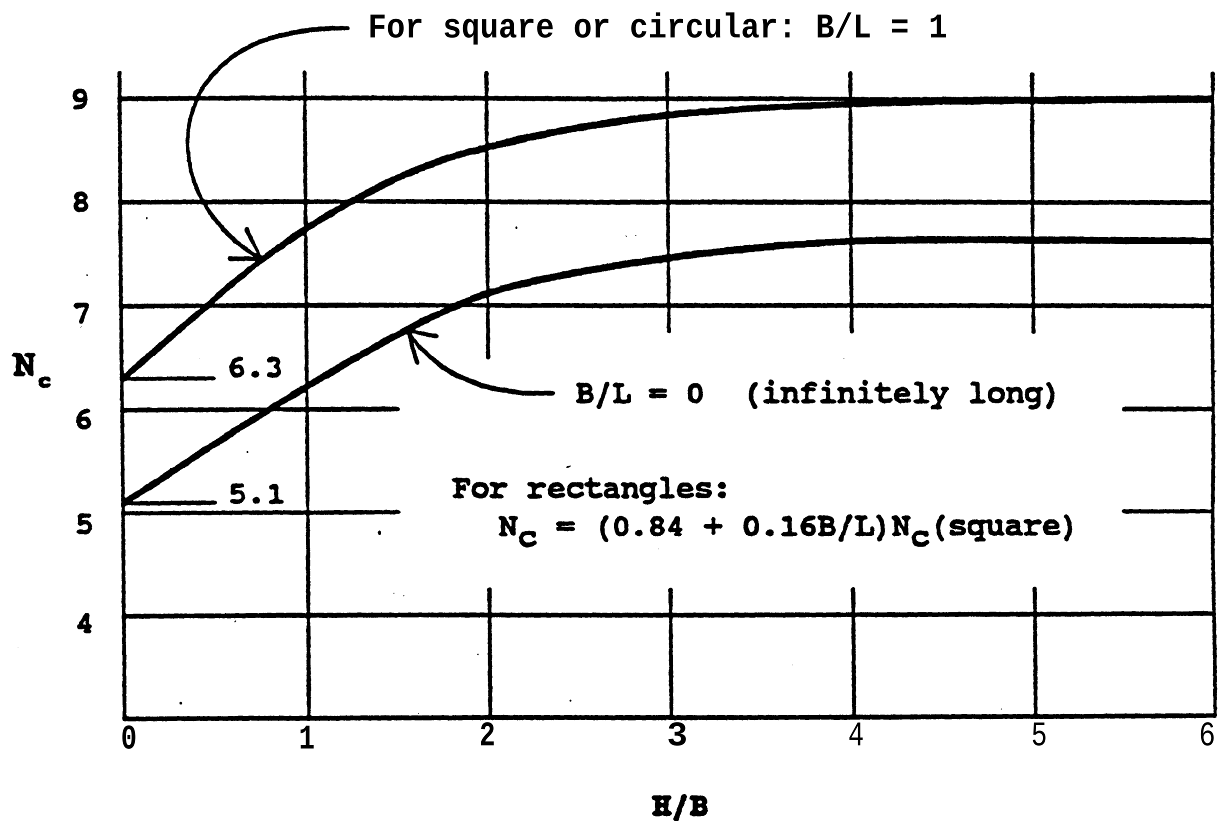


Diagram for determining the Bearing Capacity Factor, N_C

FIGURE 26

EXAMPLE PROBLEM for H < B

Given: $H = 12'$, $B = 20'$, $L = 40'$
$Q = 300$ psf, $C = 400$ psf, $\gamma = 115$ pcf

Solution:
$$H_c = (5.7C - Q)/\{\gamma - (C/B)[2]^{1/2}\}$$
$$= \{5.7(400) - 300\}/\{115 - (400/20)(1.414)\}$$
$$= 22.8' > 12'$$

If we were to apply the minimum recommended Safety Factor of 1.5, the value of C to use would be:

$C/1.5 = 267$ psf.

$$H_c = \{5.7(267) - 300\}/\{115 - (267/20)(1.414)\}$$
$$= 12.7' > 12'$$

Since H is less than H_c with a Safety Factor- considered, heave is not expected to occur.

EXAMPLE PROBLEM for H > B

Given: $H = 25'$, $B = 10'$, $L = 40'$
$Q = 300$ psf, $C = 400$ psf, $\gamma = 115$ pcf

Solution: $H/B = 25/10 = 2.5$, $B/L = 10/40 = 0.25$

From FIGURE 26 on the previous page,

N_c square = 8.8

For rectangle:

$N_c = \{0.84 + 0.16(0.25)\}8.8 = 7.74$

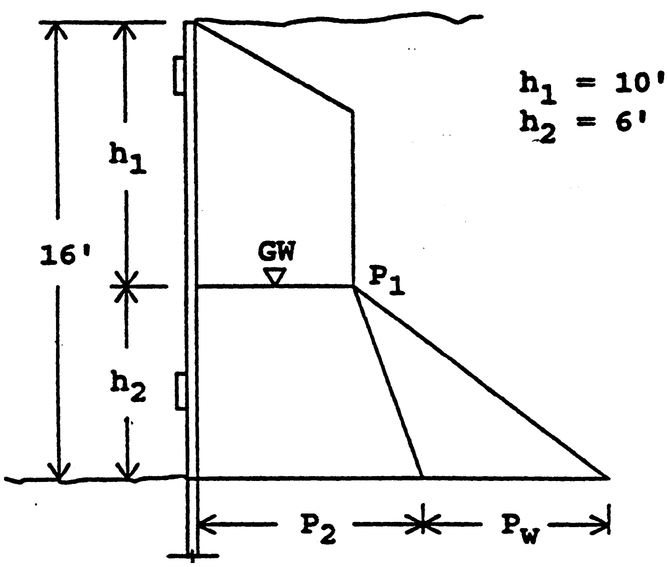
Now applying the Safety Factor of 1.5 to the Cohesive value; $C = 400/1.5 = 267$ psf.

$$H_c = (CN_c - Q)/\gamma$$
$$(267(7.74) - 300)/115$$
$$15.4' < 25'$$

Since $H_c < H$, heave is likely to occur. For this case extending the shoring +5' deeper should be considered.

GROUND WATER

EXAMPLE: Soldier Piles w/Struts (restrained system)



$h_1 = 10'$
$h_2 = 6'$

From Soils Report

$\gamma = 110$ pcf
$\gamma_b = 0.6\gamma$
$= 66$ pcf
$Kw = 38$ pcf

$Kw = K_a\gamma$
$K_a = Kw/\gamma = 38/110$
$= 0.345$

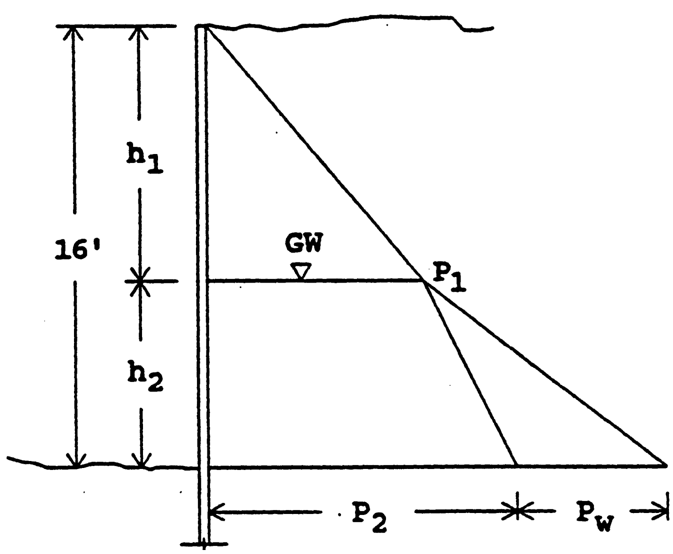
$P_1 = 0.71K_a\gamma h_1 = 0.71Kwh_1$

$P_1 = (0.71)(38)(10)$
$= 270$ psf

$P_2 = P_1 + 0.71K_a\gamma_b h_2 = 270 + (0.71)(0.345)(66)(6) = 367$ psf

$P_w = (62.4)(6) = 374$ psf

EXAMPLE: Sheet Piling (cantilevered system)



Same conditions as above

$Kw = K_a\gamma$
$K_a = Kw/\gamma = 38/110$
$= 0.345$

$P_1 = K_a\gamma h_1 = Kwh_1$

$P_1 = (38)(10)$
$= 380$ psf

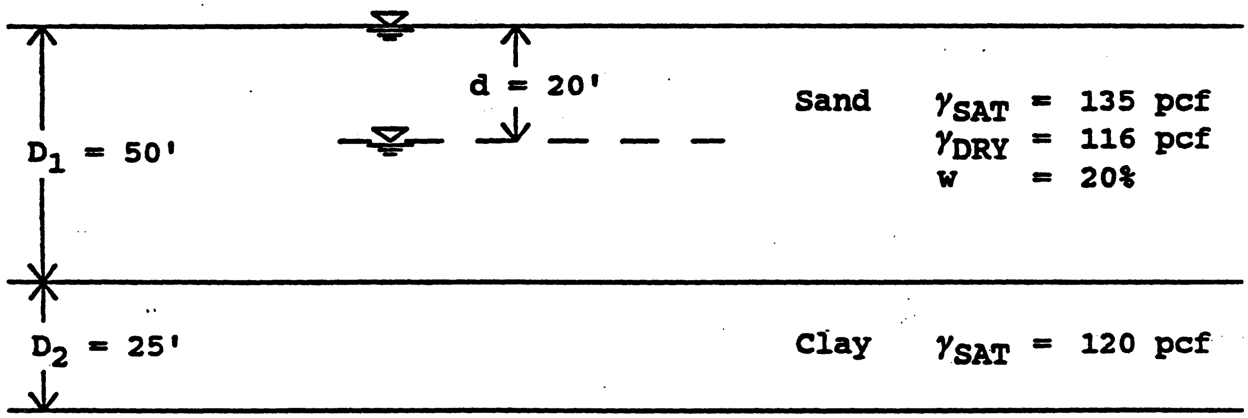
$P_2 = P_1 + K_a\gamma_b h_2 = 380 + (0.345)(66)(6) = 517$ psf

$P_w = (62.4)(6) = 374$ psf

LOWERED WATER TABLE

When the water table is lowered more load is transmitted to the underlying soil. This is due to the fact that the water's bouyant force on the soil particles is removed when the water table is lowered.

EXAMPLE PROBLEM



Given: Water surface originally at ground surface.
Water surface is lowered 20 Ft.

Find: Pressure at the center of the clay layer.

Solution:

Initial

$$P = D_1(\gamma_{SAT} - \gamma_{WATER}) + (D_2/2)(\gamma_{SAT} - \gamma_{WATER})$$

$$= 50(135 - 62.5) + (25/2)(120 - 62.5) = \quad 4344 \text{ psf}$$

Lowered

$$P = d\{(\gamma_{DRY} + 0.2(\gamma_{SAT} - \gamma_{DRY})\} + (D_1 - d)(\gamma_{SAT} - \gamma_{WATER}) + (D_2/2)(\gamma_{SAT} - \gamma_{WATER})$$

$$= 20\{116 + 0.2(135 - 116)\} + 30(135 - 62.5) + 12.5(120 - 62.5)$$
$$= 5290 \text{ psf}$$

$\Delta P = 5290 - 4344 = 946$ psf increase

Check: $\Delta P = 20\{62.5 - 0.8(135 - 116)\} = 946$ psf

SPECIAL CONDITIONS

PIPING

For excavation in pervious materials (sands), the condition of piping can occur when an unbalanced hydrostatic head exists. This causes large upward flows of water through the soil and into the bottom of the excavation. Material will be transported, which, if allowed to continue, will cause settlement of the soil adjacent to the excavation. This is also known as a sand boil or a quick condition. The passive resistance of embedded members will be reduced.

To correct this problem, either equalize the unbalanced hydraulic head by allowing the excavation to fill with water or lower the water table outside the excavation by dewatering.

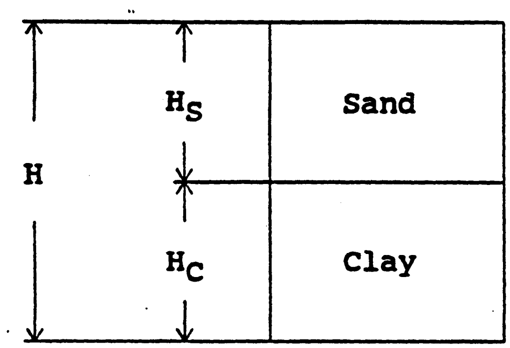
If the embedded length of the shoring system member is long enough, the condition of piping should not develop. Charts giving lengths of sheet pile embedment which will result in an adequate factor of safety against piping shown on page 65 of the USS Steel Sheet Piling Design Manual. These charts are of particular interest for cofferdams constructed of sheet piling.

STRATIFIED LAYER CONVERSION

Mixed layers of soil can be converted to an approximate: equivalent: clay type soil by use of the equations given below. These equations merely convert the soil to an equivalent clay based on the weighted averages of the individual layers. Note that this approximationmay result in a total horizontal pressure which is less than that calculated by the trial wedge method.

Do not use this method when there is a clay layer on top. In this situationmake a separate calculation for the top layer, then use an equivalent soil for the remainder of the depth. Another acceptable method which will take care of any situation is the semi-graphical trial wedge.

Case I (sand over clay)



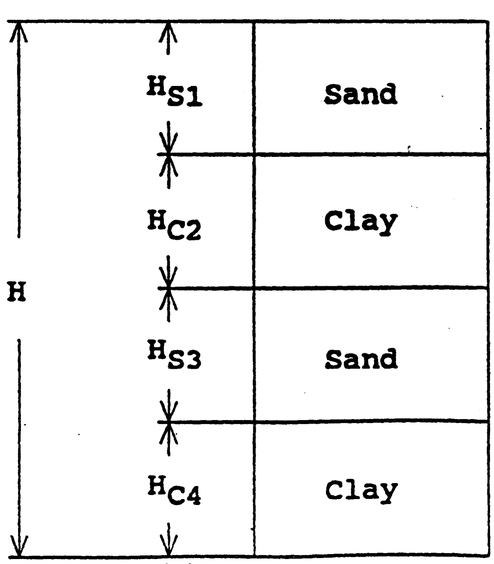
Equivalent Clay

$q = (K_S \gamma_S H_S^2 \tan \phi + H_C n q_u)/H$

$\gamma = (\gamma_S H_S + H_C \gamma_C)/H$

(See Note Below)

Case II (multiple layers, sand on top)



Use $H' = K_S H_S^2 \tan \phi$

$q = [\gamma_1 H'_1 + \gamma_2 H_{C2} + \gamma_3 H'_3 + \gamma_4 H_{C4}]/H$

$\gamma = [\gamma_1 H_{S1} + \gamma_2 H_{C2} + \gamma_3 H_{S3} + \gamma_4 H_{C4}]/H$

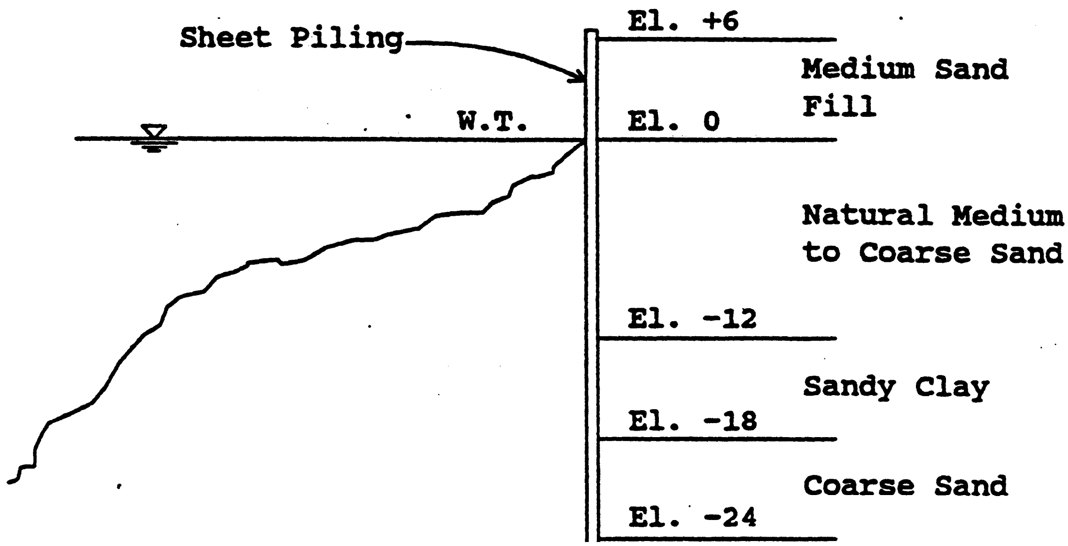
Note: For practical purposes K_S and n can be assumed to be equal to 1.0.

Subscripts S and C refer to sand and clay.

For sand $q_u = 0$

EXAMPLE PROBLEM - SHEET PILING USING STRATIFIED SOIL.

This sheet piling problem shows the effect on horizontal pressure after the sandy clay layer is drained; see inital and final pressure diagrams. Two pressures exist at each soil boundary elevationbecause a different K value is used for each soil type.



Assume

G = 2.65 for all soils.

P_V = Vertical Pressure

P_A = Active Horizontal Pressure

$P_A = P_V \tan^2(45° - \phi/2) - 2C\tan(45° - \phi/2)$

$\gamma_{eff} = (1 + w)\gamma dry$

Void Ratio = $e = (G\gamma_w/\gamma_{dry}) - 1$, $\gamma_w = 62.4$

W = water content

Layer	γ_{dry}	w	γ_{eff}	$\phi°$	e	C
Sand Fill:	100	10.0%	110	28		
Natural Sand:	102	24.5%	127	28	0.62	
Sandy Clay:	83	37.0%	114	16	0.99	275
Coarse Sand:	105	21.6%	128	36	0.575	

Submerged Weight = $\gamma_{SUB} = (G - 1)\gamma_w/(1 + e)$

Natural Sand: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.62) = 63.6$ pcf

Sandy Clay: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.99) = 51.7$ pcf

Coarse Sand: $\gamma_{SUB} = (1.65)(62.4)/(1 + 0.575) = 65.4$ pcf

CALCULATION OF PRESSURES

No Fill Condition (See diagram A on next sheet)

El. 0 $P_A = 0$ psf
El. -12 $P_A = 12(63.6)\tan^2(45° - 28°/2) = 276$ psf

El. -12 $P_A = 12(63.6)\tan^2(45° - 16°/2) - 2(275)\tan(45° - 16°/2)$
$= 19$ psf
El. -18 $P_A = \{12(63.6) + 6(51.7)\}\tan^2(45° - 16°/2)$
$- 2(275)\tan(45° - 16°/2)$
$= 195$ psf

El. -18 $P_A = \{12(63.6) + 6(51.7)\}\tan^2(45° - 36°/2) = 279$ psf
El. -24 $P_A = \{12(63.6) + 6(51.7) + 6(65.4)\}\tan^2(45° - 36°/2)$
$= 381$ psf

Drained Condition (See diagram B on next sheet)

El. +6 $P_A = 0$ psf
El. 0 $P_A = 6(110)\tan^2(45° - 28°/2) = 238$ psf

El. 0 $P_A = 6(110)\tan^2(45° - 28°/2) = 238$ psf
El. -12 $P_A = \{6(110) + 12(63.6)\}\tan^2(45° - 28°/2) = 514$ psf

El. -12 $P_A = \{6(110) + 12(63.6)\}\tan^2(45° - 16°/2) -$
$2(275)\tan(45° - 16°/2)$
$= 394$ psf
El. -18 $P_A = \{6(110) + 12(63.6) + 6(51.7)\}\tan^2(45° - 16°/2)$
$- 2(275)\tan(45° - 16°/2)$
$= 570$ psf

El. -18 $P_A = \{6(110) + 12(63.6) + 6(51.7)\}\tan^2(45° - 36°/2)$
$= 450$ psf
El. -24 $P_A = \{6(110) + 12(63.6) + 6(51.7) + 6(65.4)\}$
$\tan^2(45° - 36°/2)$
$= 552$ psf

Immediately after the sand fill is placed an increase of 660 PSF overburden pressure (vertical) is applied to the soil and the water contained therein. For granular soils the increased water pressure quickly dissipates. Cohesive soils do not allow free flow of water, therefore for some period of time the pressure acting horizontally is equal to the vertical pressure, See diagram c on the next page.

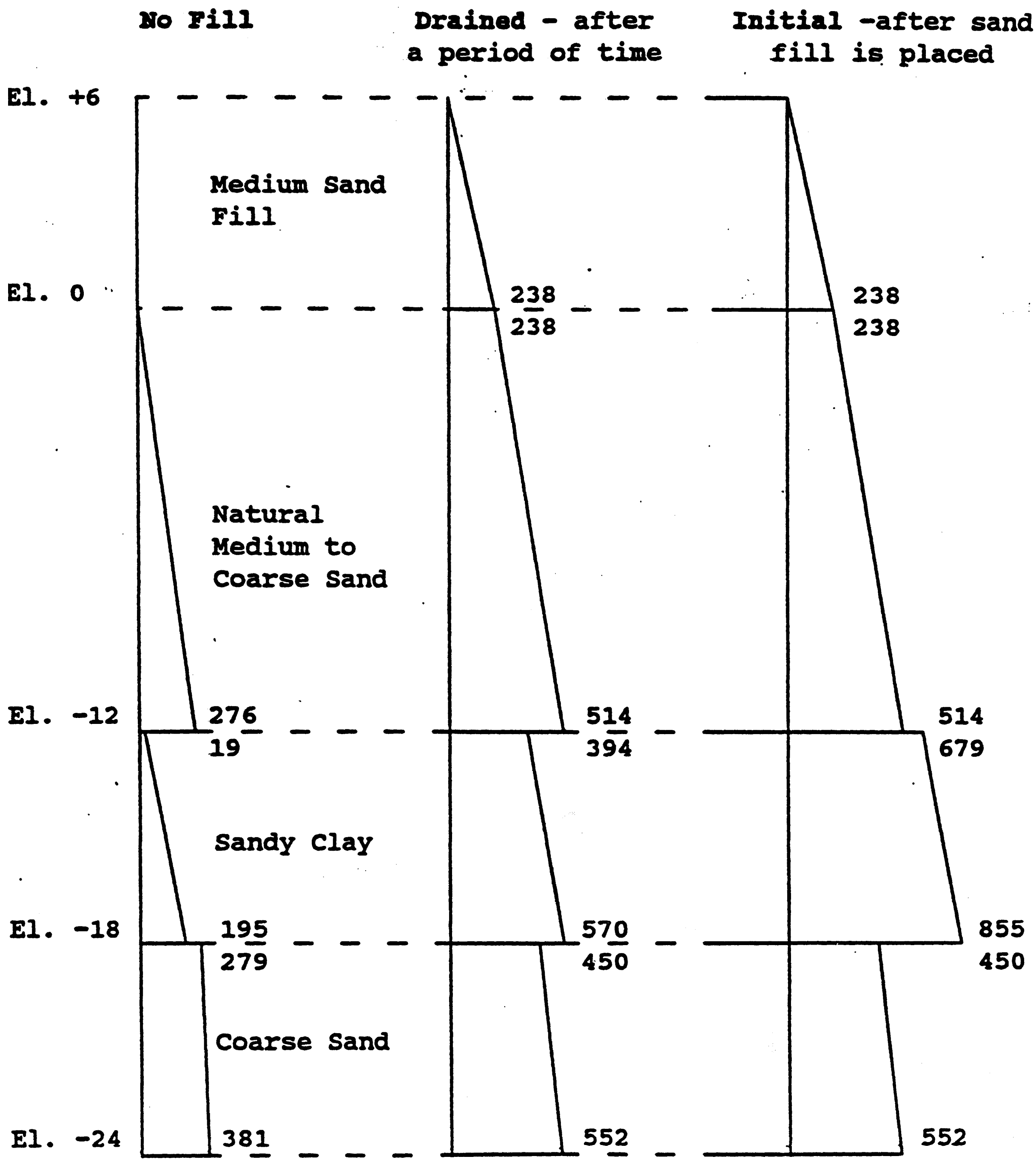
SPECIAL CONDITIONS

Initial Fill Condition (diagram, C)

Pressure in Clay Layer. Immediatly After Sand Fill is Placed.

El. −12 P_A = 6(110) + 19 = 679 psf

El. −18 P_A = 6(110) + 195 = 855 psf



Diagram

SLOPE STABILITY

The most critical failure surface will be dependent on site geology and is not necessarily circular. Non-circular failure surfaces can be caused by adversely dipping bedding planes, zones of weak soil or unfavorable ground water conditions. Circular solutions to slope stability have been developed primarily because of the ease this geometry lends to the computational procedure.

Two approximate methods used for investigating the factor of safety for potential stability failure are:

'Fellenius Method of Slices'
'Simplified Bishop Method of Slices'

The basic equation for each of these methods is:

$$F = \{\bar{C}\,L + \tan\bar{\phi}\sum_{i=1}^{i=n}\bar{N}_i\}/\{\sum_{i=1}^{i=n} W_i \sin\theta_i\}$$

Nomehclature

F = Factor of safety

F_a = Assumed factor of safety

i = Represents the current slice

$\bar{\phi}$ = Friction angle based on effective stresses

C = Cohesion intercept based on effective stresses

W_i = Weight of the slice

Ni = Effective normal force

θ_i = Angle from the horizontal of a tangent at the center of the slice along the slip surface

T_i = Tensile force

u_i = Pore-water pressure force on a slice

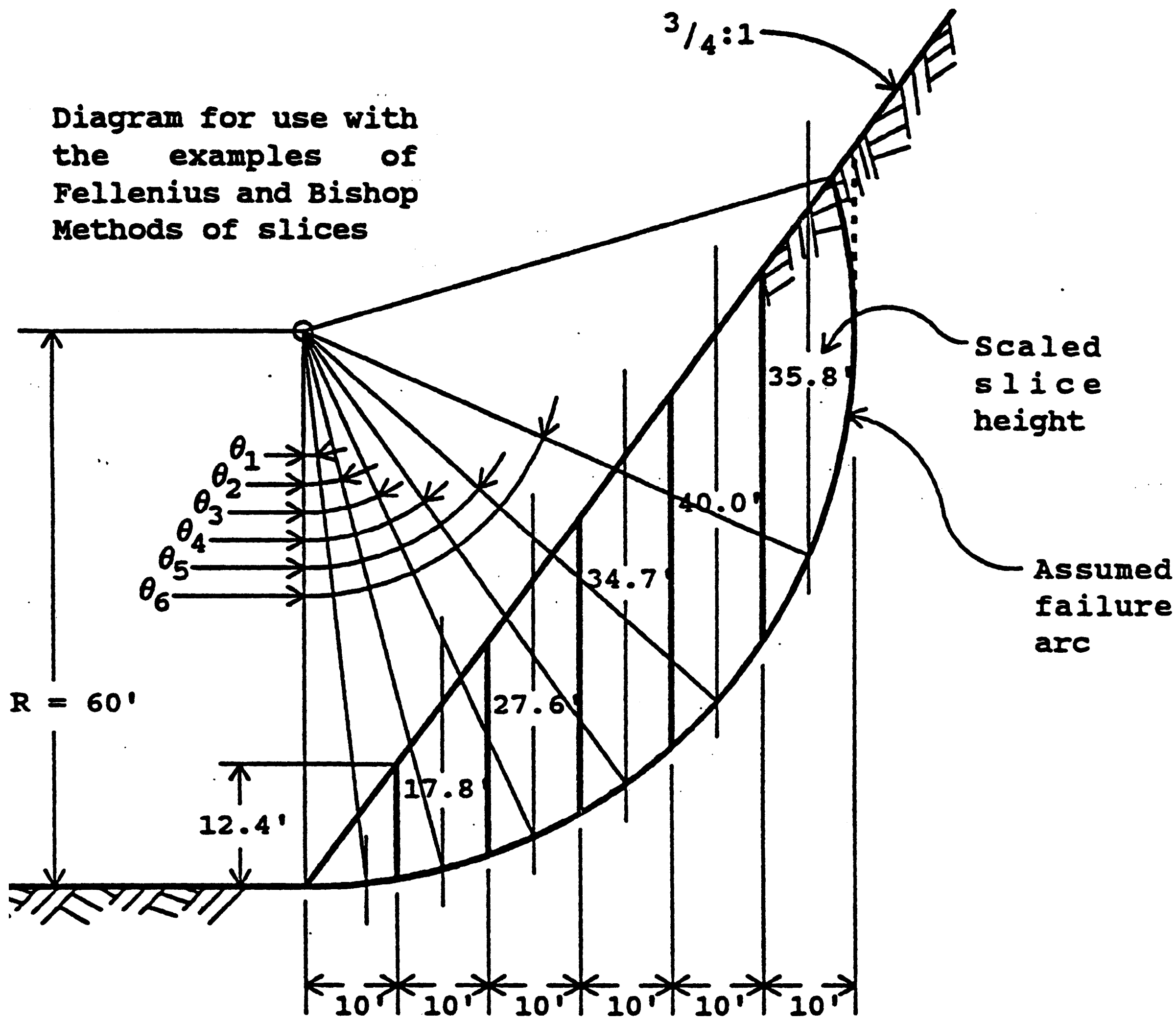
u_i = Resultant neutral (pore-water pressure) force

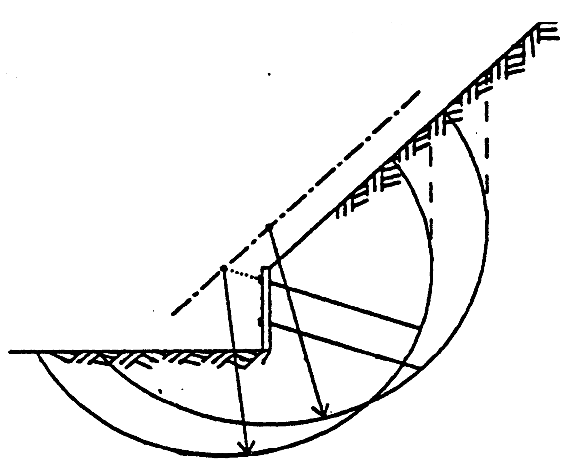
Δl_i = Length of the failure arc cut by the slice

L = Length of the entire failure arc

SPECIAL CONDITIONS

For major excavations in side slopes, slope stability failure for the entire system should be investigated.



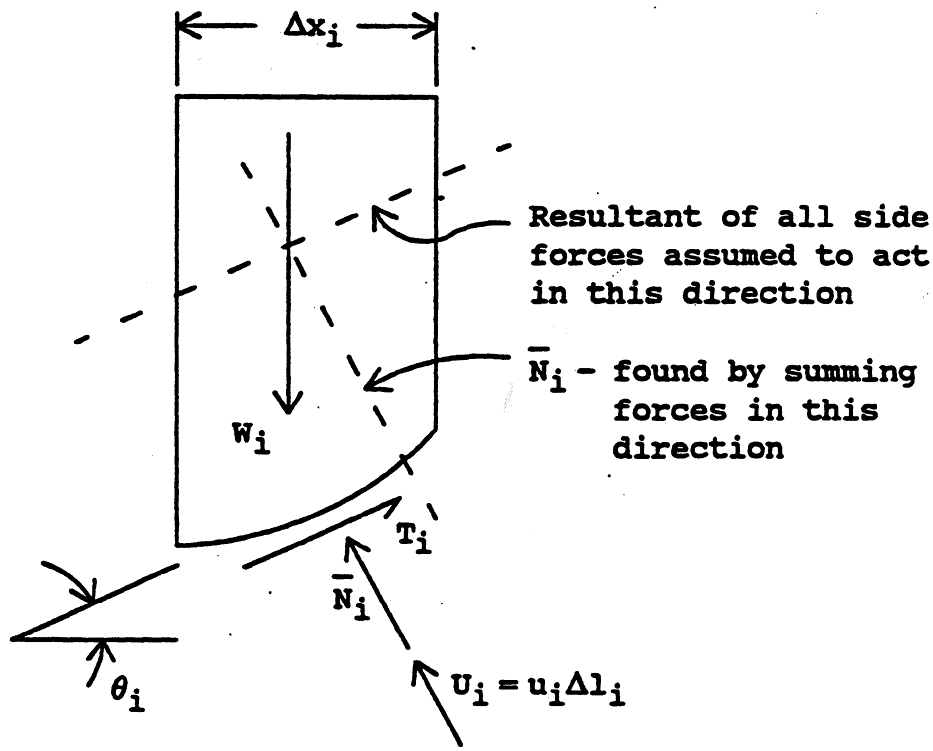


For major tie-back systems in other than optimum soils (as cohesive) over-all system failure should be investigated.

Minimum trial radii extend to end of ties; centers are on a line parallel to the slope.

FELLENIUS METHOD

Also known as 'Ordinary Method of Slices' or 'Swedish Circle'. This method assumes that for any slice, the forces acting upon its sides has a resultant of zero in the direction normal to the failure arc. This method errs on the safe side, but is widely used in practice because of its early origins and simplicity.



$$\bar{N}_i = W_i \cos\theta_i - u_i \Delta l_i$$

The basic equation becomes:

$$F = \{\bar{C}\, L + \tan\bar{\phi} \sum_{i=1}^{i=n} (W_i \cos\theta_i - u_i \Delta l_i)\} / \{\sum_{i=1}^{i=n} W_i \sin\theta_i\}$$

The procedure is to investigate many possible failure planes, with different centers and radii, to zero in on the most critical.

SAMPLE PROBLEM No. 20 - FELLENIUS METHOD OF SLICES

GIVEN:

γ = 115 pcf $\bar{\phi}$ = 30°   C = 200 psf   No groundwater

SOLUTION:

Scaled dimensions from graphic layout are satisfactory.

Angles

$\theta_1 = \sin^{-1}(6.7/60.0) = 6.41°$
$\theta_2 = \sin^{-1}(15.0/60.0) = 14.47$
$\theta_3 = \sin^{-1}(25.0/60.0 = 24.62$
$\theta_4 = \sin^{-1}(35.0/60.0) = 35.69$
$\theta_5 = \sin^{-1}(45.0/60.0) = 48.59$
$\theta_6 = \sin^{-1}(55.0/60.0) = 66.44$

Slice Weights

$\gamma = 0.115$ kcf
$W_1 = (1/2)(12.4)(10)(0.115) = 7.13k$
$W_2 = (17.8)(10)(0.115) = 20.47$
$W_3 = (27.6)(10)(0.115) = 31.74$
$W_4 = (34.7)(10)(0.115) = 39.91$
$W_5 = (40.0)(10)(0.115) = 46.00$
$W_6 = (35.8)(10)(0.115) = 41.17$

Slice	θ_i (°)	W_i (kips)	$W_i \sin\theta_i$	$W_i \cos\theta_i$	$\bar{N}_i$
1	6.41	7.13	0.80	7.09	7.09
2	14.47	20.47	5.11	19.82	19.82
3	24.62	31.74	13.22	28.85	28.85
4	35.69	39.91	23.28	32.41	32.41
5	48.59	46.00	34.50	30.43	30.43
6	66.44	41.17	37.74	16.46	16.46
			Σ=114.66		Σ=135.06

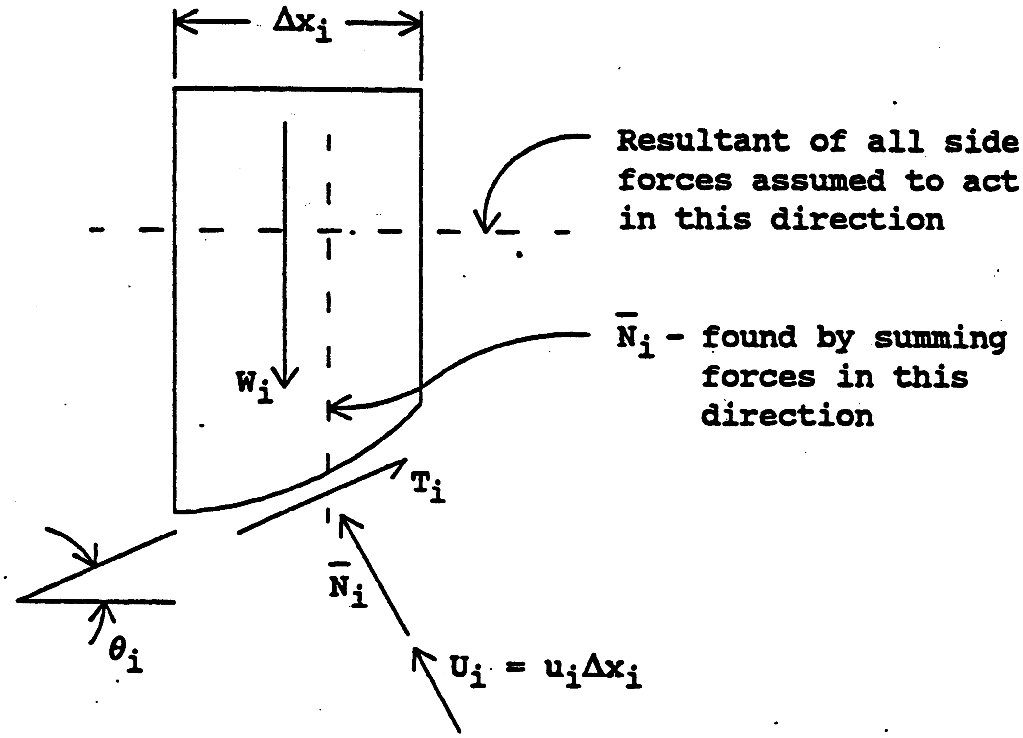
$U_i = 0$ L = 111.9'
(found by geometry & scaling)

$$F = \{(0.2)(111.9) + (0.577)(135.06)\}/114.66$$
$$= 0.87 < 1$$

This is the value for one trial failure plane. Additional trials are necessary to determine the critical one which gives minimum factor of safety. The slope for this sample problem is deemed to tie unstable since he computed safety factor determined by this single calculation is less than one.

BISHOP METHOD

This method, assumes that the forces acting on the sides of tiny slice have a zero resultant in the vertical direction.



$$\bar{N}_i = \{W_i - u_i\Delta x_i - (1/F_a)\bar{c}\Delta x_i \tan\theta_i\}/\cos\theta_i\{1 + (\tan\theta_i \tan\bar{\phi}\}/F_a$$

The basic equation becomes:

$$F = \{\sum_{i=n}^{i=n} (\bar{c}\ \Delta x_i + (W_i - u_i\Delta x_i)\tan\bar{\phi})(1/M_i)\}/\{\sum_{i=n}^{i=n} W_i \sin\theta_i\}$$

Where $\quad M_i = \cos\theta_i\{1 + (\tan\theta_i \tan\bar{\phi}/F_a)\}$

For Bishop Method, the Factors of Safety (F_a) must be assumed and a trial and error solution is required. The assumed "F_a's" converge on the Factor of safety for that trial failure plane. Good agreement between the assumed "F_a" and the calculated "F" indicates the selection of center and radius was good.

SAMPLE PROBLEM No. 21 - BISHOP METHOD

GIVEN:

Same as the previous example.

SOLUTION:

Column	A	B	C	D	E	F	G
Slice	θ_i	W_i	$\bar{C}\Delta x_i$	$W_i \tan\bar{\phi}$	$\cos\theta_i$	$\tan\theta_i \tan\bar{\phi}$	C + D
1	6.41	7.13	2.0	4.12	0.99	0.06	6.12
2	14.47	20.47	2.0	11.82	0.97	0.15	13.82
3	24.62	31.74	2.0	18.33	0.91	0.26	20.33
4	35.69	39.91	2.0	23.04	0.81	0.41	25.04
5	48.59	46.00	2.0	26.56	0.66	0.65	28.56
6	66.44	41.17	2.0	23.77	0.40	1.32	25.77

Column	Ha	Hb	Ia	Ib	J
Slice	M_i		G÷Ha	G÷Hb	$W_i \sin\theta_i$
	F_a=1.5	F_a=0.8	F_a=1.5	F_a=0.8	
1	1.04	1.07	5.94	5.72	0.80
2	1.06	1.15	12.92	12.02	5.11
3	1.07	1.21	19.00	16.94	13.22
4	1.04	1.23	24.31	20.36	23.28
5	0.95	1.20	30.06	23.80	34.50
6	0.75	1.06	34.36	24.31	37.74
			Σ=126.59	Σ=103.15	Σ=114.65

For $F_a = 1.5$

$F = 126.59/114.65 = 1.104$

For $F_a = 0.8$

$F = 103.15/114.65 = 0.900$

The factor of safety for this trial converges to ≈ 0.9.

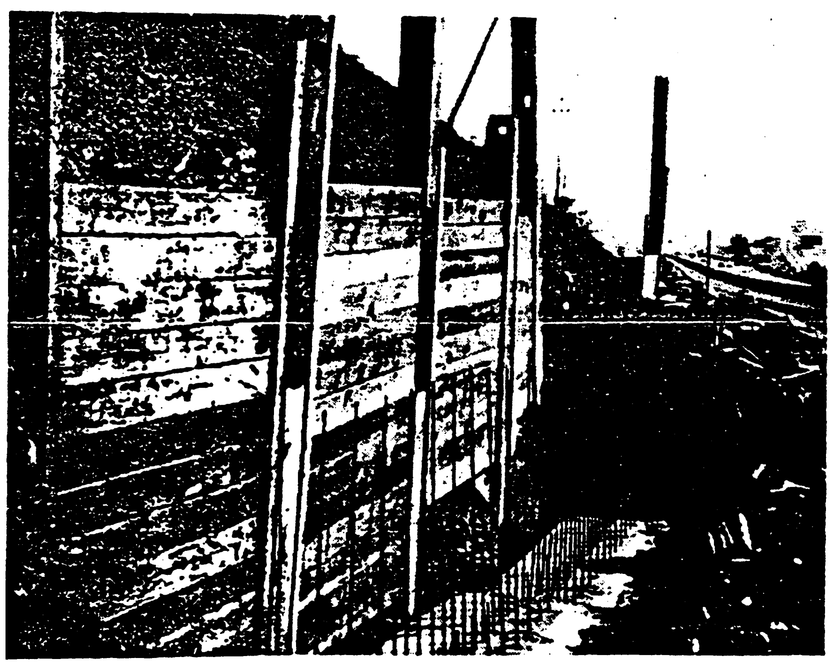
Again, this is the value for one trial failure plane. Additional trials are necessary to determine the critical one which gives minimum factor of safety.

If groundwater was present pore pressure would need to be considered. These values are most typically field measured.

The foregoing slope stability presentation serves to demonstrate the complexity of stability analysis. Soil failure analysis should not be limited to circular arc solutions. There are a number of computer programs for slope stability analysis using non-circular shapes. Slope stability analysis is most properly within the realm of geotechnical engineering.

When it appears that shoring or a cut slope presents a possibility of some form of slip failure, a stability analysis should be requested. In addition, the Transportation Materials and Research Laboratory in Sacramento has the capability of performing computer aided stability analysis to verify the submitted analysis.

Submittals relative to soils data and analysis should be from a recognized soils lab or from a qualified Geotechnical Engineer or Geologist.



HYDRAULIC FORCES ON COFFERDAMS AND OTHER STRUCTURES

Moving water imposes drag forces on obstructions in waterways. The drag force in equation form (after Ratay) is:

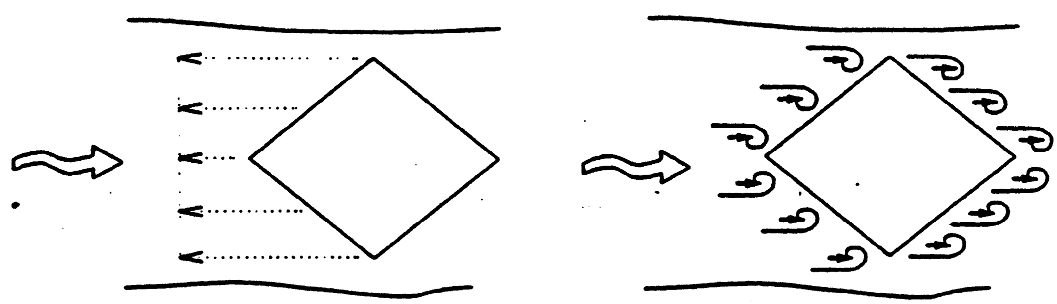
$$F_z = \rho(A)(C_d)(V^2/2g)$$

Where: ρ = Water density.
A = Projected area of the obstruction normal to the current.
C_d = Coefficient of drag.
V = Velocity of the current.
g = Acceleration due to gravity.

In english units $\rho \approx 2g$ so that:

$$F_z = A(C_d)(V^2)$$

Where: A = Ft^2
C_d = pounds
V = ft/sec.



Considering roughness along the sides of the obstructions (as for a sheetpile cofferdam the practical value for c_d = 2.0.

$$F_z = 2AV^2$$

Which may be considered to be applied in the same manner as a wind rectangular load on the loaded height of the obstruction.

Example: Determine the drag force on a six foot diameter corrugated metal pipe placed vertically in water of average depth of 6 feet flowing at 4 feet per second.

Projected Area = 6(6) = 36 Ft^2.

F_Z = 2{36)$(4)^2$ = 1,152 Lbs.

CONSTRUCTION CONSIDERATIONS

CONSTRUCTION CONSIDERATIONS

CONSTRUCTION

The integrity of a shoring system, like any other structure, is dependent on the quality of the actual construction as well as the adequacy of the design. Frequent and thorough inspection of the excavation and the shoring system during all stages of construction must be performed by qualified personnel. An awareness of changing conditions is essential.

1. Prior to the beginning of excavation work, become familiar with all aspects of the approved plans, location of the work, assumptions made, available soils data, groundwater conditions, surcharge loads expected, sequence of operations, location of utilities and underground obstructions, and any other factors restricting the work at the site.

2. Since the primary function of shoring is the protection of the workmen and adjacent property, it is essential that the inspector be knowledgeable of the minimum safety requirements.

3. Check the soil being excavated to see that it is the same material as anticipated.

4. Check for changes in ground water conditions.

5. As the excavation progresses be alert for indicators of distress such as cracking of members or subsidence near the excavation.

6. If the excavation is sloped back, without shoring, the need for inspection remains. Sloughing and cave-ins can occur.

7. For shored excavations, check the shoring members for size and spacing as shown on the approved plans. Any sequence of operations shown on the plans must be followed. Check for full bearing at ends of jacks and struts and make sure that they are secure and will not fall out under impact loads. Also check members for bending, buckling, and crushing.

8. Manufactured products, such as hydraulic struts, jacks, and shields should be installed and used according to the manufacturer recommendations.

9. If a tieback system is used, the tiebacks should be placed as per approved plan and preloaded to avoid overloading individual ties. If cables are used as tiebacks, they should not be wrapped around sharp corners. Thimbles should be used and cable clamps installed properly.

10. Surcharge loads need to be monitored to verify that such loads do not exceed the design assumptions for the system.

11. Weather conditions may have adverse effects on excavations and some materials, especially clays, may fail due to change in moisture content.

12. Good workmanship makes an excavation safer and easier to inspect, Trouble spots are easier to detect when the excavation is uniform and straight.

13. Vibratory or dynamic loadings from pile driving or blasting operations require special attention to soil or shoring.

14. Utility owners should be notified prior to commencement of work if their facilities are within 5 times the excavation depth.

Underground Service Alert

Northern California (USA)	l-800-642-2444
Southern California (USA)	l-800-422-4133

South Shore Utility

Coordinating Council (DIGS)	l-800-541-3447

15. Encourage the use of benchmarks to monitor the shoring system before, during, and after ground movements in the vicinity of excavations (within a distance of 10 times the excavation depth). Ground settlements accompany shoring deflections.

ALLOWABLE WORKING STRESSES

Timber

Construction Safety Orders defining lumber and allowable stresses are included in the Appendix C to Section 1541.1 (See Appendix A of this manual). Member substitution for shoring systems to be used in conjunction with timber tables of Appendix C to Section 1541.1 requires that they be manufactured members of equivalent strength. Some alternate crossbracing manufactured members are shown in Appendix E to Section 1541.1 of the Safety Orders (See Appendix A of this manual).

Briefly, except for scaffold plank, Select or Douglas Fir l equivalent lumber or timber shall be suitable for 1,5OO psi bending stress l The tables for timber shoring in Appendix C to Section 1541.1 of the Safety Orders permit an allowable bending stress of not less than 850 psi for mixed oak or equivalent wood.

When shoring plans designed by a qualified engineer do not specify stress limitations or list type of lumber (timber) OSC will review the plans assuming Douglas Fir Larch (North) Group II with the following stress limitations:

$F_c = 480{,}000\ (L/D)^2$ psi	Compression Parallel to Grain Not to exceed 1,600 psi
F_b = 1,800 psi	Flexural (bending) Reduced to 1,500 psi for members with a nominal depth of 8 inches or less.
F_t = 1,200 psi	Direct Tension
$F'_\perp$ = 450 psi	Compression Perpendicular to Grain
V = 140 psi	Horizontal Shear
E = 1.6×10^6 psi	Modulus of elasticity Use 1.2×10^6 psi for wet or green-timber
Overstress:	Permit 33% overstress for short term loadings (see exceptions in "Shoring General Procedure," Chapter 5).

Lesser stress values shown on the shoring plans or in the accompanying calculations will be used for review.

When lumber (timber) type is listed or shown on the shoring plan without allowable stress values the "National Design Specification For Wood Construction" will be used as a guide. If the specific, lumber grading is not included, low allowable stress values will be used.

Railroads allow 1,700 psi maximum in lieu of l,800 psi for flexural stress. Shoring adjacent to railroads is to be designed and reviewed in accordance with railroad requirements. Specific railroad requirements are included in Appendix C.

Steel

Refer to current AISC specifications. If grade of steel is unknown, Use A36 (F_y = 36 ksi, E = 30 x 10^6).
Steel sheet piling: use Grade A328 steel for which F_b = 25 ksi, unless specific informaton is furnished for higher grade steel.

Aluminum

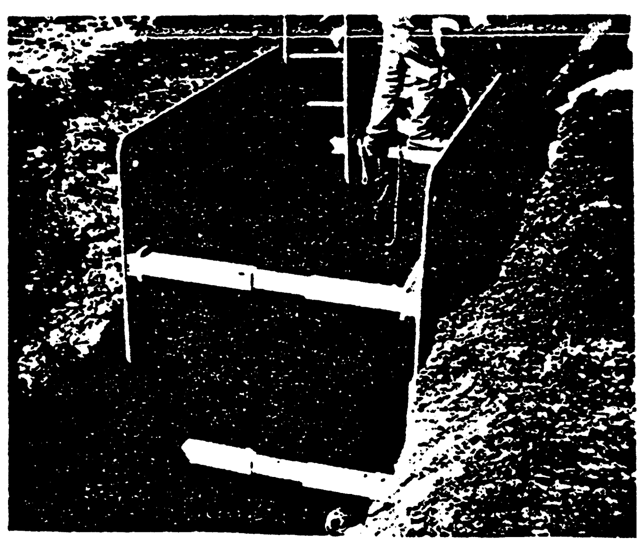
Refer to current aluminum design references.

Concrete

Use current structural concrete design criteria.

Connectors

For timber connectors use the current National Design Specification for Wood Construction, National Forest Products Association. OSC allows the single shear value to be 0.75 of the NDS tabular value for double shear connection (in lieu of 0.50).
For steel, refer to current AISC Specifications.



MECHANICS OF STRESS ANALYSIS

Use the accepted structural mechanics formulas and theories. Check members of the shoring system for flexure, shear, compression, and bearing. Check the system (with soil) for stability. Approximate calculations are satisfactory for most shoring systems.

Common structural mechanics formulas:

Flexural stress (bending)	$f_b = M/S$	M = Bending Moment S = Section Modulus
Axial Compression	$f_C = P/A$	P = Applied Load A = Area of Member
Timber		
Compression ⊥ to grain	$f_{\perp} = P/A$	
Horizontal Shear	$V = (1.5)w(L/2 - b/2 - d)/A$	

L = span length (center to center)
b = thickness of supporting member or length of bearing area, whichever is less
d = depth of member for which shear is being investigated.
W = unit load

For lagging use simple span moments. Multiply all loads by 0.6 to account for soil arching. $M = 0.6wL^2/8$

In many cases the effective span for lagging will be less than the spacing of supports.

For interior moments of uniformly loaded continuous uprights, walers, or rails, $M = wL^2/10$ may be used.

For cantilevers: M = wL2/2

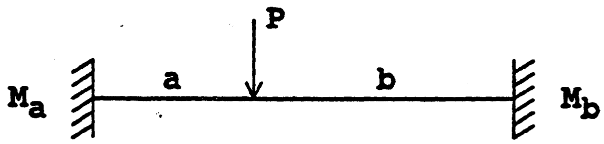
The Design Earth Pressure Diagram will be the sum of the basic earth pressure, surcharge loads, and any other applicable loads (such as ground water).

Since calculating earth pressures is not precise, an irregular-shaped composite diagram may be simplified by using standard geometrical shapes (rectangles, triangles, etc.).

LOADING DIAGRAMS

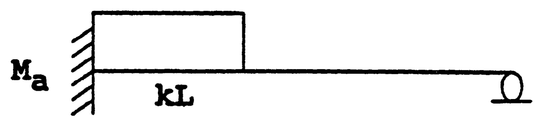
An approximation is made when triangular loads are converted to concentrated loads. For an exact solution use the following formulas to calculate fixed end moments:

$M_a = Pab^2/L^2$

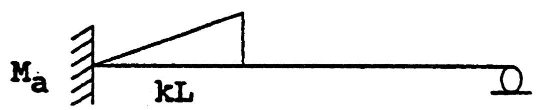


$M_b = Pa^2b/L^2$

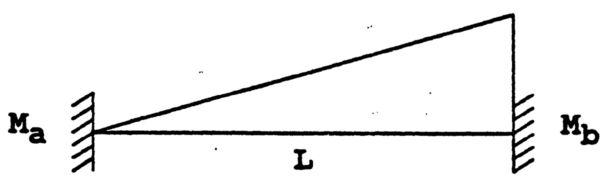
$M_a = wL^2k^2(4 - 4k + k^2)/8$



$M_a = wL^2k^2(40 - 45k + 12k^2)/120$

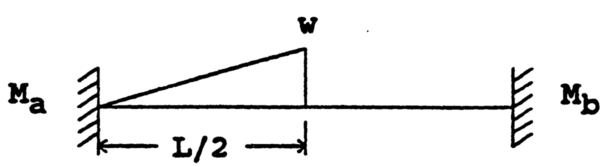


$M_a = wL^2/30$



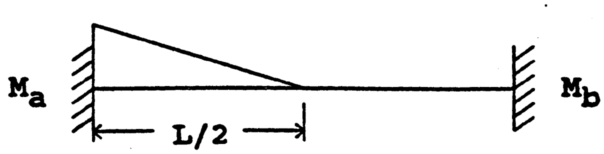
$M_b = wL^2/20$

$M_a = wL^2/30$



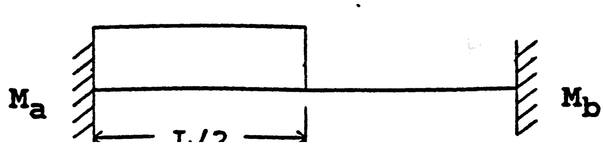
$M_b = 3wL^2/160$

$M_a = 23wL^2/960$

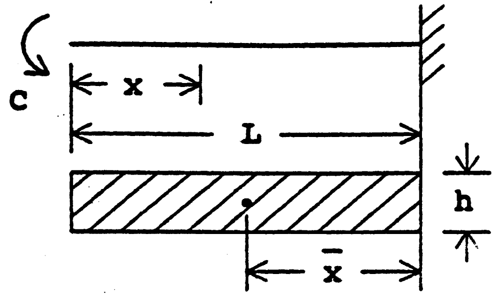
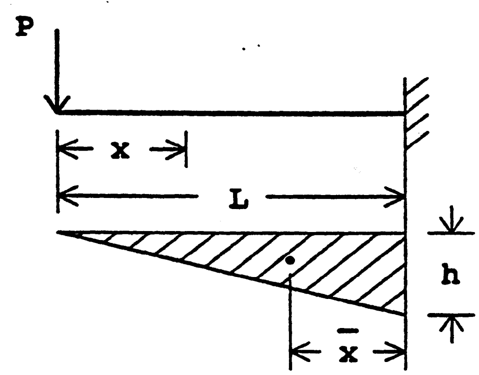
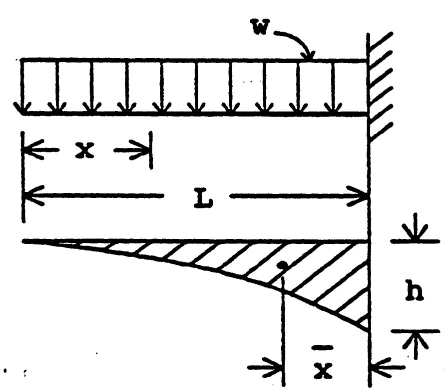
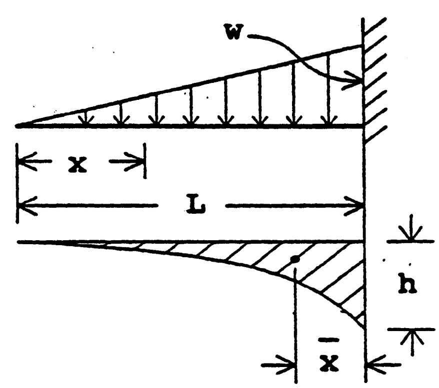


$M_b = 7wL^2/960$

$M_a = 11wL^2/192$



$M_b = 5wL^2/192$

TYPE OF LOADING	CANTILEVER BEAM	EQUATIONS
Couple		$M = -C$ $Area = Lh$ $\bar{x} = L/2$ $h = -C$
Concentrated		$M = -Px$ $Area = Lh/2$ $\bar{x} = L/3$ $h = -PL$
Uniformly Distributed		$M = -wx^2/2$ $Area = Lh/3$ $\bar{x} = L/4$ $h = -wL^2/2$
Uniformly Varying		$M = -wx^3/6L$ $Area = Lh/4$ $\bar{x} = L/5$ $h = -wL^2/6$

ENCROACHMENT PERMIT PROJECTS

This category of work includes shoring for projects by others, either within the State highway right of way, or adjacent to State highways. The work is done under an Encroachment Permit issued to the Contractor, Builder and/or Owner by the District Permits Engineer,

Usually a condition of a permit is that appropriate plans of the proposed shoring system be prepared and submitted to the State Permits Engineer for review and approval before work may be started.

Many of the encroachment permit projects are quite simple, like pipe trenches of nominal depth which may conform to the Standard DOSH Details. However, complex shoring systems will be needed for large building excavations, multiple tier tie back systems, etc.

The District Permits Engineer, on receipt of an implication for an encroachment permit, will decide if he needs technical assistance to review the plans. The Permits Engineer may ask District Construction to complete the review if the conditions are such that the Standard DOSH Details may apply and the plans conform to these Details.

For more complex shoring systems, District Construction or the Permits Engineer may request technical review assistance from a structures person. For a very complex major shoring project the Permits Engineer will usually route the plans to the Office of Structures Maintenance for review. Structures Maintenance, in turn, may then forward the plans either (or both) to Structure Design or Office of Structure Construction for structural review. Encroachment Permit shoring plan review is an ongoing engineering responsibility.

The inspection of the actual work in the field is handled in a similar way. Intermittent inspection may be handled by the District. For average or simple projects, this is usually on an informal basis.

For major encroachment permit projects the District may request that OSC assign an Engineer as a representative of the District Permits Engineer.

A part of the field review or monitoring will be to see that the Contractor and/or owner have the proper permits.

If it is necessary to request a Contractor to make a major correction or improvement of work during the course of construction, remember that the administrative or control procedure is different from State

CONSTRUCTION CONSIDERATIONS

Construction Contracts. The field reviewer is a representative of the Permits Engineer, not the Resident Engineer. If there are difficulties the Permits Engineer can always withdraw an Encroachment Permit, which would have the effect of stopping work. It is suggested that discussions be held with the area structure construction engineer.

Note that consultants who prepare shoring -plans for Encroachment Permit projects do not necessarily use the recommended allowable stresses given in this manual. In making a review, keep this in mind. Acceptance should be based on nothing less than that required for a State project, with due considerationbeing given to the background of the Contractor, the work to be done, and the degree of risk involved. Remember, geotechnical engineering is not an exact or precise science.

In order for the State to review and approve a Contractor's shoring system, a plan of work to be done must be submitted. As a minimum the shoring plan will contain the following information:

ENCROACHMENT PERMIT NO. (Contractor)

Contractor: (Name, address, phone)

Owner: For who the work is being done.
Include Contract No. or Designation.

OWNER ENCROACHMENT PERMIT NO.

Location: Road, street, highway stationing, etc. This shows the scope or extent of the project.

Purpose: Describe what the trench or excavation is for (24" sewer line, retaining wall, etc).

Soil Profile: A description of the soil including the basis- of identification; surface observation, test borings, observation of adjacent work in same type of material, reference to a soils investigation report, etc. In the absence of specific soils data the reviewer must assume very conservative values.

Include any observed ground water data.

Surcharge Loadings:

Any loads, including normal construction loads, that are adjacent to the excavation or trench should be identified and shown on the plans with all pertinent dimensions; examples are highways, railroads, existing structures, etc. The lateral pressures due to these loads will then be added to the basic soil pressures. The type of loading will also effect the type of shoring that can be accepted (an adjacent building will necessitate restriction of movement in the shoring system for example). The minimum surcharge is to be used where not exceeded by above loading considerations.

Trenching & Shoring Plan:

A complete description of the shoring system including all members, materials, spacing, etc. As most trenches are rarely uniform in depth and width, it might become necessary to average various sections. However, there are practical limits as to how much averaging is acceptable and it may be necessary to break the trench into smaller units which are similar in size.

Information may be in the form of a drawing, or referenced to the applicable portions of the Construction Safety Orders.

If a shoring system varies from Title 8 of the Safety Orders, then by law (California Code), a Professional Engineer (Civil or Structural) must prepare the shoring plans.

Refer to the order of work and/or traffic control plan if pertinent to the shoring system.

The plan for simple trench work can be in the form of a letter which covers the items required.

CONSTRUCTION CONSIDERATIONS

Manufactured Data:

Catalogs or engineering data for a product should be identified in the plan as supporting data. All specific items or applicable conditions must be outlined on the submittal (yellow or high lighting is one way to do this).

Construction Permit:

Any plan or information submitted should confirm that a permit has been secured from DOSH to perform the excavation work. This is not an approval of the shoring system by DOSH.

Inspection: Permit projects will require the same level of inspection that is used on contract work; watching for changing soil conditions, member overstress, potential shoring movement, etc.

The State Department of Transportation will review a Contractor's Shoring Plan in accordance with applicable State Specifications and the Construction Safety Orders, Deviations from DOSH or different approaches will be considered, providing adequate supporting data (calculations, soils investigations, manufacturer's engineering data and references) are submitted. The "CALIFORNIA TRENCHING AND SHORING MANUAL" will be used as a guide for plan review and approval.

CONCLUSIONS

The Department has an obligation with respect to trenching and shoring. work. Be informed of legal responsibilities and requirements (Refer to Chapter 1).

Soil Mechanics (Geotechnical Engineering) is not a precise science. Be aware of the of effects assumptions can make. Simplified engineering analysis procedures can be used for much of the trenching and shoring work that will be encountered.

The actual construction work is of equal importance to the engineering design or planning. The Contractor and the Engineer must always be alert to changed conditions and must take appropriate action. Technical assistance is available. The Engineer at the jobsite must be able to recognize when he needs help. The need for good engineering judgment is essential.

Work involving railroads requires additional controls and specific administrative procedures.

Following is a summary of D.0.T: policy in regard to trench and excavation shoring work:

1. The law (State Statute, Section 137.6) requires that a California registered professional engineer review the Contractor's plans for temporary structures in connection with State Highway work. Shoringplans areincluded in this category.

2. The Resident Engineer will ascertain that the Contractor has obtained a proper excavation or trenching permit from DOSH before any work starts, and that the permit (or copy) is properly posted at the work site.

3. If the trench is less than 20 feet deep and the Contractor submits a plan in accordance with the Construction Safety Order Details, it is not necessary to have the plans prepared by a Professional Engineer. The Resident Engineer will confirm that the Contractor's plan does indeed conform to the DOSH Details and need not make an independent engineering analysis.

CONSTRUCTION CONSIDERATIONS

4. If a trench is over 20 feet in depth, the DOSH Details cannot be used; the plans must be prepared by a Professional Engineer.

   When shoring plans are designed by firms specializing in temporary support systems and soil restraint (including sloping), good engineering judgement is to prevail for review. Shoring designed by such firms will often be less conservative than would shoring designed by conservative use of this manual.

6. If the Contractor shoring plan deviates from the Construction Safety Order Details, the plan must be prepared by a California registered professional engineer and the reviewing Engineer will make a structural analysis.

7. For any shoring work which requires review and approval by a Railroad; the Sacramento OSC Office will be the liaison between the project and the Railroad. The Structure Representative will submit the Contractor shoring plans to OSC Sacramento after review. The review should be so complete that the plans are ready for approval.

   The Structure Representative should inform the Contractor of the proper procedure, and the time, required for Railroad review and approval.

8. Any revisions to plans should be done by the plan originator or by his authorized representative. Minor revisions may be made on plans but the revisions should be initialed and dated by the person making the changes.

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DIVISION OF OCCUPATIONAL SAFETY AND HEALTH

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Sacramento	2424 Arden Way, Ste. 125, Sacramento 95825	Bill Krycia	(916) 263-2803
Anaheim	2100 East Katella Avenue, Ste. 125, Anaheim 92806	Tom Hanley	(714) 939-8611
Los Angeles	3550 West 6th St., Rm. 413, Los Angeles 90020	Robert Garcia	(213) 736-4911
DISTRICT			
Anaheim	2100 East Katella Avenue, Ste. 140, Anaheim 92806	James Brown	(714) 939-0145
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Fresno	2550 Mariposa St., Rm. 4000, Fresno 93721	Larry Baca	(209) 445-5302
Los Angeles	3550 West Sixth St., Rm. 431, Los Angeles 90020	Joan Lee	(213) 736-3041
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Santa Rosa	1221 Farmers Lane, Ste. 300, Santa Rosa 95405	P. Stan Bethel	(707) 576-2388
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Ventura	1655 Mesa Verde, Rm. 150, Ventura 93003	Bill Siener	(805) 654-4581
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APPENDIX A

CALIFORNIA OCCUPATIONAL SAFETY AND HEALTH STANDARDS FOR EXCAVATIONS

Article 2. Definitions

1504. Definitions

Competent Person: One who is capable of identifying existing and predictable hazards in the surroundings or working conditions which are unsanitary, hazardous, or dangerous to employees, and who has authorization to take prompt corrective measures to eliminate them.

Excavation, Trenches, Earthwork.

- **(A) Bank.** A mass of soil rising above a digging level.
- **(B) Exploration Shaft.** A shaft created and used for the purpose of obtaining subsurface data.
- **(C) Geotechnical Specialist (GTS).** A person registered by the state as a Certified Engineering Geologist, or a Registered Civil Engineer trained in soil mechanics, or an engineering geologist or civil engineer with a minimum of three years applicable experience working under the direct supervision of either a Certified Engineering Geologist or Registered Civil Engineer.
- **(D) Hard Compact (as it applies to Section 1542).** All earth materials not classified as running soil.
- **(E) Lagging.** Boards which are joined, side-by-side, lining an excavation.
- **(F) Running Soil (as it applies to Section 1542).** Earth material where the angle of repose is approximately zero, as in the case of soil in a nearly liquid state, or dry, unpacked sand which flows freely under slight pressure. Running material also includes lose or disturbed earth that can only be contained with solid sheeting.
- **(G) Shaft.** An excavation under the earth's surface in which the depth is much greater than its cross-sectional dimensions, such as those formed to serve as wells, cesspools, certain foundation footings, and under streets, railroads, buildings, etc.

Article 6. Excavations

1539. Permits.

For regulations relating to Permits for excavations and trenches, refer to the California Code of Regulations Title 8, Chapter 3.2, Article 2, Section 341 of the California Occupational Safety and health Regulations (Cal/OSHA).

1540. Excavations.

(a) Scope and application.

This article applies to all open excavations made in the earth's surface. Excavations are defined to include trenches.

(b) Definitions applicable to this article.

Accepted engineering practices means those requirements which are compatible with standards of practice required by a registered professional engineer.

Aluminum hydraulic shoring. A pre-engineered shoring system comprised of aluminum hydraulic cylinders (crossbraces) used in conjunction with vertical rails (uprights) or horizontal rails (walers). Such system is designed specifically to support the sidewalls of an excavation and prevent cave-ids.

Bell-bottom pier hole. A type of shaft or footing excavation, the bottom of which is made larger than the cross section above to form a belled shape.

Benching (Benching system). A method of protecting employees from cave-ins by excavating the sides of an excavation to form one or a series of horizontal levels or steps usually with vertical or near-vertical surfaces between levels.

Cave-in. The separation of a mass of soil or rock material from the side of an excavation, or the loss of soil from under a trench shield or support system, and its sudden movement into the excavation, either by falling or sliding, in sufficient quantity so that it could entrap, bury, or otherwise injure and immobilize a person.

Crossbraces. The horizontal members of a shoring system installed perpendicular to the sides of the excavation, the ends of which bear against either uprights or wales.

Excavation. Any man-made cut, cavity, trench, or depression in an earth surface, formed by earth removal.

Faces or sides. The vertical or inclined earth surfaces formed as a result of excavation work.

Failure. The breakage, displacement, or permanent deformation of a structural member or connection so as to reduce its structural integrity and its supportive capabilities,

Hazardous atmosphere. An atmosphere which by reason of being explosive, flammable, poisonous, corrosive, oxidizing, irritating, oxygen deficient, toxic, or otherwise harmful, may cause death, illness, or injury.

Kickout. The accidental release or failure of a cross brace.

Protective system. A method of protecting employees from cave-ins, from material that could fall or roll from an excavation face or into an excavation, or from the collapse of adjacent structures. Protective systems include support systems, sloping and benching systems, shield systems, and other systems that provide the necessary protection.

Ramp. An inclined walking or working surface that is used to gain access to one point from another, and is constructed from earth or from structural materials such as steel or wood.

Registered professional engineer. A person who is registered as a professional engineer in the state where the work is to be performed. However, a professional engineer, registered in any state is deemed to be a "registered professional engineer" within themeaning of this standard when approving designs for "manufactured protective systems" or "tabulated data" to be used in interstate commerce.

Sheeting. The members of a shoring system that retain the earth in position and in turn are supported by other members of the shoring system.

Shield (Shield system). A structure that is able to withstand the forces imposed on it by a cave-in and thereby protect employees within the structure. Shields can be permanent structures or can be designed to be portable and move along as work progresses, Additionally, shields can be either premanufactured or job-built in accordance with Section 1541.1(c)(3) or (c)(4). Shields used in trenches are usually referred to as "trench boxes" or "trench shields."

Shoring (Shoring system). A structure such as metal hydraulic, mechanical or timber shoring system that supports the sides of an excavation and which is designed to prevent cave-ins.

Sides. See "Faces."

Sloping (Sloping system). A method of protecting employees from cave-ins by excavating to form sides of an excavation that are-inclined away from the excavation so as to prevent cave-ins. The angle of incline required to prevent a cave-in varies with differences in such factors as the soil type, environmental conditions of exposure, and application of surcharge loads.

Stable rock. Natural solid mineral material that can be excavated with vertical sides and will remain intact while exposed. Unstable rock is considered to be stable when the rock material on the side or sides of the excavation is secured against caving-in or movement by rock bolts or by another protective system that has been designed by a registered professional engineer.

Structural ramp. A ramp built of steel or wood, usually used for vehicle access. Ramps made of soil or rock are not considered structural ramps.

Support system. A structure such as underpinning, bracing, or shoring, which provides support to an adjacent stucture, underground installation, or the sides of an excavation.

Tabulated data. Tables and charts approved by a registered professional engineer and used to design and construct a protective system.

Trench (Trench excavation). A narrow excavation (in relation to its length) made below the surface of the ground. In general, the depth is greater than the width, but the width of a trench (measured at the bottom) is not greater than 15 feet. If forms or other structures are installed or constructed in an excavation so as to reduce the dimension measured from the forms or structure to the side of the excavation to 15 feet or less, (measured at the bottom of the excavation), the excavation is also considered to be a trench.

Trench box. See "Shield."

Trench shield. See "Shield."

Uprights. The vertical members of a trench shoring system placed in contact with the earth and usually positioned so that individual members do not contact each other. Uprights placed so that individual members are closely spaced, in contact with or interconnected to each other, are often called "sheeting."

Wales. Horizontal members of a shoring system placed parallel to the excavation face whose sides bear against the vertical members of the shoring system or earth.

1541. General Requirements.

(a) Surface Encumbrances.

All surface encumbrances that are located so as to create a hazard to employees shall be supported, as necessary , to safeguard employees .

(b) Underground installations.

(1) The estimated location of utility installations, such as sewer, telephone, fuel, electric, water, lines, or any other underground installations that reasonably may be expected to be encountered during excavation work, shall be determined prior to opening an excavation.

(2) All Regional Notification Centers as defined by Government Code Section 4216(a) in the area involved and all known owners of underground facilities who are not members of a Notification Center shall be advised of the proposed work at least 2 working days prior to the start of any digging or excavation work.

EXCEPTION: Emergency repair work to underground facilities.

(3) When excavation operations approach the estimated location of underground installations, the exact location of the installations shall be determined by safe and acceptable means.

(4) While the excavation is open, underground installations shall be protected, supported or removed as necessary to safeguard employees.

(c) Access and egress.

(1) Structural ramps.

(A) Structural ramps that are used solely by employees as a means of access from excavations shall be designed by a competent person. Structural ramps used for access or egress of equipment shall be designedby a competent person qualified in structural design, and shall be constructed in accordance with the design.

(B) Ramps and runways constructed of two or more structural members shall have the structural members connected together to prevent displacement.

(C) Structural members used for ramps and runways shall be of uniform thickness.

(D) Cleats or other appropriate means used to connect runway structural members shall be attached to the bottom of the runway or shall be attached in a manner to prevent tripping.

(E) Structural ramps used in lieu of steps shall be provided with cleats or other surface treatments to the top surface to prevent slipping.

(2) Means of egress from trench excavations.

A stairway, ladder, ramp or other safe means of egress shall be located in trench excavations that are 4 feet or more in depth so as to require no more than 25 feet of lateral travel for employees.

(d) Exposure to vehicular traffic.

Employees exposed to public vehicular traffic shall be provided with, and shall wear, warning vests or other suitable garments marked with or made of reflectorized or high-visibility material.

(e) Exposure to falling loads.

No employee shall be permitted underneath loads handled by lifting or digging equipment. Employees shall be required to stand away from any vehicle being loaded or unloaded to avoid being struck by any spillage or falling materials. Operators may remain in the cabs of vehicles being loaded or unloaded when the vehicles are equipped, in accordance with Section 1591(e), to provide adequate protection for the operator during loading and unloading operations.

(f) Warning system for mobile equipment.

When mobile equipment is operated adjacent to an excavation, or when such equipment is required to approach the edge of an excavation, and the operator does not have a clear and direct view of the edge of the excavation, a warning system shall be utilized such as barricades, hand or mechanical signals or stop logs. If possible the grade should be away from the excavation.

(g) Hazardous atmospheres.

(1) Testing and controls.
In addition to the requirements set forth in the Construction Safety Orders and the General Industry Safety Orders to prevent exposure to harmful levels of atmospheric contaminants, the following requirements shall apply:

(A) Where oxygen deficiency (atmospheres containing less than 19.5 percent oxygen) or a hazardous atmosphere exists or could reasonably be expected to exist, such as in excavations in landfill areas or excavations in areas where hazardous substances are stored nearby, the atmospheres in the excavation shall be tested before employees enter excavations greater than 4 feet in depth.

(B) Adequate precautions shall be taken to prevent employee exposure to atmospheres containing less than 19.5 percent oxygen and other hazardous atmospheres. These precautions include providing proper respiratory protection or ventilation.

(C) Adequate protection shall be taken such as providing ventilation, to prevent employees exposure to an atmosphere containing a concentration of a flammable gas in excess of 20 percent of the lower flammable limit of the gas.

(D) When controls are used that are intended to reduce the level of atmospheric contaminants to acceptable levels, testing shall be conducted as often as necessary to ensure that the atmosphere remains safe.

(2) Emergency rescue equipment.

(A) Emergency rescue equipment, such as breathing apparatus, a safety harness and line, or a basket stretcher, shall be readily available where hazardous atmospheric conditions exist or may reasonably be expected to develop during work in an excavation. This equipment shall be attended when in use.

(B) Employees entering bell-bottom pier holes, or other similar deep and confined footing excavations, shall wear a harness with a lifeline securely attached to it. The lifeline shall be separate from any line used to handle material, and shall be individually attended at all times while the employee wearing the lifeline is in the excavation.

(h) Protection from hazards associated with water accumulation.

(1) Employees shall not work in excavations in which there is accumulated water, or in excavations in which water is accumulating, unless adequate precautions have been taken to protect employeea against the hazards posed by water accumulation. The precautions necessary to protect employees adequately vary with each situation, but could include special support or shield systems to protect from cave-ins, water removal to control the level of accumulating water, or use of a safety harness and lifeline.

(2) If water is controlled or prevented from accumulating by the use of water removal equipment, the water removal equipment and operations shall be monitored by a competent person to ensure proper operation.

(3) If excavation work interrupts the natural drainage of surface water (such as streams), diversion ditches, dikes, or other suitable means shall be used to prevent surface water from entering the excavation and to provide adequate drainage of the area adjacent to the excavation. Excavations subject of runoff from heavy rains will require an inspection by a competent person and compliance with Sections 1541(h) (1) and (h)(2).

(i) Stability of adjacent structures.

(1) Where the stability of adjoining buildings, wal1s, or other structures is endangered by excavation operations, support system such as shoring, bracing, or underpinning shall be provided to ensure the stability of such structures for the protection of employees.

(2) Excavation below the level of the base or footing of any foundation or retaining wall that could be reasonably expected to pose a hazard to employees shall not be permitted except when:

(A) A support system, such as underpinning, is provided to ensure the safety of employees and the stability of the structure; or

(B) The excavation is in stable rock; or

(C) A registered professional engineer has approved the determination that such excavation work will not pose as hazard to employees.

(3) Sidewalks, pavements and appurtenant structures shall not be undermined unless a support system or another method of protection is provided to protect employees from the possible collapse of such structures.

(J) Protection of employees from loose rock or soil.

(1) Adequate protection shall be provided to protect employees from loose rock or soil that could pose a hazard by falling or rolling from an excavation face. Such protection shall consist of scaling to remove loose material; installation of protective barricades at intervals as’ necessary on the face to stop and contain falling material; or other means that provide equivalent protection.

(2) Employees shall be protected from excavated or other materials or equipment that could pose a hazard by falling or rolling into excavations. Protection shall be provided by placing and keeping such materials or equipment at least 2 feet from the edge of excavations, or by the use of retaining devices that are sufficient to prevent materials or equipment from falling or rolling into excavations, or by a combination of both if necessary.

(k) Inspections.

(1) Daily inspections of excavations, the adjacent areas, and protective systems shall be made by a competent person for evidence of a situation that could result in possible cave-ins, indications of failure of protective systems, hazardous atmospheres, or other hazardous conditions. An inspection shall be conducted by the competent person prior to the start of work and as needed throughout the shift. inspections shall also be made after every rain storm or other hazard increasing occurrence. These inspections are only required when employee exposure can be reasonable anticipated.

(2) Where the competent, person finds evidence of a situation that could result in a possible cave-in, indications of failure of protective systems, hazardous atmospheres, or other hazardous conditions, exposed employees shall be removed from the hazardous area until the nececesary precautions have been taken to ensure their safety.

(1) Fall protection.

(1) Where employees or equipment are required or permitted to cross over excavations, walkways or bridges with standard guardrails shall be provided.

(2) Adequate barrier physical protection shall be provided at all remotely located excavations. All wells, pits, shafts, etc., shall be barricaded or covered. Upon completion of exploration and other similar operations, temporary wells, pits, shafts, etc., shall be backfilled.

1541.1 Requirements For Protective Systems.

(a) Protection of employees in excavations.

(1) Each employee in an excavation shall be protected from cave-ins by an adequate protective system designed in accordance with Section 1541.1(b) or (c) except when:
- (A) Excavations are made entirely in rock; or
- (B) Excavations are less than 5 feet in depth and examination of the ground by a competent person provides no indication-of potential cave-in.

(2) protective systems shall have the capacity to resist without failure all loads that are intended or could reasonably be expected to be applied or transmitted to the system.

(b) Design of sloping and benching systems.

The slopes and configurations of sloping and benching systems shall be selected sand constructed by the employer or his designee,and shall be in accordance with the requirements of Section 1541.1(b) (1); or, in the alternative, Section 1541.1(b) (2); or, in the alternative 1541.1(b) (3); or, in the alternative, Section 1541.1(b) (4), as follows:

(1) Option (1) Allowable configurations and slopes,
- (A) Excavations shall be sloped at an angle not steeper than one and one-half horizontal to one vertica1 (34 degrees measured from the

horizontal), unless the employer uses one of the options listed below:

(B) Slopesspecified in Section 1541.1(b) (1) (A) shall be excavated to form configurations that are in accordance with the slopes shown for Type C soil in Appendix B to this article.

(2) Option (2) - Determination of slopes and configurations using Appendices A and B. Maximum allowable slopes, and allowable configurations for sloping and benching systems, shall be determined in accordance with the conditions and requirements set forth in Appendices A and B to this article.

(3) option (3) - Designs using other tabulated data.

(A) Designs of sloping or benching systems shall be selected from and be in accordance with tabulated data, such as tables and charts.

(B) The tabulated data shall be in written form and shall include all of the following:

1. Identification of the parameters that affect the selection of a sloping or benching system drawn from such data;
2. Identification of the limits of use of the data, to include the magnitude and configuration of slopes determined to be safe;
3. Explanatory information as may be necessary to aid the user in making a correct selection of a protective system from the data;
4. At least one copy of the tabulated data which identifies the registered professional engineer who approved the data, shall be maintained at the jobsite during construction of the protective system. After that time the data may be stored off the jobsite, but a copy of the data shall be made available to the Division upon request.

(4) Option (4) - Design by a registered professional engineer.

(A) Sloping and benching systems not utilizing option (1) or option (2) or Option (3) under Section 1541.1(b) shall be approved by a registered professional engineer.

(B) Designs shall be in written form and shall include at least the following:

1. The magnitude of the slopes that were determined to be safe for the particular project;
2. The configuration's that were determined to be safe for the particular project;
3. The identity of the registered professional engineer approving the design.

(C) At least one copy of the design shall be maintained at the jobsite while the slope is being constructed. After that time the design need not be at the jobsite, but a copy shall be made available to the Division upon request.

(c) Design of support systems, shield systems, and other protective systems.

Designs of support systems, shield systems, and other protective systems shall be selected and constructedby the employer or his designee and shall be in accordance with the requirements of Section 1541.1(c) (1); or, in the alternative, Section 1541.1(c) (2); or, in the alternative Section 1541.1(c) (3); or, in the alternative Section 1541.1(c) (4) as follows:

(1) Option (1) - Designs using Appendices A, C, and D. Designs for timber shoring in trenches shall be determined in accordance with the conditions and requirements set forth in Appendices A and C to this article. Designs for aluminum hydraulic shoring shall be in accordance with Section 1541.l(c) (2), but if manufacturer's tabulated data cannot be utilized, designs shall be in accordance with Appendix D.

(2) Option (2) - Designs using manufacturers Tabulated Data
 (A) Design of support systems, shield systems, or other protective systems that are drawn from manufacturer's tabulated data shall be in accordance with all recommendations, and limitations issued or made by the manufacturer.
 (8) Deviation from the specifications, recommendations, and limitations issued or made by the manufacturer shall only be allowed after the manufacturer issues specific written approval.
 (C) Manufacturer's specifications, recommendations, and limitations, and manufacturer's approval to deviate from the specifications, recommendations, and limitations shall be in written form at the jobsite during construction of the protective system. After that time this data may be stored off the jobsite, but a copy shall be made available to the Division upon request.

(3) Option (3) - Designs using other tabulated data.
 (A) Designs of support systems, shield systems, or other protective systems shall be selected from and be in accordancewith tabulated data, such as tables and charts.
 (B) The tabulated data shall be in written form and include all of the following:
  1. Identification of the parameters that affect the selection of a protective system drawn from such data;
  2. Identification of the limits of use of the data;
  3. Explanatory information as may be necessary to aid the user. in making a correct selection of a protective system from the data.
 (C) At least one copy of the tabulated data, which identifies the registered professional engineer who approved the data, shall be maintained at the jobsite during construction of the protective system. After that time the data may be stored off the jobsite, but a copy of the data shall be made available to the Division on request.

(4) Option (4) - Design by a registered professional engineer.
 (A) Support systems, shield systems, and other protective systems not utilizing Option 1, Option 2, or Option 3 above, shall be approved by a registered professional engineer.
 (B) Designs shall be in written form and shall include the following:
  1. A plan indicating the sizes, types, and configurations of the materials to be used in the protective system; and
  2. The identity of the registered professional engineer approving the design.
 (C) At least one copy of the design shall be maintained at the jobsite during construction of the protective system. After that time, the designmay be stored off the jobsite, but a copy of the design shall be made available to the Division upon request.

(d) Materials and equipment.

(1) Materials and equipment used for protective systems shall be free from damage or defects that might impair their proper function.

(2) Manufactured materials and equipment used for protective systems shall be used and maintained in a manner that is consistent with the recommendations of the manufacturer, and in a manner that will prevent employee exposure to hazards.

(3) When material or equipment that is used for protective systems is damaged, a competent person shall examine the material or equipment and evaluate its suitability for continued use. If the competent person cannot assure the material or equipment is able to support the intended loads or is otherwise suitable for safe use, then such material or equipment shall be removed from service, and shall be evaluated and approved by a registered professional engineer before being returned to service.

(e) Installation and removal of supports.

(l) General.

(A) Members of support systems shall be securely connected together to prevent sliding, falling, kickouts, or other predictable failure.

(B) Support systems shall be installed and removed in a manner that protects employees from cave-ins, structural collapses, or from being struck be members of the support system.

(C) Individual members of support systems shall not be subjected to loads exceeding those which those members were designed to withstand.

(D) Before temporary removal of individual members begins, additional precautions shall be taken to ensure the safety of employees, such as installing other structural members to carry the load imposed on the support system.

(E) Removal shall begin at, and progress from, the bottom of the excavation. Members shall be released s1ow1y so as to note any indication of possible failure of the remaining members of the structure or possible cave-in of the sides of the excavation.

(F) Backfilling shall progress together with the removal of support systems from excavations.

(2) Additional requirements for support systems for trench excavations.

(A) Excavations of material to a level no greater than 2 feet below the bottom of the members of a support system shall be permitted, but only if the system is designed to resist the forces calculated for the full depth of the trench, and there are no indications while the trench is open of a possible loss of soil from behind or below the bottom of the support system.

(B) Installation of a support system shall be closely coordinated with the excavation of trenches.

(f) Sloping and benching systems.

Employees shall not be permitted to work on the faces of sloped or benched excavations at levels above other employees except when employees at the lower levels are adequately protected from the hazards of falling, rolling, or sliding material or equipment.

(g) Shield systems.

(1) General.

(A) Shield systems shall not be subjected to loads exceeding those which the system was designed to withstand.

(B) Shields shall be installed in a manner to restrict lateral or other hazardous movement of the shield in the event of the application of sudden lateral loads.

(C) Employees shall be protected from the hazard of cave-ins when entering of exiting the areas protected by the shields.

(D) Employees shall not be allowed in shields when shields are being installed, or moved vertically.

(2) Additional requirement fort shield systems used in trench excavations. Excavations of earth material to a level not greater than 2 feet below the bottom of a shield shall be permitted, but only if the shield is designed to resist the forces calculated for the full depth of the trench, and there are no indications while the trench is open of a possible loss of soil from behind or below the bottom of the shield.

APPENDIX A

Appendix A to Section 1541.1

SOIL CLASSIFICATION

(a) Scope and application.

(1) Scope l This appendix describes a method of classifying soil and deposits based on site and environmental conditions, and on the structure and composition of the earth deposits. The appendix contains definitions, sets forth requirements, and describes acceptable visual and manual tests for use in classifying soils.

(2) Application. This appendix applies when a sloping or benching system is designed in accordance with the requirements set forth in Section 1541.1(b)(2) as a method of protection for employees from cave-ins. This appendix also applies when timber shoring for excavations is designed as a method of protection from cave-ins in accordance with Appendix C of this article, and when aluminum hydraulic shoring is designed in accordance with Appendix D. This appendix also applies if other protective systems are designed and selected for use from data prepared in accordance with the requirements set forth in Section 1541.1(c), and the use of the data is predicated on the use of the soil classification system set forth in this appendix.

(b) Definitions.

Cemented soil. A soil in which the particles are held together by a chemical agent, such as calcium carbonate, such that a hand-size sample cannot be crushed into powder or individual soil particles by finger pressure.

Cohesive soil. Clay (fine grained soil), or soil with a high clay content, which has cohesive strength. Cohesive soil does not crumble, can be excavated with vertical side slopes, and is plastic when moist. Cohesive soil is hard to break up when dry, and exhibits significant cohesion when submerged. Cohesive soils include clayey silt, sandy clay, silty clay, clay and organic clay.

Dry soil. Soil that does not exhibit visible signs of moisture content.

Fissured. A soil material that has a tendency to break along definite planes of fracture with little resistance, or a material that exhibits open cracks, such as tension cracks, in an exposed surface.

Granular Soil. Gravel, sand, or silt (coarse grained soil) with little or no clay content. Granular soil has no cohesive strength. Some moist granular soils exhibit apparent cohesion. Granular soil cannot be molded when moist and crumbles easily when dry.

Layered system. Two or more distinctly different soil or rock types arranged in layers. Micaceous seams or weakened planes in rock or shale are considered layered.

Moist soil. A condition in which a soil looks and feels damp. Moist cohesive soil can easily be shaped into a ball and rolled into small diameter threads before crumbling. Moist granular soil that contains some cohesive material will exhibit some signs of cohesion between particles.

Plastic. A property of a soil which allows the soil to be deformed or molded without cracking, or appreciable volume change.

Saturated soil. A soil in which the voids are filled with water. Saturation does not require flow. Saturation, or near saturation, is necessary for the proper use of instruments such as a pocket penetrometer or shear vane.

Soil classification system. A method of categorizing soil and rock deposits in a hierarchy of Stable Rock, Type A, Type B, and Type C, in decreasing order of stability. The categories are determined based on an analysis of the properties and performance characteristics of the deposits and the characteristics of the deposits and the-environmental conditions of exposure.

Stable rock. Natural solid mineral matter that can be excavated with vertical sides and remain intact while exposed.

Submerged soil. Soil which is underwater or is free seeping.

Type A soil.

Cohesive soils with an unconfined compressive strength of 1.5 tons per square foot (tsf) or greater. Examples of cohesive-soils are: clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. Cemented soils such as caliche and hardpan are also considered Type A. However, no soil is type A if:

(1) The soil is fissured; or
(2) The soil is subject to vibration from heavy traffic, pile driving, or similar effects; or
(3) The soil has been previously disturbed; or
(4) The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:lV) or greater; or
(5) The material is subject to other factors that would require it to be classified as a less stable material.

Type B soil.

(3) Cohesive soil with an unconfined compressive strength greater than 0.5 tsf but less than 1.5 tsf; or
(2) Granular cohesionless soils including: angular gravel (similar to crushed rock), silt, silt loam, sandy loam and, in some cases, silty clay loam and sandy clay loam.
(3) Previously disturbed soils except those which would otherwise be classified as Type C soil.
(4) Soil that meets the unconfined compressive strength or cementation requirements for Type A, but is fissured or subject to vibration; or
(5) Dry rock that is not stable; or

(6) Material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep than four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.

Type C soil.

(1) Cohesive soil with an unconfined compressive strength of 0.5 tsf or less; or
(2) Granular soils including gravel, sand, and loamy sand; or
(3) Submerged soil or soil from which water is freely seeping; or
(4) Submerged rock that is not stable, or
(5) Material in a sloped, layered system where the layers dip into the excavation or a slope of four horizontal to one vertical (4H:1V) or steeper.

Unconfined compressive strength. The load per unit area at which a soil will fail in compression. It can be determined by laboratory testing, or estimated in the field using a pocket penetrometer, by thumb penetration tests, and other methods.

Wet soil. Soil that contains significantly more moisture than moist soil, but in such a range of values that cohesive material will slump or begin to flow when vibrated. Granular material that would exhibit cohesive properties when moist will lose those cohesive properties when wet.

(c) Requirements.

(1) Classification of soil and rock deposits. Each soil and rock deposit shall be classified by a competent person as Stable Rock, Type A, Type B, Type C in accordance with the definitions set forth in paragraph (b) of this appendix.

(2) Basis of classification. The classification of the deposits shall be made based on the results of at least one visual and at least one manual analysis. Such analysis shall be conducted by a competent person using tests described in paragraph (d) below, or in other approved methods of soil classification and testing such as those adopted by the American Society for Testing Materials, or the U.S. Department of Agriculture textural classification system.

(3) Visual and manual analysis. The visual and manual analysis, such as those noted as being acceptable in paragraph (d) of this appendix, shall be designed and conducted to provide sufficient quantitative and qualitative information as may be necessary to identify properly the properties, factors, and conditions affecting the classification of the deposits.

(4) Layered systems. In a layered system, the system shall be classified in accordance with its weakest layer. However, each layer may be classified individually where a more stable layer lies under a less stable layer.

(5) Reclassification. If, after classifying a deposit, the properties, factors, or conditions affecting its classification change in any way, the changes shall be evaluated by a competent person. The deposit shall be reclassified as necessary to reflect the changed circumstances.

(d) Acceptable visual and manual tests.

(1) Visual tests. Visual analysis is conducted to determine qualitative information regarding the excavation site in general the soil adjacent to the excavation, the soil forming the sides of the open excavation, and the soil taken as samples from excavated material.

(A) Observe samples of soil that are excavated and soil in the sides of the excavation. Estimate the range of particle sizes and the relative amounts of the particle sizes. Soil that is primarily composed of fine-grained material is cohesive material. Soil composed primarily of coarse-grained sand or gravel is granular material.

(B) Observe soil as it is excavated. Soil that remains in clumps when excavated is cohesive. Soil that breaks up easily and does not stay in clumps is granular.

(C) Observe the side of the opened excavation and the surface area adjacent to the excavation. Crack-like openings such as tension cracks could indicate fissured material. If chunks of soil spa11 off a vertical side, the soil could be fissured. Small spalls are evidence of moving ground and are indications of potentially hazardous situations.

(D) Observe the area adjacent to the excavation and the excavation itself for evidence of existing utility and other underground structures, and to identify previously disturbed soil.

(E) Observe the opened side of the excavation to identify layered systems. Examine layered systems to identify if the layers slope toward the excavation. Estimate the degree of slope of the layers.

(F) Observe the area adjacent to the excavation and the sides of the opened excavation for evidence of surface water, water seeping from the sides of the excavation, or the location of the level of the water table.

(G) Observe the area adjacent to the excavation and the area within the excavation for sources of vibration that may affect the stability of the excavation face.

(2) Manual tests. Manual analysis of soil samples is conducted to determine quantitative as well as qualitative properties of soil and to provide more information in-order to classify soil properly.

(A) Plasticity. Mold a moist or wet sample of soil into a ball and attempt to roll it into threads as thin as l/8-inch in diameter. Cohesive material can be successfully rolled into threads without crumbling. For example, if at least a two inch length of 1/8-inch thread can be held on one end without tearing, the soil is cohesive.

(8) Dry strength. If the soil is dry and crumbles on its own or with moderate pressure into individual grains or fine powder, it is granular (any combination of gravel, sand, or silt). If the soil is dry and falls into clumps which break up into smaller clumps, but the smaller clumps can only be broken up with difficulty, it may be clay in any combination with gravel, sand or silt. If the dry soil breaks into clumps which do not break up into small clumps and which can

only be broken with difficulty, and there is no visual indication the soil is fissured, the soil may be considered unfissured.

(C) Thumb penetration. The thumb penetration test can be used to estimate the unconfined compressive strength of cohesive soils. Type A soils with an unconfined compressive strength of 1.5 tsf can be readily indented by the thumb; however, they can be penetrated by the thumb only with very great effort. Type C soils with an unconfined compressive strength of 0.5 tsf can be easily penetrated several inches by the thumb, and can be molded by light finger pressure. This test should be conducted on an undisturbed soil sample, such as a large clump of soil, as soon as practicable after excavation to keep to a minimum the effects of exposure to drying influences. If the excavation is later exposed to wetting influences (rain, flooding), the classification of the soil must be changed accordingly.

(D) Other strength tests. Estimates of unconfined compressive strength of soils can also be obtained by use of a pocket penetrometer or by using a hand-operated shear vane.

(E) Drying test. The basic purpose of the drying test is to differentiate between cohesive material with fissures, unfissured cohesive material, and granular material. The procedure for the drying test involves drying a sample of soil that is approximately one inch thick and six inches in diameter until it is thoroughly dry:

1. If the sample develops cracks as it dries, significant fissures are indicated.

2. Samples that dry without cracking are to be broken by hand. If considerable force is necessary to break a sample, the soil has significant cohesive material content. The soil can be classified as an unfissured cohesive material and the unconfined compressive strength should be determined.

3. If a sample breaks easily by hand, it is either a fissured cohesive material or a granular material. To distinguish between the two, pulverize the dried clumps of the sample by hand or by stepping on them. If the clumps do not pulverize easily, the material is cohesive with fissures. If they pulverize easily into very small fragments, the material is granular.

Appendix B to Section 1541.1

SLOPING AND BENCHING

(a) Scope and application.

This appendix contains specifications for sloping and benching when used as methods of protecting employees working in excavations from cave-ins. The requirements of this appendix apply when the design of sloping and benching protective systems is to be performed in accordance with the requirements set forth in Section 1541.1 (b).

(b) Definitions.

Actual slope means the slope to which an excavation face is excavated.

Distress means that the soil is in a condition where a cave-in is imminent or is likely to occur. Distress is evidenced by such phenomena as the development of fissures in the face of or adjacent to an open excavation; the subsidence of the edge of an excavation; the slumping of material from the face or the bulging or heaving of material from the bottom of an excavation; the spalling of material from the face of an excavation; and ravelling, i.e., small amounts of material such as pebbles or little clumps of material suddenly separating from the face of an excavation and trickling or rolling down into the excavation.

Maximum allowable slope means the steepest incline of an excavation face that is acceptable for the most favorable site conditions as protection against cave-ins, and is expressed as the ratio of horizontal distance to vertical rise (H:V).

Short term exposure means a period of time less than or equal to 24 hours that an excavation is open.

(c) Requirements.

(1) Soil classification. Soil and rock deposits shall be classified in accordance with Appendix A to Section 1541.1.

(2) Maximum allowable slope. The maximum allowable slope for a soil or rock deposit shall be determined from Table B-l of this appendix.

(3) Actual slope.

(A) The actual slope shall not be steeper than the maximum allowable slope.

(B) The actual slope shall be less steep then the maximum allowable slope when there are signs of distress. If that situation occurs, the slope shall be cut back to an actual slope which is at least l/2 horizontal to one vertical (1/2H:lV) less steep than the maximum allowable slope.

(C) When surcharge loads from stored material or equipment, operating equipment, or traffic are present, a competent person "shall determine the degree to which the actual slope must be reduced below the maximum allowable slope, and shall assure that such reduction is achieved. Surcharge loads from adjacent structures shall be evaluated in accordance with Section ~~1541.1(i)~~ 1541(i)

(4) Configurations. Configurations of sloping and benching systems shall be in accordance with Figure B-1.

TABLE B-1

MAXIMUM ALLOWABLE SLOPES

SOIL OR ROCK TYPE	MAXIMUM ALLOWABLE SLOPES (H:V)[1] FOR EXCAVATIONS LESS THAN 20 FEET DEEP [3]
STABLE ROCK	VERTICAL. (90°)
TYPE A[2]	3/4 : 1 (53°)
TYPE B	1:1 (45°)
TYPE C	1 l/2 : 1 (34°)

NOTES:

1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. A short-term maximum allowable slope of 1/2H:lV (63 degrees) is allowed in excavations in Type A soil that are 12 feet or less in depth. Short-term maximum allowable slopes for excavations greater than 12 feet in depth shall be 3/4H:lV (53 degrees).
3. Sloping or benching for excavationsgreater than 20 feet deep shall be designed by a registered professional engineer.

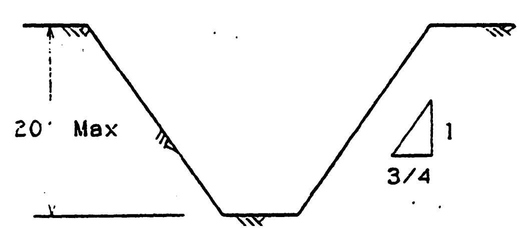
FIGURE B-1

SLOPE CONFIGURATIONS

(All slopes stated below are in the horizontal to vertical ratio).

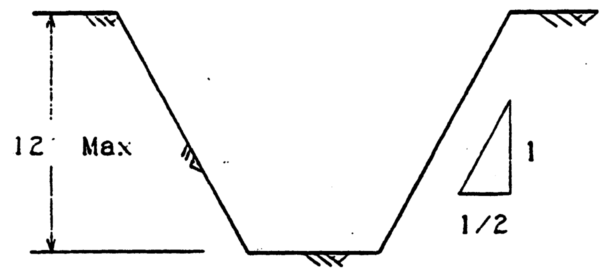
B-1.1 Excavations Made in Type A soil.

1. All simple slope excavations 20 feet or less in depth shall have a maximum allowable slope of 3/4:1.



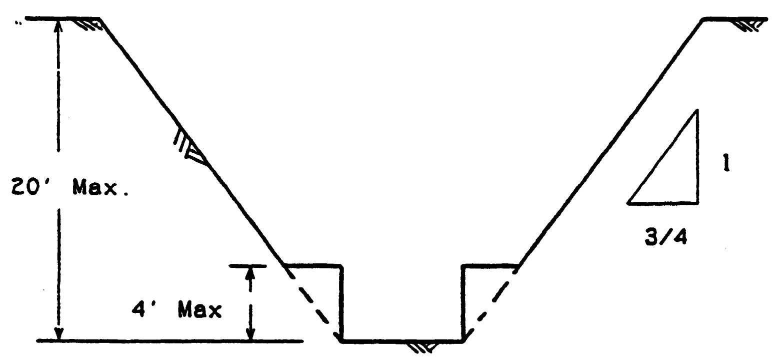
Simple Slope - General

Exception: Simple slope excavations which are open 24 hours or less (short term) and which are 12 feet or less in depth shall have a maximum allowable slope of l/2:1.

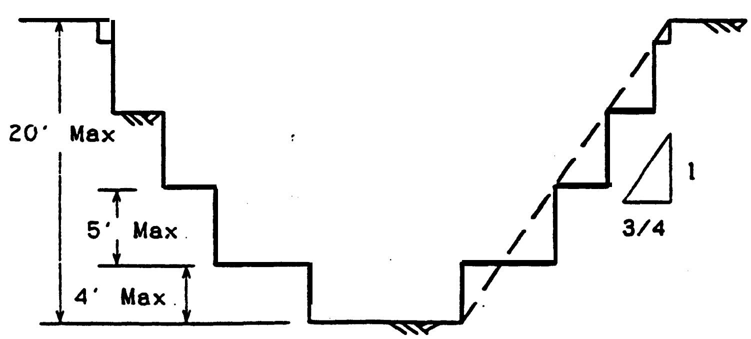


Simple Slope - Short Term

2. All benched excavations 20 feet or less in depth shall have a maximum allowable slope of 3/4:1 and maximum bench dimensions as follows:

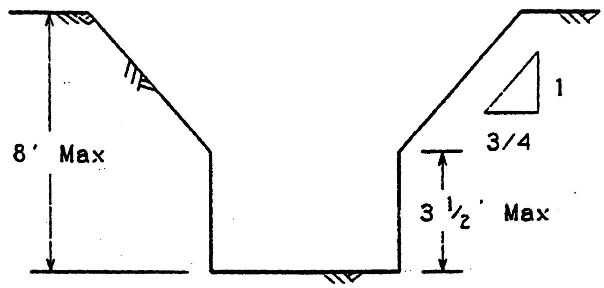


Simple Bench



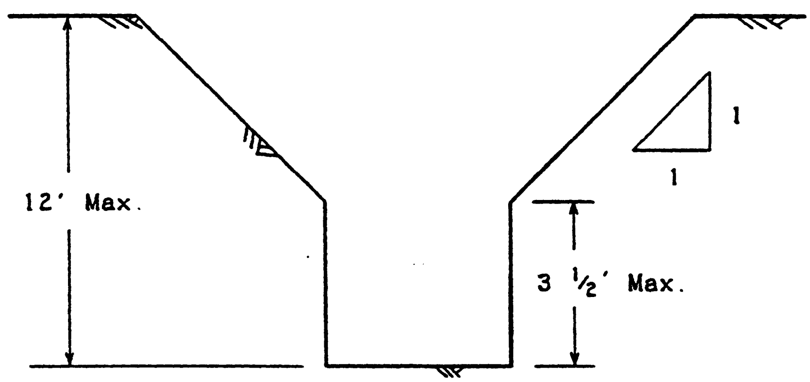
Multiple Bench

3. All excavations 8 feet or less in depth which have unsupported vertically sided lower portions shall have a maximum vertical side of 3 l/2 feet.



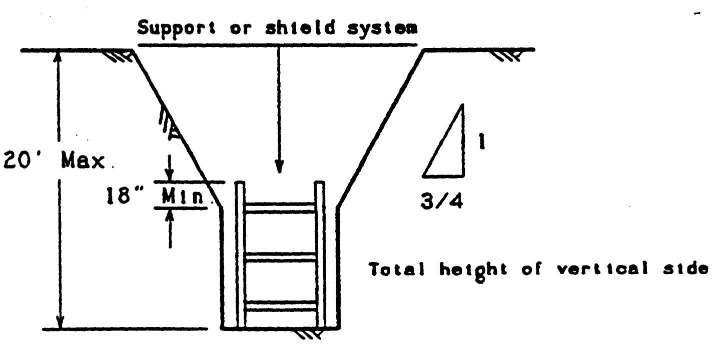
Unsupported Vertically Sided Lower Portion
Maximum 8 Feet in Depth

All excavations more than 8 feet but not more than 12 feet in depth with &unsupported vertically sided lower portions shall have a maximum allowable slope of 1:l and a maximum vertical side of 3 l/2 feet.



Unsupported Vertically Sided Lower Portion
Maximum 12 Feet in Depth

All excavations 20 feet or less in depth which have vertically sided lower portions that are supported or shielded shall have a maximum allowable slope of 3/4:1. The support or shield system must extend at least 18 inches above the top of the vertical side.

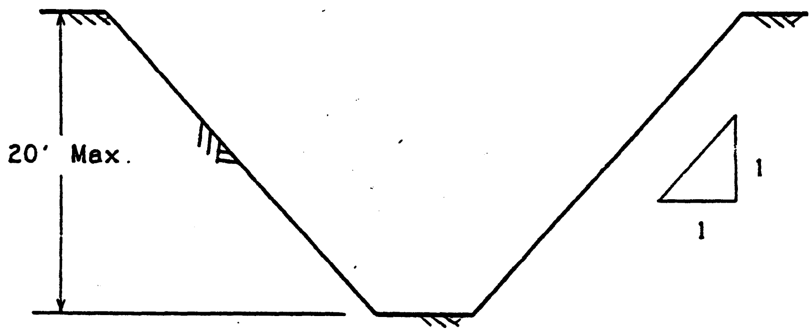


Supported or Shielded Vertically Sided Lower Portion

4. All other simple slope, compound slope, and vertically sided lower portion excavations shall be in accordance with the other options permitted under 1541.1(b).

B- 1.2 Excavations Made in Type B Soil

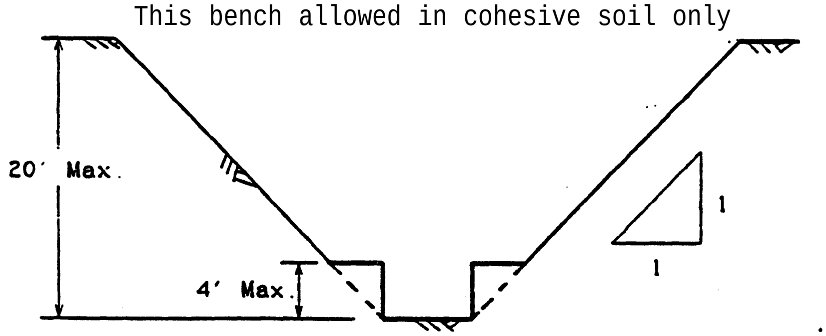
1. All simple slope excavations 20 feet or less in depth shall have a maximum allowable slope of 1:l.



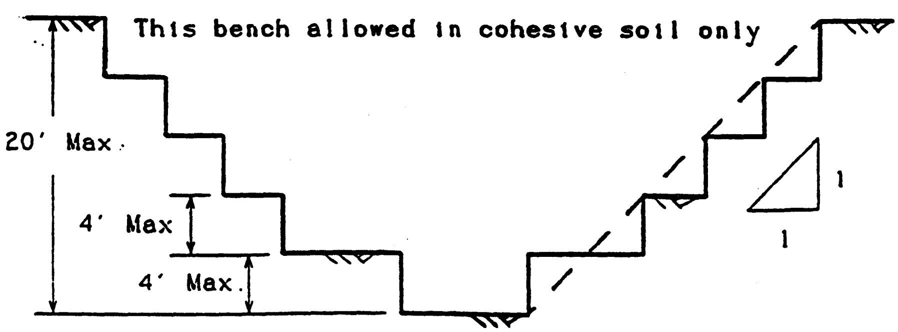
Simple Slope

2. All benched excavations 20 feet or less in depth shall have a maximum allowable slope of 1:l and maximum bench dimensions os follows:

This bench allowed in cohesive soil only

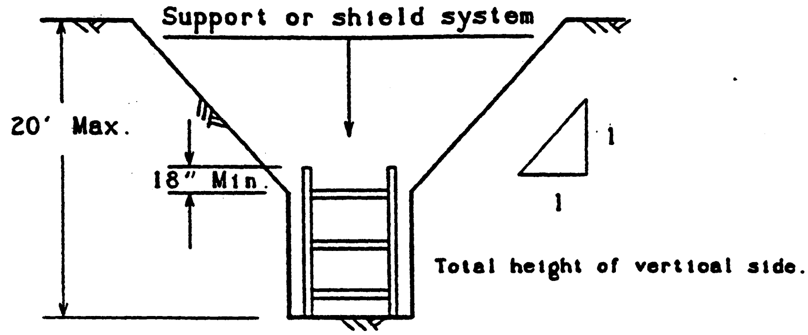


Single Bench



Multiple Bench

3. All excavations 20 feet or less in depth which have vertically sided lower portions shall be shielded or supported to a height at least 18 inches above the top of the vertical side. All such excavations shall have a maximum allowable slope of l:1.

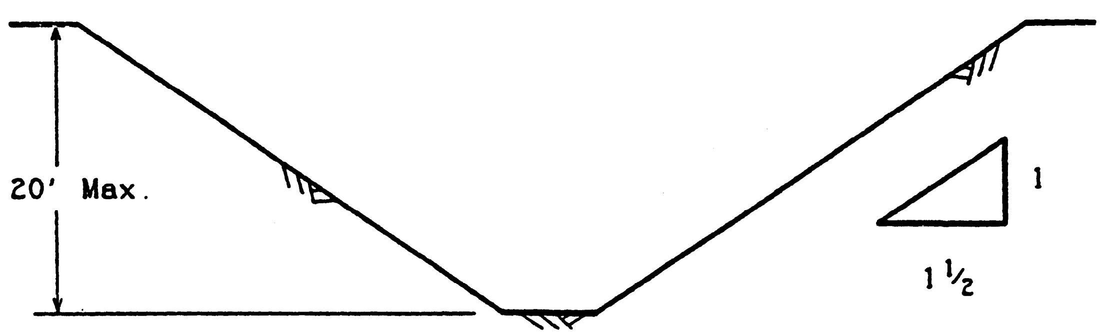


Vertically Sided Lower Portion

4. All other sloped excavations shall be in accordance with the other options permitted in 1541.1(b).

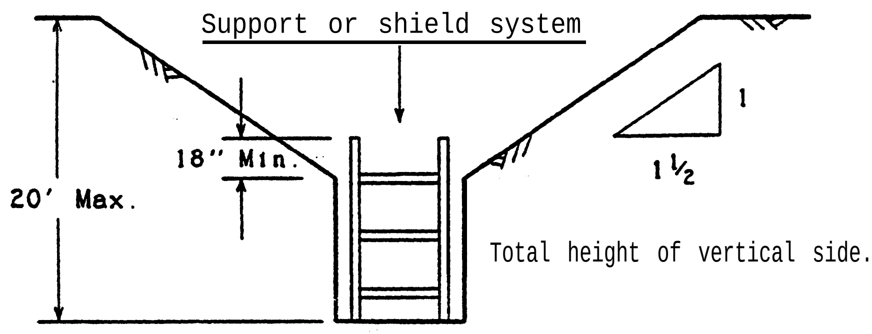
B - 1.3 Excavations Made in Type C Soil

1. All simple slope excavations 20 feet or less in depth shall have a maximum allowable slope of 1 l/2:1.



Simple Slope

2. All excavations 20 feet or less in depth which have vertically sided lower portions shall be shielded or supported to a height at least 18 inches above the top of the vertical side. All such excavations shall have a maximum allowable slope of 1 1/2:1.

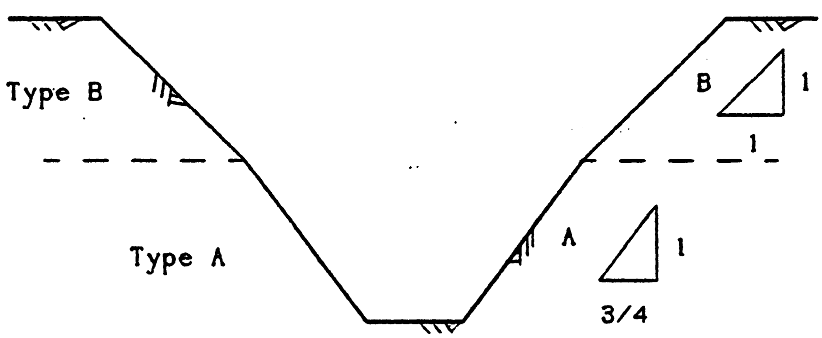


Vertical Sided Lower Portion

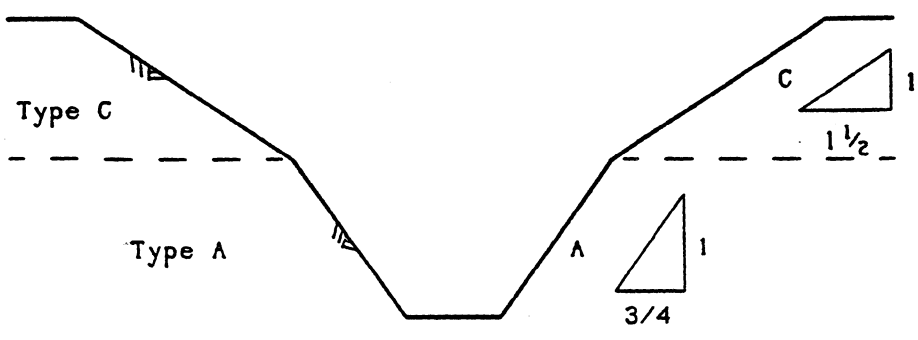
3. All other sloped excavations shall be in accordance with the other options permitted in 1541.1(b).

B - 1.4 Excavations Made in Layered Soils

1. All excavations 20 feet or less in depth made in layered soils shall have a maximum allowable slope for each layer as set forth below:

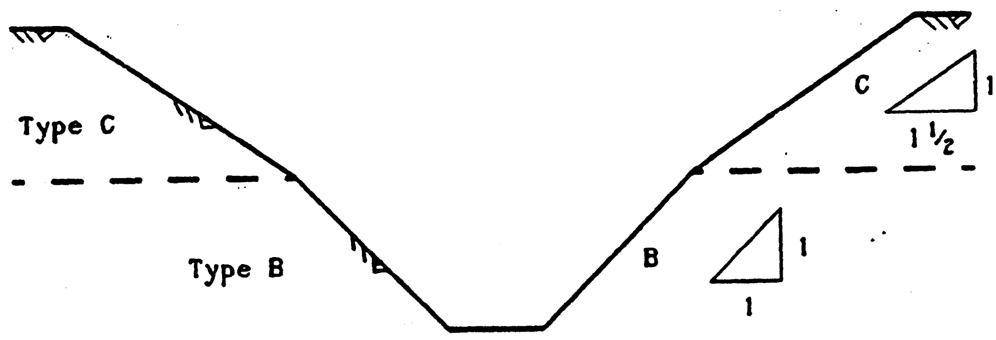


B OVER A

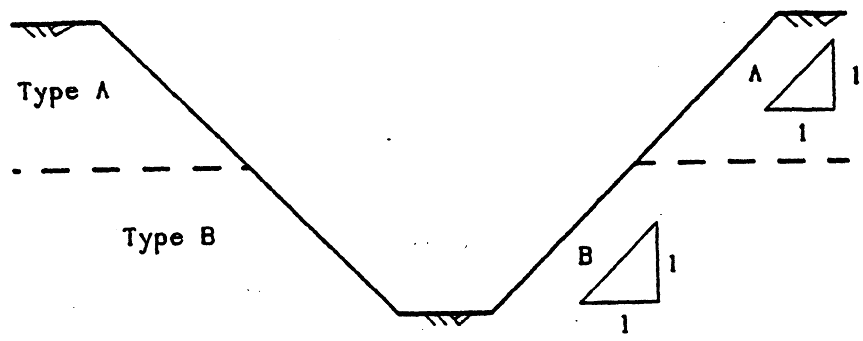


C OVER A

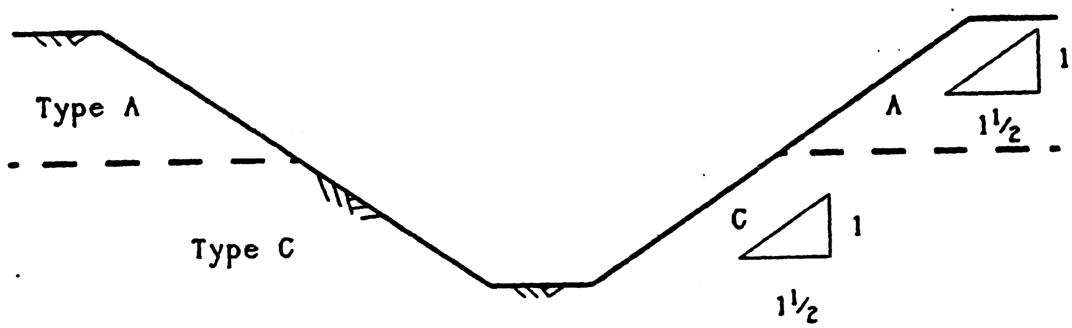
APPENDIX A



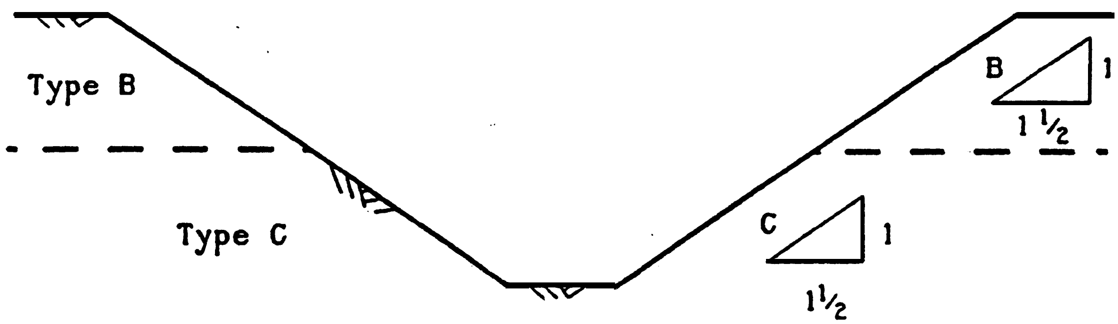
C OVER B



A OVER B



A OVER C



B OVER C

2. All other sloped excavations shall be in accordance with the other options permitted in 1541.1(b).

Appendix C to Section 1541.l

TIMBER SHORING FOR TRENCHES

(a) Scope

This appendix contains information that can be used when timber shoring is provided as a method of protection from cave-ins in trenches that do not exceed 20 feet in depth. This appendix must be used when design of timber shoring protective systems is to be performed in accordance with Section 1541.1(c)(l). Other timber shoring configurations; other systems of support such as hydraulic and pneumatic systems; and other protective systems such as sloping, benching, shielding, and freezing systems must be designed in accordance with the requirements set forth in Section 1541.1(c).

(b) Soil Classification.

In order to use the data presented in this appendix, the soil type or types in which the excavation is made must first be determined using the soil classification method set forth in Article 6.

(c) Presentation of Information.

Information is presented in several forms as follows:

(1) Information is presented in tabular form in Tables C-1.1, C-1.2, and C-l.3 and Tables C-2.1, C-2.2 and C-2.3 following Section (g) of Appendix C. Each table presents the minimum sizes of timber members to use in a shoring system, and each table contains data only for the particular soil type in which the excavation or portion of the excavation is made. The data are arranged to allow the user the flexibility to select from among several acceptable configurations of members baked on varying the horizontal spacing of the crossbraces. Stable rock is exempt from shoring requirements and therefore, no data are presented for this condition.

(2) Information concerning the basis of the tabular data and the limitations of the data is presented in Section (d) of this appendix, and on the tables themselves.

(3) Information explaining the use of the tabular data is presented in Section (e) of this appendix.

(4) Information illustrating the use of the tabular data is presented in Section (f) of this appendix.

(5) Miscellaneous notations regarding Tables C-l.1 through C-1.3 and Tables C-2.1 through C-2.3 are presented in Section (g) of this appendix,

(d) Basis and limitations of the data.

(1) Dimensions of timber members.

(A) The sizes of the timber members listed in Tables C-1.1 through C-1.3 are taken from the National Bureau of Standards (NBS) report, "Recommended Technical Provisions for Construction Practice in Shoring and Sloping of Trenches and Excavations." In addition, where NBS did not recommend specific sizes of members, member sizes are based on an analysis of the sizes required for use by existing codes and on empirical practice.

(B) The required dimensions of the members listed in Tables C-1.1 through C-1.3 refer to actual dimensions and not nominal dimensions of the timber. Employers wanting to use nominal size shoring are directed to Tables C-2.1 through C-2.3, or have this choice under Section 1541.1(c)(3).

(2) Limitations of application.

(A) It is not intended that the timber shoring specification apply to every situation that may be experienced in the field. These data were developed to apply to the situations that are most commonly experienced in current trenching practice. Shoring systems for use in situations that are not covered by the data in this appendix must be designed as specified in Section l541.l(c).

(B) When any of the following conditions are present, the members specified in the tables are not considered adequate. Either an alternate timber shoring system must be designed or another type of protective system designed in accordance with Section 1541.1.

1. When loads imposed by structures or by stored material adjacent to the trench weigh in excess of the load imposed by a two-foot soil surcharge. The term "adjacent" as used here means the area within a horizontal distance from the edge of the trench equal to the depth of the trench.

2. When vertical loads imposed on crossbraces exceed a 240-pound gravity load distributed on a one-foot section of the center of the crossbrace.

3. When surcharge loads are present from equipment weighing in excess of 20,000 pounds.

4. When only the lower portion of the trench is shored and the remaining portion of the trench is sloped or benched unless: The sloped portion is sloped at an angle less steep than three horizontal to one vertical; or the members are selected from the tables for use at a depth which is determined from the top of the overall trench, and not from the top of the sloped portion.

(e) Use of Tables.

The members of the shoring system that are to be selected using this information are the crossbraces, the uprights, and the wales, where wales are required. Minimum sizes of members are specified for use in different types of soil. There are six tables of information, two for each soil type. The soil type must first be determined in accordance with the soil classification system described in Appendix A. Using the appropriate table, the selection of the size and spacing of the members is then made. The selection is based on the depth and width of the trench where the members are to be installed and, in most instances, the selection is also based on the horizontal spacing of the crossbraces. Instances where a choice of. horizontal spacing of crossbraces is available, the horizontal spacing of the crossbraces must be chosen by the user before the size of any member can be determined. When the soil type, the width and depth of the trench, and the horizontal spacing of the crossbzaces are known, the size and vertical spacing of the crossbracing, the size and vertical spacing of the wales, and the size and horizontal spacing of the uprights can be read from the appropriate table.

(f) Examples to Illustrate the Use of Tables C-1.1 through C-1.3

(1) Example 1.

A trench dug in Type A soil is 13 feet deep and five feet wide. From Table C-l.1 four acceptable arrangements of timber can be used.

Arrangement #l

Space 4X4 crossbraces at six feet horizontally and four feet vertically. Wales are not required.
Space 3X8 uprights at six feet horizontally. This arrangement is commonly called "skip shoring."

Arrangement #2

Space 4X6 crossbraces at eight feet horizontally and four feet vertically.
Space 8X8 wales at four feet vertically.
Space 2X6 uprights at four feet horizontally.

Arrangement #3

Space 6X6 crossbraces at 10 feet horizontally and four feet vertically.
Space 8X10 wales at four feet vertically.
Space 2X6 uprights at six feet horizontally.

Arrangement #4

Space 6X6 crossbraces at 12 feet horizontally and 4 feet vertically.
Space 10X10 wales at four feet vertically.
Space 3X8 uprights at six feet horizontally.

(2) Example 2.

A trench dig in Type B soil is 13 feet deep and five feet wide. From Table C-1.2 three acceptable arrangements of members are listed.

Arrangement #l

Space 6X6 crossbraces at six feet horizontally and five feet vertically.
Space 8X8 wales at five feet vertically.
Space 2X6 uprights at two feet horizontally.

Arrangement #2

Space 6X8 crossbraces at eight feet horizontally and five feet vertically.
Space 10X10 wales at five feet vertically.
Space 2X6 uprights at two feet horizontally.

Arrangement #3

Space 8X8 crossbraces at 10 feet horizontally and five feet vertically.
Space 10X12 wales at five feet vertically.
Space 2X6 uprights at two feet vertically.

(3) Example 3.

A trench dug in Type C soil is 13 feet deep and five feet wide From Table C- 1.3 two acceptable arrangements of member s can be used.

Arrangement #1

Space 8X8 crossbraces at six feet horizontally and five feet vertically.
Space 10X12 wales at five feet vertically.
Position 2X6 uprights as closely together as possible.
If water must be retained use special tongue and groove uprights to form tight sheeting.

Arrangement #2

Space 8X30 crossbraces at eight feet horizontally and five feet vertically.
Space 12X12 wales at five feet vertically.
Position 2X6 uprights in a close sheeting configuration unless water pressure must be resisted. Tight sheeting must be used where water must be retained.

(4) Example 4.

A trench dug in Type C soil is 20 feet deep and 11 feet wide. The size and spacing of members for the section of trench that is over 15 feet in depth is determined using Table C-1.3. Only one arrangement of members is provided.

Space 8X10 crossbraces at six feet horizontally and five feet vertically.
Space 12X12 wales at five feet vertically.
Use 3X6 tight sheeting.

Use of Tables C-2.1 through C-2.3 would follow the same procedures.

(g) Notes for all Tables.

1. Member sizes at spacing other than indicated are to be determined as specified in Section 1541.1(c), "Design of Protective Systems."

2. When conditions are saturated or submerged use Tight Sheeting. Tight Sheeting refers to the use of specially-edged timber planks (e.g. tongue and groove) at least three inches thick, steel sheet piling, or similar construction that when driven or placed in position provide a tight wall to resist the lateral pressure of water and to prevent the loss of backfill material. Close Sheeting refers to the placement of planks side-by-side allowing as little space as possible between them.

3. All spacing indicated is measured center to center.

4. Wales to be installed with greater dimension horizontal.

5. If the vertical distance from the center of the lowest crossbrace to the bottom of the trench exceeds two and one-half feet, uprights shall be firmly embedded or a mudsill shall be used. When the uprights are embedded, the vertical distance from the center of the lowest crossbrace to the bottom of the trench shall not exceed 36 inches. When mudsills are used, the vertical distance shall not exceed 42 inches. Mudsills are wales that are installed at the toe of the trench side.

6. Trench jacks may be used in lieu of or in combination with timber crossbraces.

7. Placement of crossbraces. When the vertical spacing of crossbraces is four feet, place the top crossbrace no more than two feet below the top of the trench. When the vertical spacing of crossbraces is five feet, place the top crossbrace no more than 2.5 feet below the top of the trench.

TABLE C-1 1 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE A: $P_a = 25 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (ACTUAL) AND SPACING OF MEMBERS **													
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE	4	5	6	8
5 TO 10	UP TO 6	4X4	4X4	4X6	6X6	6X6	4	Not Req'd	---				2X6	
	UP TO 8	4X4	4X4	4X6	6X6	6X6	4	Not Req'd	---					2X8
	UP TO 10	4X6	4X6	4X6	6X6	6X6	4	8X8	4			2X6		
	UP TO 12	4X6	4X6	6X6	6X6	6X6	4	8X8	4				2X6	
10 TO 15	UP TO 6	4X4	4X4	4X6	6X6	6X6	4	Not Req'd	---				3X8	
	UP TO 8	4X6	4X6	6X6	6X6	6X6	4	8X8	4		2X6			
	UP TO 10	6X6	6X6	6X6	6X8	6X8	4	8X10	4			2X6		
	UP TO 12	6X6	6X6	6X6	6X8	6X8	4	10X10	4				3X8	
15 TO 20	UP TO 6	6X6	6X6	6X6	6X8	6X8	4	6X8	4	3X6				
	UP TO 8	6X6	6X6	6X6	6X8	6X8	4	8X8	4	3X6				
	UP TO 10	8X8	8X8	8X8	8X8	8X10	4	8X10	4	3X6				
	UP TO 12	8X8	8X8	8X8	8X8	8X10	4	10X10	4	3X6				
OVER 20	SEE NOTE 1													

* Mixed oak or equivalent with a bending strength not less than 850 psi.

** Manufactured members of equivalent strength may be substituted for wood.

TABLE C-1.2 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE B: $P_a = 45 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (ACTUAL) AND SPACING OF MEMBERS **													
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE	2	3		
5 TO 10	UP TO 6	4X6	4X6	6X6	6X6	6X6	5	6X8	5			2X6		
	UP TO 8	6X6	6X6	6X6	6X8	6X8	5	8X10	5			2X6		
	UP TO 10	6X6	6X6	6X6	6X8	6X8	5	10X10	5			2X6		
	See Note 1													
10 TO 15	UP TO 6	6X6	6X6	6X6	6X8	6X8	5	8X8	5		2X6			
	UP TO 8	6X8	6X8	6X8	8X8	8X8	5	10X10	5		2X6			
	UP TO 10	8X8	8X8	8X8	8X8	8X10	5	10X12	5		2X6			
	See Note 1													
15 TO 20	UP TO 6	6X8	6X8	6X8	8X8	8X8	5	8X10	5	3X6				
	UP TO 8	8X8	8X8	8X8	8X8	8X10	5	10X12	5	3X6				
	UP TO 10	8X10	8X10	8X10	8X10	10X10	5	12X12	5	3X6				
	See Note 1													
OVER 20	SEE NOTE 1													

* Mixed oak or equivalent with a bending strength not less than 850 psi.

** Manufactured members of equivalent strength may be substituted for wood.

TABLE C - 1.3 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE C: $P_a = 80 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (ACTUAL) AND SPACING OF MEMBERS **													
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE				
5 TO 10	UP TO 6	6X8	6X8	6X8	8X8	8X8	5	8X10	5	2X6				
	UP TO 8	8X8	8X8	8X8	8X8	8X10	5	10X12	5	2X6				
	UP TO 10	8X10	8X10	8X10	8X10	10X10	5	12X12	5	2X6				
	See Note 1													
10 TO 15	UP TO 6	8X8	8X8	8X8	8X8	8X10	5	10X12	5	2X6				
	UP TO 8	8X10	8X10	8X10	8X10	10X10	5	12X12	5	2X6				
	See Note 1													
	See Note 1													
15 TO 20	UP TO 6	8X10	8X10	8X10	8X10	10X10	5	12X12	5	3X6				
	See Note 1													
	See Note 1													
	See Note 1													
OVER 20	SEE NOTE 1													

* Mixed oak or equivalent with a bending strength not less than 850 psi.

** Manufactured members of equivalent strength may be substituted for wood.

TABLE C-2.1 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE A: $P_a = 25 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (S4S) AND SPACING OF MEMBERS **													*
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE	4	5	6	8
5 TO 10	UP TO 6	4X4	4X4	4X4	4X4	4X6	4	Not Req'd	Not Req'd				4X6	
	UP TO 8	4X4	4X4	4X4	4X6	4X6	4	Not Req'd	Not Req'd					4X8
	UP TO 10	4X6	4X6	4X6	6X6	6X6	4	8X8	4			4X6		
	UP TO 12	4X6	4X6	4X6	6X6	6X6	4	8X8	4				4X6	
10 TO 15	UP TO 6	4X4	4X4	4X4	6X6	6X6	4	Not Req'd	Not Req'd				4X10	
	UP TO 8	4X6	4X6	4X6	6X6	6X6	4	6X8	4		4X8			
	UP TO 10	6X6	6X6	6X6	6X6	6X6	4	8X8	4			4X8		
	UP TO 12	6X6	6X6	6X6	6X6	6X6	4	8X10	4		4X6		4X10	
15 TO 20	UP TO 6	6X6	6X6	6X6	6X6	6X6	4	6X8	4	3X6				
	UP TO 8	6X6	6X6	6X6	6X6	6X6	4	8X8	4	3X6	4X12			
	UP TO 10	6X6	6X6	6X6	6X6	6X8	4	8X10	4	3X6				
	UP TO 12	6X6	6X6	6X6	6X8	6X8	4	8X12	4	3X6	4X12			
OVER 20	SEE NOTE 1													

*Douglas fir or equivalent with a bending strength not less than 1500 psi.

**Manufactured members of equivalent strength may be substituted for wood.

TABLE C-2.2 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE B: $P_a = 45 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (S4S) AND SPACING OF MEMBERS **													
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE	2	3	4	6
5 TO 10	UP TO 6	4X6	4X6	4X6	6X6	6X6	5	6X8	5			3X12 4X8		4X12
	UP TO 8	4X6	4X6	6X6	6X6	6X6	5	8X8	5		3X6		4X8	
	UP TO 10	4X6	4X6	6X6	6X6	6X8	5	8X10	5			4X8		
	See Note 1													
10 TO 15	UP TO 6	6X6	6X6	6X6	6X8	6X8	5	8X8	5	3X6	4X10			
	UP TO 8	6X8	6X8	6X8	8X8	8X8	5	10X10	5	3X6	4X10			
	UP TO 10	6X8	6X8	8X8	8X8	8X8	5	10X12	5	3X6	4X10			
	See Note 1													
15 TO 20	UP TO 6	6X8	6X8	6X8	6X8	8X8	5	8X10	5	4X6				
	UP TO 8	6X8	6X8	6X8	8X8	8X8	5	10X12	5	4X6				
	UP TO 10	8X8	8X8	8X8	8X8	8X8	5	12X12	5	4X6				
	See Note 1													
OVER 20	SEE NOTE 1													

*Douglas fir or equivalent with a bending strength of 1500 psi.

**Manufactured nenbers of equivalent strength nay be substituted for wood.

TABLE C-2.3 TIMBER TRENCH SHORING - - MINIMUM TIMBER REQUIREMENTS*

SOIL TYPE C: $P_a = 80 \times H + 72$ psf (2 Ft. Surcharge)

DEPTH OF TRENCH (FEET)	SIZE (S4S) AND SPACING OF MEMBERS **													
	CROSS BRACES							VALES		UPRIGHTS				
	HORIZ. SPACING (FEET)	WIDTH OF TRENCH (FEET)					VERT. SPACING (FEET)	SIZE (IN)	VERT. SPACING (FEET)	MAXIMUM ALLOWABLE HORIZONTAL SPACING (FEET)				
		UP TO 4	UP TO 6	UP TO 9	UP TO 12	UP TO 15				CLOSE				
5 TO 10	UP TO 6	6X6	6X6	6X6	6X8	8X8	5	8X8	5	3X6				
	UP TO 8	6X6	6X6	6X6	8X8	8X8	5	10X10	5	3X6				
	UP TO 10	6X6	6X6	8X8	8X8	8X8	5	10X12	5	3X6				
	See Note 1													
10 TO 15	UP TO 6	6X6	6X6	6X6	8X8	8X8	5	10X10	5	4X6				
	UP TO 8	8X8	8X8	8X8	8X8	8X8	5	12X12	5	4X6				
	See Note 1													
	See Note 1													
15 TO 20	UP TO 6	8X8	8X8	8X8	8X10	8X10	5	10X12	5	4X6				
	See Note 1													
	See Note 1													
	See Note 1													
OVER 20	SEE NOTE 1													

* Douglas fir or equivalent with a bending strength not less than 1500 psi.

** Manufactured members of equivalent strength may be substituted for wood.

Appendix D to Section 1541.1

ALUMINUM HYDRAULIC SHORING FOR TRENCHES

(a) Scope.

This appendix contains information that can be used when aluminum hydraulic shoring is provided as a method of protection against cave-ins in trenches that do not exceed 20 feet in depth. This appendix must be used when design of the aluminum hydraulic protective system cannot be performed in accordance with Section 1541.1(c)(2).

(b) Soil Classification.

In order to use data presented in this appendix, the soil type or types in which the excavation is made must first be determined using the soil classification method set forth in Appendix A of this Article.

(c) Presentation of Information.

Information is presented in several forms as follows:

(1) Information is presented in tabular form in Tables D-1.1, D-1.2, D-l.3 and D-1.4. Each table presents the maximum vertical and horizontal spacings that may be used with various aluminum member sizes and various hydraulic cylinder sizes. Each table contains data only for the particular. soil type in which the excavation or portion of the excavation is made. Tables D-1.1 and D-1.2 are for the vertical shores in Types A and B soil. Tables D-l.3 and D-1.4 are for horizontal waler systems in Type B and C soil.

(2) information concerning the basis of the tabular data and the limitations of the data is presented in Section (d) of this appendix.

(3) Information explaining the use of the tabular data is presented in Section (e) of this appendix.

(4) Information illustrating the use of the tabular data is presented in Section (f) of the appendix.

(5) Miscellaneous notations (footnotes) regarding Table D-1.1 through D-1.4 are presented in Section (g) of this appendix.

(6) Figures, illustrating typical installations of hydraulic shoring, are included just prior to the Tables. The illustrations page is entitled "Aluminum Hydraulic Shoring: Typical Installations."

(d) Basis and Limitations of the data.

(1) Vertical shore rails and horizontal wales are those that meet the Section Modulus requirements in the D-1 Tables. Aluminum material is 6061-T6 or material of equivalent strength and properties.

(2) Hydraulic cylinders specifications.

(A) 2-inch cylinders shall be a minimum 2-inch inside diameter with a minimum safe working capacity of no less than 18,000 pounds axial compressive load at maximum extension. Maximum extension is to include full range of cylinder extensions as recommended by product manufacturer.

(B) 3-inch cylinders shall be a minimum 3-inch inside diameter with a safe working capacity of not less than 30,000 pounds axial compressive load at extensions as recommended by product manufacturer.

(3) Limitation of application.

(A) It is not intended that the aluminum hydraulic specification apply to every situation that may be experienced in the field. These data were developed to apply to the situations that are most commonly experienced in current trenching practice. Shoring systems for use in situations that are not covered by the data in this appendix must be otherwise designed as specified in Section 1541.1(c).

(B) When any of the following conditions are present, the members specified in the Tables are not considered adequate. In this case, an alternative aluminum hydraulic shoring system or other type of protective system must be designed in accordance with Section 1541.1.

1. When vertical loads imposed in crossbraces exceed a 100 pound gravity load distributed on a one foot section of the center of the hydraulic cylinder.

2. When surcharge loads are present from equipment weighing in excess of 20,000 pounds.

3. When only the lower portion of the trench is shored and the remaining portion of the trench is sloped or benched unless: The sloped portion is sloped at an angle less steep than three horizontal to one vertical; or the members are selected from tables for use at a depth which is determined from the top of the overall trench, and not from the toe of the sloped portion.

(e) Use of Tables D-1.1, D-1.2, D-l.3 and D-1.4.

The members of the shoring system that are to be selected using this information are the hydraulic cylinders, and either the vertical shores or the horizontal wales. When a waler system is used the vertical timber sheeting to be used is also selected from these tables. The Tables D-l.1 and D-1.2 for vertical shores are used in Type A and B soils that do not require sheeting. Type B soils that may require sheeting, and Type C soils that always require sheeting, are found in the horizontal wale Tables D-1.3 and D-1.4. The soil type must first be determined in accordance with the soil classification system described in Appendix A to Section 1541.1. Using the appropriate table, the selection of the size and spacing of the members is made. The selection is based on the depth and width of the trench where the members are to be installed. In these tables the vertical spacing is held constant at four feet on center. The tables show the maximum horizontal spacing of cylinders allowed for each size of wale in the waler system tables, and in the vertical shore tables, the hydraulic cylinder horizontal spacing is the same as the vertical shore spacing.

(f) Example to Illustrate the Use of the Tables:

(1) Example 1:

A trench dug in Type A soil is 6 feet deep and 3 feet wide.

From Table D-1.1: Find vertical shores and 2 inch diameter cylinders spaced 8 feet on center (o.c.) horizontally and 4 feet on center (o.c.) vertically. (See Figures 1 & 3 for typical installations.)

(2) Example 2:

A trench is dug in Type B soil that does not require sheeting, 13 feet deep and 5 feet wide.

From Table D-1.2: Find vertical shores and 2 inch diameter cylinders spaced 6.5 feet o.c. horizontally and 4 feet o.c. vertically. (See Figures 1 & 3 for typical installations.)

(3) Example 3:

A trench is dug in Type B soil that does not require sheeting, but does experience some minor raveling of the trench face. The trench is 16 feet deep and 9 feet wide.

From Table D-1.2: Find vertical shores and 2 inch diameter cylinder (with special oversleeves as designated by footnote #2) spaced 5.5 feet o.c. horizontally and 4 feet o.c. vertically. Plywood (per footnote (g)(7) to the D-l Table) should be used behind the shores. (See Figures 2 & 3 for typical installations.)

(4) Example 4:

A trench is dug in previously disturbed Type B soil, with characteristics of a Type C soil, and will require sheeting. The trench is 18 feet deep, and 12 feet wide. 8 Foot horizontal spacing between cylinders is desired for working space.

From Table D-1.3: Find horizontal wale with a section modulus of 14.0 spaced at 4 feet o.c. vertically and 3 inch diameter cylinder spaced at 9 feet maximum o.c. horizontally, 3X12 timber sheeting is required at close spacing vertically. (See Figure 4 for typical installation.)

(5) Example 5:

A trench is dug in Type C soil, 9 feet deep and 4 feet wide. Horizontal cylinder spacing in excess of 6 feet is desired for working space.

From Table D-1.4: Find horizontal wale with a section modulus of 7.0 and 2 inch diameter cylinders spaced at 6.5 feet o.c. horizontally. Or, find horizontal wale with a 14.0 section modulus and 3 inch diameter cylinder spaced at 10 feet o.c. horizontally. Both wales are spaced 4 feet o.c. vertically. 3X12 timber sheeting is required at close spacing vertically. (See Figure 4 for typical installation.)

ALUMINUM HYDRAULIC SHORING
TYPICAL INSTALLATIONS

FIGURE NO.1
VERTICAL ALUMINUM
HYDRAULIC SHORING
(SPOT BRACING)

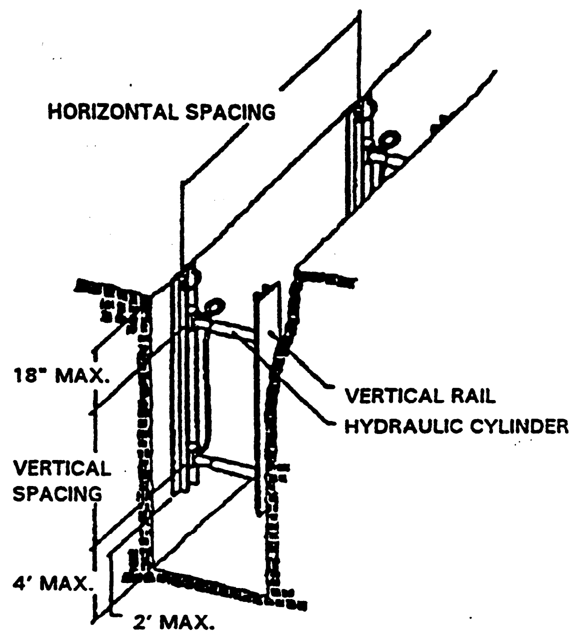
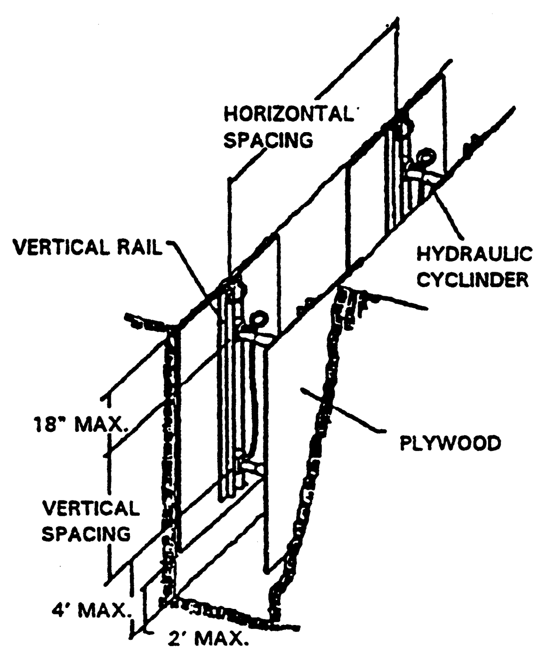


FIGURE NO.2
VERTICAL ALUMINUM
HYDRAULIC SHORING
(WITH PLYWOOD)



ALUMINUM HYDRAULIC SHORING
TYPICAL INSTALLATIONS

FIGURE NO.3

VERTICAL ALUMINUM HYDRAULIC SHORING (STACKED)

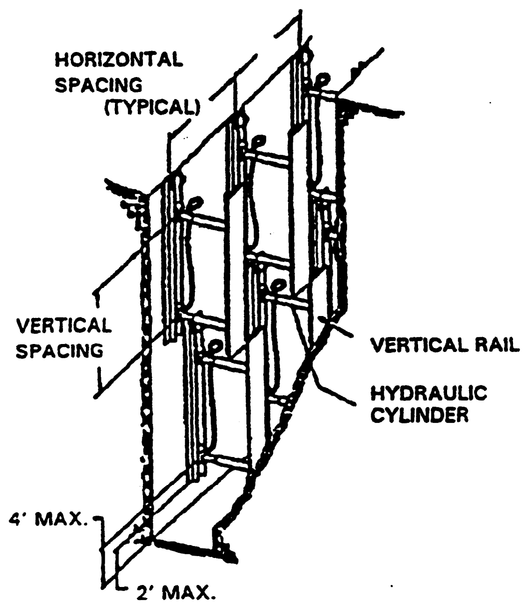
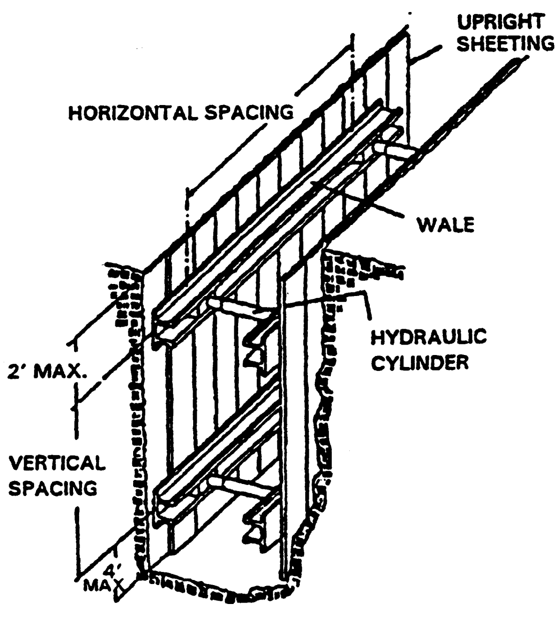


FIGURE NO.4

ALUMINUM HYDRAULIC SHORING WALER SYSTEM (TYPICAL)



(g) Footnotes, and general notes for Tables D-1.1, D-1.2, D-1.3, and D-1.4.

(1) For applications other than those listed in the tables, refer to Section 1541.1(c)(2) for use of manufacturer's tabulated data. For trench depths in excess of 20 feet, refer to Section 1541.1(c)(2) and 1541.1(c)(3).

(2) 2-inch diameter cylinders, at this width, shall have structural steel tube (3.5 X 3.5 X 0.1875) oversleeves, or structural oversleeves of manufacturer's specification, extending the full, collapsed length..

(3) Hydraulic cylinder capacities.

(A) 2-Inch cylinders shall be a minimum 20 inch inside diameter with a safe working capacity of not less than 18,000 pounds axial compressive load at maximum extension. Maximum extension is to include full range of cylinde r extension as recommended by product manufacturer.

(B) 3-Inch cylinders shall be a minimum 3-inch inside diameter with a safe working capacity of not less than 30,000 pounds axial compressive load at maximum extension. Maximum extension is to include full range of cylinder extensions as recommended by product manufacturer.

(4) All spacing indicated is measured center to center.

(5) Vertical shoring rails shall have a minimum section modulus of 0.40 inch.

(6) When vertical shores are used, there must be a minimum of three shores spaced equally, horizontally, in a group.

(7) Plywood shall be 1.125 inches thick of wood or 0.75 inch thick, 14 ply, arctic white birch (Finland form). Please note that plywood is not intended as a structural member, but only for prevention of local raveling (sloughing of the trench face) between shores.

(8) See Appendix C for timber specifications.

(9) Wales are calculated for simple span conditions.

(10) See Appendix D, Section (d), for basis and limitations of the data.

TABLE D-1.1 ALUMINIM HYDRAULIC SHORING
SOIL TYPE A VERTICAL SHORES

DEPTH OF TRENCH (FEET)	HYDRAULIC CYLINDERS				
	MAXIMUM HORIZONTAL SPACING (FEET)	MAXIMUM VERTICAL SPACING (FEET)	WIDTH OF TRENCH (FEET)		
			UP TO 8	OVER 8 UP TO 12	OVER 12 UP TO 15
OVER 5 UP TO 10	8	4	2 INCH DIAMETER	2 INCH DIAMETER NOTE (2)	3 INCH DIAMETER
OVER 10 UP TO 15	8				
OVER 15 UP TO 20	7				
OVER 20	NOTE (1)				

Footnotes to tables, and general notes on hydraulic shoring, are found in Appendix D, Item (g).
Note (1): See Appendix D, Item (g)(1).
Note (2): See Appendix D, Item (g)(2)

TABLE D 1.2
SOIL TYPE B

ALUMINIM HYDRAULIC SHORING
VERTICAL SHORES

DEPTH OF TRENCH (FEET)	HYDRAULIC CYLINDERS				
	MAXIMUM HORIZONTAL SPACING (FEET)	MAXIMUM VERTICAL SPACING (FEET)	WIDTH OF TRENCH (FEET)		
			UP TO 8	OVER 8 UP TO 12	OVER 12 UP TO 15
OVER 5 UP TO 10	8	4	2 INCH DIAMETER	2 INCH DIAMETER NOTE (2)	3 INCH DIAMETER
OVER 10 UP TO 15	6.5				
OVER 15 UP TO 20	5.5				
OVER 20	NOTE (1)				

Footnotes to tables, and general notes on hydraulic shoring, are found in Appendix D, Item (g).
Note (1): See Appendix D, Item (g) (1).
Note (2): See Appendix D, Item (g) (2).

TABLE D-1 3 ALUMINUM HYDRAULIC SHORING
WALER SYSTEM - - FOR SOIL TYPE B

DEPTH OF TRENCH (FEET)	WALES		HYDRAULIC CYLINDERS						TIMBER UPRIGHTS		
			WIDTH OF TRENCH (FEET)						MAX. HORIZ. SPACING (ON CENTER)		
	VERTICAL SPACING (FEET)	SECTION MODULUS * (IN ^{3})	UP TO 8		OVER 8 UP TO 12		OVER 12 UP TO 15		SOLID SHEET	2 FT	3 FT
			HORIZ SPACING	CYLINDER DIAMETER	HORIZ SPACING	CYLINDER DIAMETER	HORIZ SPACING	CYLINDER DIAMETER			
OVER 5 UP TO 10	4	3.5	8.0	2 IN	8.0	2 IN. NOTE(2)	8.0	3 IN	—	—	3X12
		7.0	9.0	2 IN	9.0	2 IN. NOTE(2)	9.0	3 IN			
		14.0	12.0	3 IN	12.0	3 IN	12.0	3 IN			
OVER 10 UP TO 15	4	3.5	6.0	2 IN	6.0	2 IN. NOTE(2)	6.0	3 IN	—	3X12	—
		7.0	8.0	3 IN	8.0	3 IN	8.0	3 IN			
		14.0	10.0	3 IN	10.0	3 IN	10.0	3 IN			
OVER 15 UP TO 20	4	3.5	5.0	2 IN	5.5	2 IN. NOTE(2)	5.5	3 IN	3X12	—	—
		7.0	6.0	3 IN	6.0	3 IN	6.0	3 IN			
		14.0	9.0	3 IN	9.0	3 IN	9.0	3 IN			
OVER 20	NOTE (1)										

Footnotes to tables, and general notes on hydraulic shoring, are found in Appendix D, Item (g).

Notes (1): See Appendix D. Item (g)(1)

Notes (2): See Appendix D. Item (g)(2)

* Consult product manufacturer and/or qualified engineer for Section Modulus of available wales.

TABLE D-1.4

ALUMINUM HYDRAULIC SHORING
WALER SYSTEMS - - FOR SOIL TYPE C

DEPTH OF TRENCH (FEET)	WALES: VERTICAL SPACING (FEET)	WALES: SECTION MODULUS* (IN^3)	HYDRAULIC CYLINDERS, WIDTH OF TRENCH (FEET): UP TO 8 — HORIZ SPACING	UP TO 8 — CYLINDER DIAMETER	OVER 8 UP TO 12 — HORIZ SPACING	OVER 8 UP TO 12 — CYLINDER DIAMETER	OVER 12 UP TO 15 — HORIZ SPACING	OVER 12 UP TO 15 — CYLINDER DIAMETER	TIMBER UPRIGHTS, MAX. HORIZ. SPACING (ON CENTER): SOLID SHEET	2 FT	3 FT
OVER 5 UP TO 10	4	3.5	6.0	2 IN	6.0	2 IN. NOTE(2)	6.0	3 IN	3X12	—	—
		7.0	6.5	2 IN	6.5	2 IN. NOTE(2)	6.5	3 IN			
		14.0	10.0	3 IN	10.0	3 IN	10.0	3 IN			
OVER 10 UP TO 15	4	3.5	4.0	2 IN	4.0	2 IN. NOTE(2)	4.0	3 IN	3X12	—	—
		7.0	5.5	3 IN	5.5	3 IN	5.5	3 IN			
		14.0	8.0	3 IN	8.0	3 IN	8.0	3 IN			
OVER 15 UP TO 20	4	3.5	3.5	2 IN	3.5	2 IN. NOTE(2)	3.5	3 IN	3X12	—	—
		7.0	5.0	3 IN	5.0	3 IN	5.0	3 IN			
		14.0	6.0	3 IN	6.0	3 IN	6.0	3 IN			
OVER 20	NOTE (1)										

Footnotes to tables, and general notes on hydraulic shoring, are found in Appendix D, Item (g).

Notes (1): See Appendix D, Item (g)(1)

Notes (2): See Appendix D, Item (g)(2)

* Consult product manufacturer and/or qualified engineer for Section Modulus of available wales.

Appendix E to Section 1541.1

ALTERNATIVES TO TIMBER SHORING

FIGURE 1
ALUMINUM HYDRAULIC SHORING

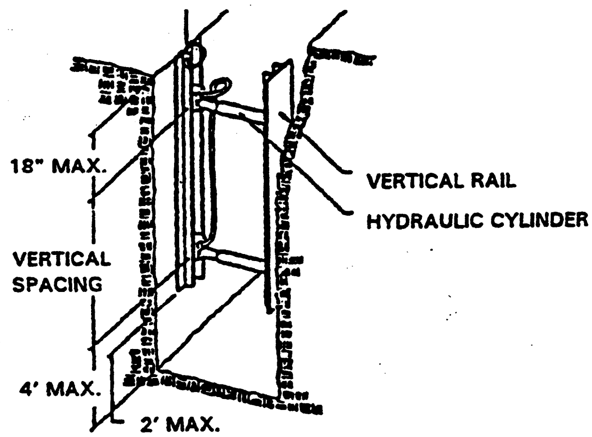
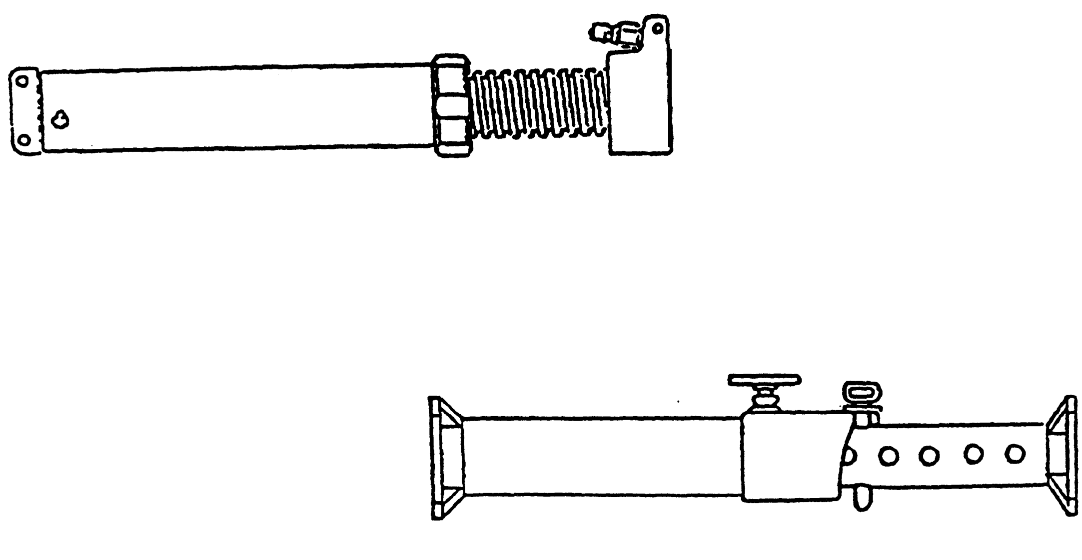


FIGURE 2
PNEUMATIC / HYDRALIC SHORING



F I G U R E 3

TRENCH JACKS (SCREW JACKS)

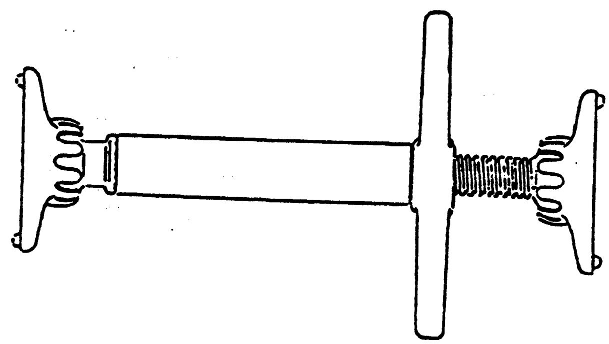
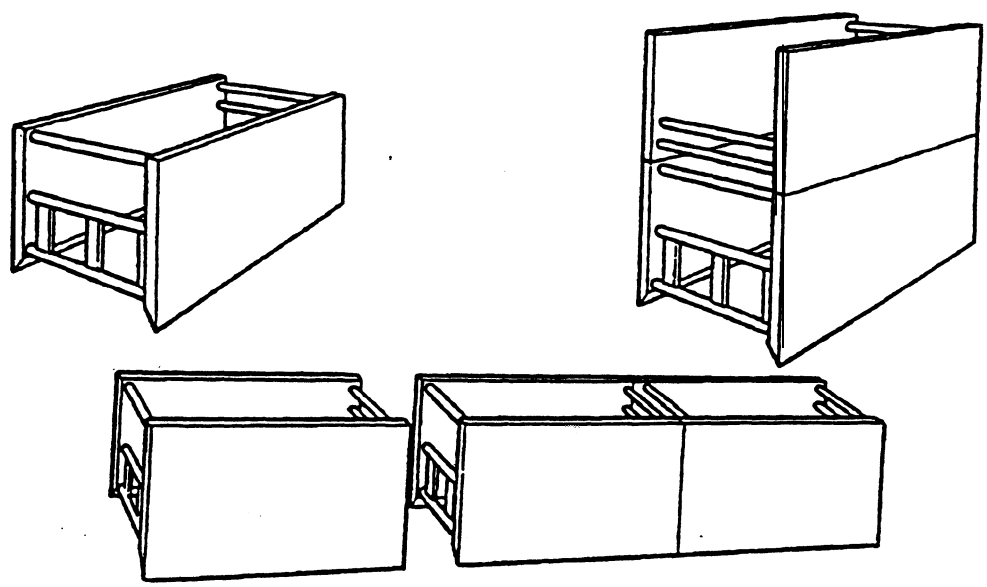


FIGURE 4

TRENCH SHIELDS



Appendix F to Section 1541.l

SELECTION OF PROTECTIVE SYSTEMS

The following figures are a graphic summary of the requirements contained in Article 6 for excavations 20 feet or less in depth. Protective systems for use in excavations more than 20 feet in depth must be designed by a registered professional engineer in accordance Section 1541.1 (b) and (c).

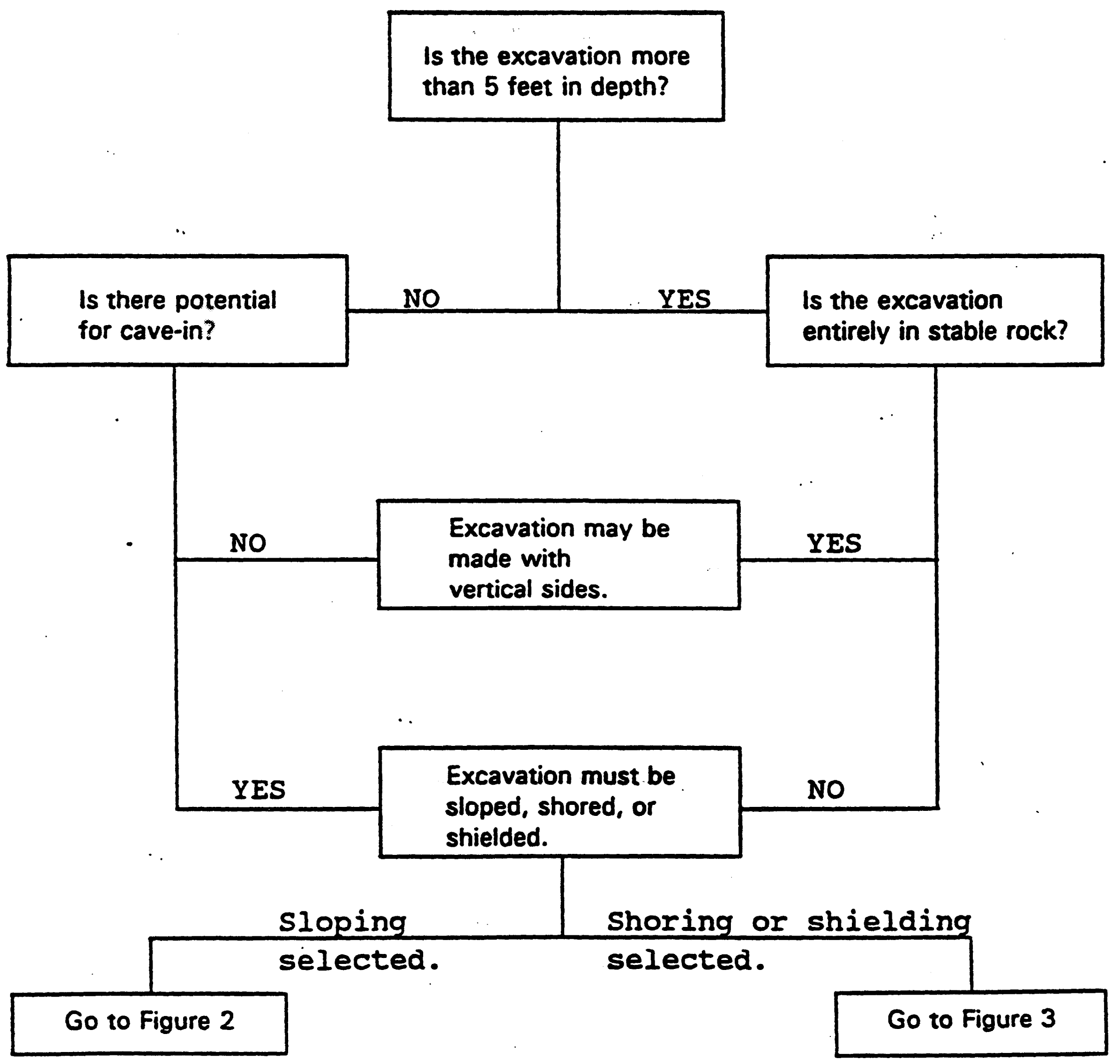


FIGURE 1 - PRELIMINARY DECISIONS

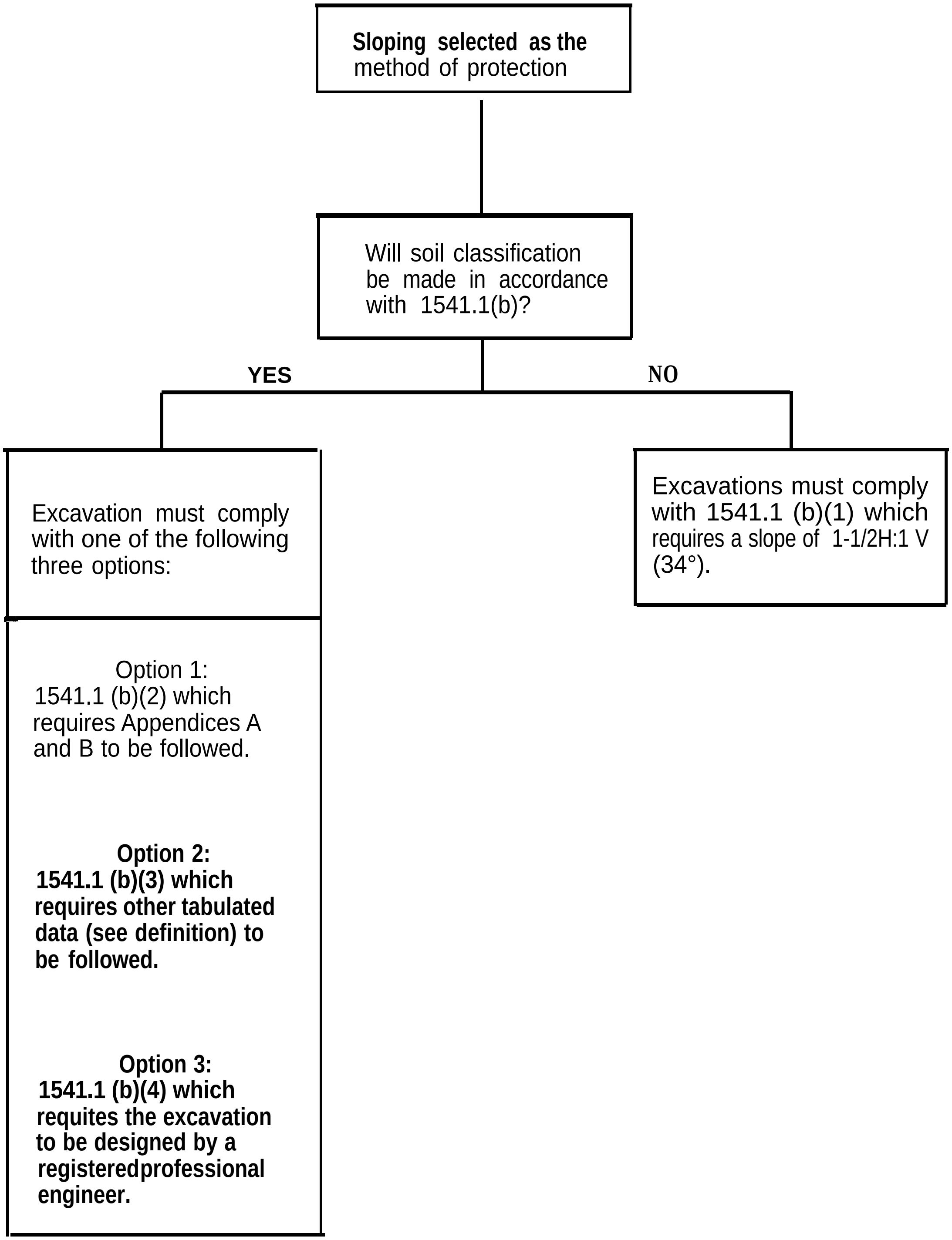


FIGURE 2 - SLOPING OPTIONS

Shoring or shielding selected as the method of protection.

Soil classification is required when shoring or shielding is used. The excavation must comply with one of the following four options:

Option 1:
1541.1 (c)(1) which requires Appendices A and C to be followed (e.g. timber shoring).

Option 2:
1541.l (c)(2) which requires manufacturers data to be followed (e.g. hydraulic shoring, trench jacks, air shores, shields).

Option 3:
1541.1 (c)(3) which requites tabulated data (see definition) to be followed (e.g. any system as per the tabulated data).

Option 4:
1541.1 (c)(4) which require the excavation to be designed by **a registered professional engineer** (e.g. any designed system).

FIGURE 3 - SHORING AND SHIELDING OPTIONS

Section 1541

SHAFTS

(a) General.

(1) All wells or shafts over S feet in depth into which employees are permitted to enter shall be retained with lagging, spiling, or casing.
EXCEPTION: Exploration shafts; see Section 1542(e).

(2) The lagging, spiling or casing shall extend at least one foot above ground level and shall be provided the full depth of the shaft or at least five feet into solid rock if possible.

(3) All wells, pits, shafts, caissons, etc., shall be barricaded or securely covered.

(4) Upon completion of exploration and similar operations, temporary wells, pits, shafts, etc., shall be backfilled.

(b) Sma11 Shafts in Hard Compact Soil.

Two inch (nominal) cribbing may be used in square shafts not over 4 feet square in hard compact soil. Each member shall be cut l/2 -way through the width of the member and dovetailed into position so each member will act as a shore as well as lagging. Strips shall be nailed in each corner to prevent the boards from dropping down.

(c) Shafts in Other Than Hard Compact Soil.

(3) A system of lagging supported by braces and corner posts shall be used for square or rectangular shafts. Corner posts of 4-inch by 4-inch material are normally acceptable in shafts 4 feet square, or smaller, if they are braced in each direction with horizontal 4-inch by 4-inch members at intervals not exceeding 4 feet. Braces and corner posts in larger shafts shall be correspondingly larger as determined be a civil engineer.

(2) Round shafts shall be completely lagged with 2-inch material which is supported at intervals not greater than 4 feet by means of adjustable rings of metal or timber that are designed to resist the collapsing force, or cased in a manner that provides equivalent protection.

(d) Exploration shafts.

Only a geotechnical specialist shall be permitted to enter an exploration shaft without lagging, spiling or casing for the purpose of subsurface investigations under the following conditions.

(1) Initial Inspection. The type of materials and stability characteristics of the exploration shaft shall be personally

observed and recorded by the geotechnical specialist during the drilling operation. Potentially unsafe exploration shafts shall not be entered.

(2) Surface Casing. The upper portion of the exploration shaft shall be equipped with a surface ring-collar to provide casing support of the material within the upper 4 feet of the exploration shaft. The ring-collar shall extend at least 1-foot above the ground surface.

(3) Gas Tests. Prior to entry into exploration shafts, tests and/or procedure&hall be instituted to assure that the atmosphere within the shaft does not contain dangerous air contamination or oxygen deficiency. These tests and/or l procedures shall be maintained while working within the shaft to assure that dangerous air contamination or oxygen deficiency will not occur. See Section 5156 of the General Industry Safety Orders.)

(4) Unstable Local Conditions. The geotechnical specialist shall not descend below any portion of any exploration shaft where caving or groundwater seepage is noted or suspected,

(5) Ladder and Cable Descents. A ladder may be used to inspect exploration shafts 20 feet or less in depth. In deeper exploration shafts, properly maintained mechanical hoisting devices with a safety factor of at least 6 shall be provided and used. Such devices shall be under positive control of the operator being positive powered up and down with fail-safe breaks.

(6) Emergency Standby Employee. An emergency standby employee shall be positioned at the surface near the exploration shaft whenever a geotechnical specialist is inside the shaft.

(7) Communication. A two-way, electronically-operated communication system shall be in operation between the standby employee and the geotechnical specialist whenever boring inspections ate being made in exploration shafts over 20 feet in depth or when ambient noise levels make communication difficult.

(8) Safety Equipment. The following safety equipment shall be used to protect the geotechnical specialist:

(A) An approved safety harness which will suspend a person upright and that is securely attached to the hoist cable.

(B) A 12-inch to 18-inch diameter steel cone shaped headguard/deflector that is attached to the hoist cable above the harness.

(C) A hoist cable having a minimum diameter of 5/16 inches.

(D) Approved head protection. (See Section 1515.)

(9) Electrical Devices. All electrical devices used within the exploration shaft by the geotechnical specialist shall be approved for hazardous locations.

(10) Surface Hazards. The storage and use of flammable or other dangerous materials shall be controlled at the surface to prevent them from entering the exploration shaft.

Section 1543

COFFERDAMS

(a) If overtopping of the cofferdam by high waters is possible, means shall be provided for controlled flooding of the work area,

(b) Warning signs for evacuation of employees in case of emergency shall be developed and posted.

(c) Cofferdam walkways, bridges, or ramps with at least two means of rapid exit, shall be provided with guardrails as specified in Section 1620.

(d) Cofferdams located close to navigable shipping channels shall be protected from vessels in transit, where possible.

APPENDIX B

SOIL TEST METHODS

Soil testing can be divided into three general categories: in situ testing, laboratory testing, and soil classification. In situ tests are those conducted in the field on a soil while it is still in the ground. Laboratory testing involves extracting a sample of soil, transporting it to either a field or office laboratory, and manipulating it in some way so as to acquire information about the deposit from which it came. Soil classification is the determination of various physical properties which can be used to evaluate the uniformity of a depositand to provide general correlations with engineering properties.

The-most important soil tests for trench and shoring work are those used to determine soil shear strength. Some of the more significant tests used for soil classification and the determination of shear strength parameters are listed below. The applicable American Society for Testing and Materials (ASTM) test designation is shown in parenthesis.

IN SITU TESTS

FIELD VANE SHEAR TEST (ASTM D2573)

The vane shear test consists of pushing a four bladed vane into undistributed soil at the bottom of a bore hole and rotating it from the surface to determine the torsional force required to cause a cylindrical volume of soil to be sheared by the vane. The torsional force is then related to a undrained shear strength (S_u) using a conversion factor which depends on the dimensions and shape of the vane,

To assure undrained conditions the soil in which this test is conducted must have low permeability. As such, this test is used primarily in fine grained soils. In addition, the soil should be free of gravel or large shell particles which would influence the test results.

This test attempts to provide a direct measurement of S_u and is therefore preferred to an estimation of q_u from the standard penetration test.

A hand held version of the vane shear test uses a device known as the torvane on samples recovered from a test boring. The torvane is one inch in diameter and has blades 0.2 inch long. Undrained shear strength is measured by inserting the torvane into the soil sample andtwisting. Undrained shear strength is indicated by a dial mounted on-the handle. The torvane test does not have an ASTM designation.

CONE PENETRATION TEST (ASTM D3441)

The cone penetration test (CPT) consists of pushing a conically tipped, cylindrical probe into the ground at a slow rate. The probe is instrumented with strain gages used to measure resisting force against the tip (10 cm^2 cross-sectional area) and along the side (15O cm^2 area) while the probe is advancing downward. A computer is typically used to control the advance of the probe, acquisition and recording of data. As such, many readings can be obtained and a nearly continuous record of subsurface information collected.

Relationships exist which correlate subsurface resistance data collected with this instrument to: soil description; relative density for granular soils; and undrained shear strength(SU) for fine grained soils.

Because this probe is advanced slowly, under so called quasi-static loading, estimation of s_u made to using CPT data is preferable to estimation of q_u from the Standard Penetration Test where the sampler is advanced under dynamic loading and unknown damping forces may influence the data in soils with low permeability.

STANDARD PENETRATION TEST (ASTM D1588)

The Standard Penetration Test (SPT) consists of driving a 1.4 inch I.D. (2.0 inch O.D.) sampler 18 inches into the bottom of a bore hole using a 140 hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the SPT blow count (N).

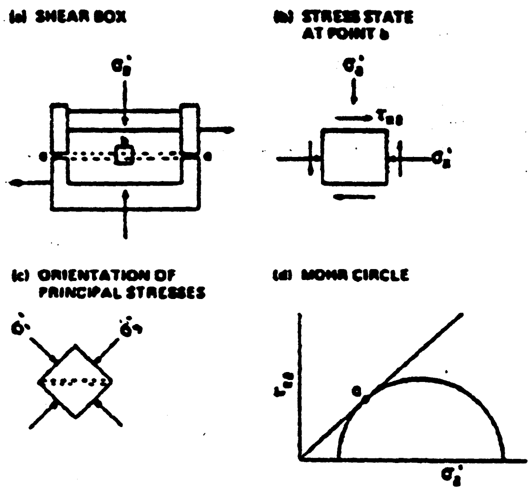
In addition to N, this test provides a means of retrieving soil samples for visual description or laboratory tests appropriate for highly disturbed soil. Empirical relationships exist which can be used to correlate N to relative density of granular soils and unconfined compressive strength (q_u) for fine grained soils.

APPENDIX B

LABORATORY TESTS

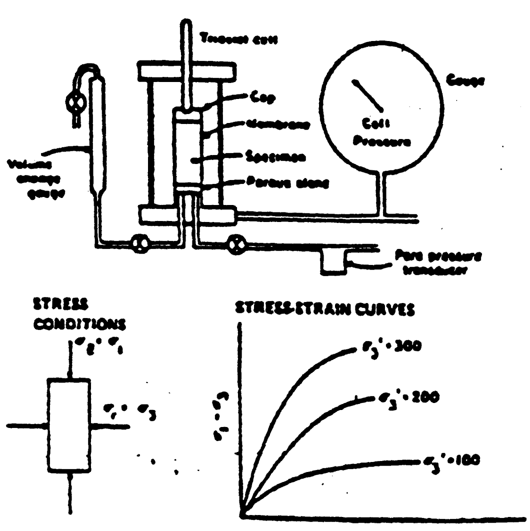
DIRECT SHEAR TEST (ASTM D3080)

In a direct shear test, the soil is placed in a split shear box and stressed to failure by moving one part of the container relative to the other. The specimen is subjected to a normal force and a horizontal shear force. The normal force is kept constant throughout the test and the shear force is increased usually at constant rate of strain to cause the specimen to shear along a predetermined horizontal plane. If it is assumed that the horizontal plane is equivalent to the failure plane for the soil, then the friction angle can be calculated from the results of a series of tests performed at various normal stresses.



The direct shear test offers the easiest way to measure the friction angle of a sand or other dry soil. It is not useful for testing soils containing water unless they are free draining and-have a very high permeability, because it is difficult to control the drainage and thus volume changes during testing. For this reason, the direct shear tests should be used with caution in determining the undrained shear strength of cohesive soils.

TRIAXIAL COMPRESSION TEST (ASTM D2850)



The triaxial test is most versatile test available to determine the stress-strainproperties of soil. For the most common triaxial test (the triaxial compression test) a cylindrical specimen is sealed in a rubber membrane and placed in a cell and subjected to a uniform fluid pressure in the horizontal and vertical directions. A vertical load is applied axially to the specimen increasing the axial stress until the specimen fails. Under these conditions, the axial stress is the major principal stress,

σ_1, and the intermediate and minor principal stresses, σ_2, and σ_3 respectively, are equal to the cell pressure. The increment of axial stress, $\sigma_1 - \sigma_3$, is referred to as the deviator stress or principal stress difference.

Drainage of water from the specimen is controlled by connections to the bottom cap. Change in sample volume is measured if drainage is allowed. Alternatively, pore water pressures may be measured if no drainage is allowed. Triaxial tests are generally classified as to one of three conditions of drainage during application of the cell pressure and loading. The three drainage conditions for testing are the (UU), (CU), and (CD) referenced below.

Unconsolidated-Undrained (UU)

No drainage is allowed during application of the cell pressure or confining stress and no drainage is allowed during application of the deviator stress. This test is generally performed on undisturbed saturated samples of fine grained soils (clay, silt and peat) to measure the in situ undrained shear strength ($\phi = 0$ analysis). For soils which exhibit peak stress-strain characteristics, the failure stress is taken as the maximum deviator stress ($\sigma_1 - \sigma_3$) measured during the test. For soils which exhibit an increasing deviator stress with strain, the failure stress. is generally taken as the deviator stress at a strain equal to 20 percent. Theundrained shear strength, s_u is taken as half the deviator stress or:

$$s_u = (\sigma_1 - \sigma_3)/2$$

The in situ undrained shear strength is applicable to conditions in which construction occurs rapidly enough so that no drainage and hence, no dissipation of excess pore pressures occur during construction. Examples of typical situations in which the in situ undrained shear strength would govern stability include construction of embankments on clay deposits or rapid loading of footings on clay.

Unconsolidated-undrained tests are also performed on samples of partially saturated cohesive soils. The principal application of tests on partially saturated samples is to earth-fill materials which are compacted under specified conditions of water content and density. It also applies to undisturbed samples of partially saturated and to samples recovered from existing fills. However, because the tests are performed on partially saturated soil, the deviator stress at failure will increase with continuing pressure.

The failure envelope expressed in terms of total stress is non-linear and values of C and ϕ can be reported only for specific ranges of confining pressures.

Consolidated-Undrained (CU)

Drainage is allowed during application of the confining stress so that the specimen is fully consolidated under this stress. No drainage is permitted during application of the deviator stress. This test is performed on undisturbed samples of cohesive soil, on reconstituted specimens of cohesionless soil and, in some instances, on undisturbed samples of cohesionless soils which have developed some apparent cohesion resulting from partial drainage.

Generally, the specimen is allowed to consolidate under a confining stress of known magnitude and is then failed under undrained conditions by applying an axial load. The volume change that occurs during consolidation should be measured. The results of CU tests, in terms of total stress or undrained shear strength, must be applied with caution because of uncertainties in the effects of stress history and stress system (isotropic consolidation) on the magnitude of strength increase with consolidation.

If the pore pressure is measured during the test, the results can be expressed in terms of effective stress, **c' and ϕ'.**

The principal application of results of CU tests on cohesive soils is to the situation where additional load is rapidly applied to soil that has been consolidated under previous loading (shear stresses). The principal application to cohesionless soils is to evaluate the stress-strain properties as a function of effective confining stress.

Consolidated-Drained (CD)

Drainage is permitted both during application of the confining stress and the deviator stress, such that the specimen is fully consolidated under the confining stress and no excess pore pressures are developed during testing. Consolidated drained tests are performed on all types of soil samples, including undisturbed, compacted and reconstituted samples.

In a standard test, the specimen is allowed to consolidate under a predetermined confining stress and the specimen is then sheared by increasing the axial load at a sufficiently slow rate to prevent development of excess pore pressure. Since the excess pore pressure is zero, the applied stresses are equal to the effective stresses and the strength parameters, **c' and φ'** are obtained directly from the stresses at failure. The volume changes that occur during consolidation and shear should be measured.

The principal application of the results of CD tests on cohesive soils is for the case where either construction will occur at a sufficiently slow rate that no excess pore pressures will develop or sufficient time will have elapsed that all excess pore pressures will have dissipated (i.e. long term conditions).

The principal application to cohesionless soils is to determine the effective friction angle.

UNCONFINED COMPRESSION TEST (ASTM D2166)

The unconfined compression test measures the compressive strength (q_u) of a cylinder of cohesive soil which has no lateral confinement (unconfined). The undrained shear strength (S_u) is normally taken as approximately equal to one-half of the compressive strength. This test can be considered as a special case of the UU triaxial test in which the confining stress is zero.

The test is generally performed on an undisturbed specimen of cohesive soil at its natural water content. Cohesionless soils, such as sands and non-plastic silts and fissured or layered materials, should not be tested unconfined because the shear strength of these types of soils is a function of the in situ confining stress.

A hand held device known as the pocket penetrometer is often used to estimate q_u from samples recovered from a test boring. Basically a spring loaded scale, the pocket penetrometer is used by pushing the 0.25 inch diameter penetrometer rod into a sample. It is calibrated so that an estimate of q_u is indicated on the scale. The pocket penetrometer test does not have an ASTM designation.

SOIL CLASSIFICATION

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX (ASTM D4318)

Liquid and plastic (Atterberg) limits are empirical boundaries which separate the states of fine grained soil. For example, a soil at a very high water content is in a liquid state. As the water content decreases, the soil passes the liquid limit and changes to a plastic state. As the water content decreases further, the soil passes the plastic limit and changes to a semi-solid state.

The liquid limit (LL) is defined as the water content at which a standard groove closes after 25 blows in a liquid limit device. The plastic limit (PL) is the water content at which the soil begins to crumble when rolled into 0.125 inch diameter threads. The thread should break into numerous pieces between 0.125 inch and 0.375 inch long. Plasticity Index (PI) is the difference in the water content between the LL and the PL. This value represents the range of water content over which soil behavior can be characterized as being in a plastic state.

The purpose of the limits is to aid in the classification of fine-grained soils (silts and clays), to evaluate the uniformity of a deposit and to provide some general correlations with engineering properties.

In accordance with the Unified Soil Classification System, a fine-grained soil is classified as to its position on the plasticity chart, TABLE 8. The uniformity of a fine grained soil deposit can be evaluated by plotting the test results of natural water content and Atterberg limits versus depth or elevation.

The liquid and plastic limits are not well correlated with engineering properties that are a function of soil structure or its undisturbed state. However, some general empirical correlations for fine-grained soils have been developed based on index properties, natural water content and Atterberg limits.

CORRELATION OF VARIOUS PROPERTIES:

l Expansion Potential:	According to NAVFAC DM7.1-38 and DM7.3-82.
l Consolidation Stress versus Liquidity Index:	According to NAVFAC DM7.1-142.
l Coefficient of Consolidation versus Liquid Limit:	According to NAVFAC DM7.1-224.

CLASSIFICATION AND FIELD IDENTIFICATION OF SOILS (ASTM D2487 & D2488)

The Unified Soil Classification System (USCS) (ASTM D2487), is based on the identification of soils according to the type and predominance of the constituents considering the following:

- Grain size
- Gradation (shape of grain size distribution curve)
- Plasticity and compressibility

The System divides soils into three major divisions:

- Coarse grained
  (more than 50 percent retained on the No. 200 sieve)
  The smallest size in this category is about the smallest particle size which can be distinguished with the naked eye. Coarse grained soils are classified as to their particle size and shape of the grain size distribution curve.

- Fine grained
  (more than 50 percent passing the No. 200 sieve)
  Fine grained soils are classified as to their position on the plasticity chart shown in TABLE 8.

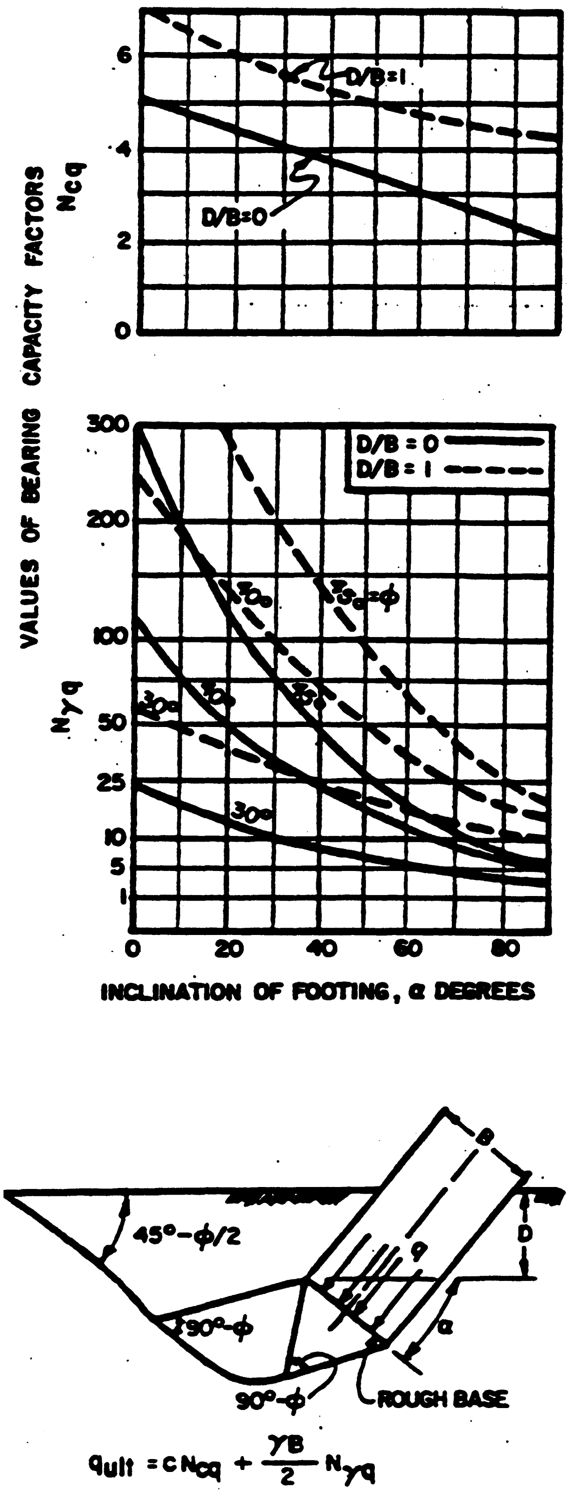
- Highly organic soils
  These soils are peat or other soils which contain substantial amounts of organic matter. No laboratory criteria exist for the highly organic soils; however, they can generally be identified in the field by their distinctive color and odor and by their spongy feel and fibrous texture.

Details of the System are summarized in TABLE 8. Only particles sized 3 inches or less are considered in USCS. Fragments which are larger than 3 inches are classified as cobbles or, if larger than 8 inches, boulders.

Soils can be USCS classified by simple laboratory procedures. However, with practice and experience, it is possible to accurately identify a soil in the USCS by visual means, some of the techniques used for field description and identification are described in ASTM D2488.

APPENDIX B

INCLUDED LOAD ON INCLINED FOOTINGS



INCLUDED LOAD ON INCLINED FOOTINGS

Ultimate Bearing Capacity of Continuous Footings with Inclined Load as taken from Department of the Navy (1982) *Foundations and Earth Structures, Design Manual 7.2,* NAVFAC DM-7.2, Naval Facilities Engineering Command

APPENDIX C

EXTRACT FROM S.P.T.CO. SUPPLEMENT TO AREA MANUAL

PART 20-a

SUPPLEMENTAL SPECIFICATIONS FOR DESIGN OF SHORINGS

20a-1 GENERAL

Part 20-a is a supplement to part 20 of Chapter 8 of the AREA Manual for Railway Engineering. Where this supplement expands or modifies the AREA Manua1, the provisions of the supplement are to be followed.

Shoring which is to be installed adjacent to Railroad operating tracks shall be designed in accordance with the following provisions:

1.1 Special railroad permission is required for installation of shoring closer than 8 '-6" from the centerline of any operating track.

1.2 Shoring between 8'-6" and 10'-0" from face of shoring to centerline of track when excavation is in natural ground or in fill ground which has been placed with proof of adequate compaction control, (also shoring between 8'-6" and 13'-0" when excavation is in fill ground other than compaction controlled fill as stated above), shall be of a type whereby the shoring is installed in place prior to any excavation being performed, and whereby the excavation can be made with no possibility of disturbance or loss of the soil material being retained between the shoring and the track. Common shoring types fulfilling this requirement are interlock edge steel sheet piling, tongue and groove edge precast concrete sheet piling, etc., which are driven into position prior to starting excavation. Shoring types using lagging elements which are placed as excavationproceeds are not permitted within the limits specified in this section.

1.3 Shoring outside the limits stated in Section 1.2 may be of types other than stated in Section 1.2 including types using lagging elements which are installed as the excavation proceeds.

1.4 Shoring, excavations, pits, etc. shall conform to the require- ments of Exhibit 'B' Relations with Railroad Company. Excava- tions, pits, etc. within 13' -0" from centerline of track shall have protection by Standard handrails.

Minimum clearance from centerline of track to face of handrails is 8'-6" on tangent track and 9'-6" on curved track.

1.5 Where the provisions of this specification are more restrictive than the requirements of the Public Utilities Commission Orders, Department of Industrial Safety, OSHA, or other governmental agencies then these supplemental specifications shall apply.

2Oa-2 CLASSIFCATION OF SOILS

2.1 Soils to be retained. as well as the soils depended upon for structural stability (passive resistance, shear strength, friction, etc.) shall be classified in accordance with the soil types listed in AREA Chapter 8, Part 5, Article 5.2.5. This classification is to be a part of the calculations submitted with the shoring plans, and which is to be verified by a Registered Professional Civil Engineer.

20a-3 LOADS ON SHORINGS

3.1 The loading systems of this section apply to shorings which have some degree of flexibility such as cantilever sheet pile walls, or cantilever soldier pile type systems, also sheet pile and soldier pile type systems using tie backs or raker struts in which the tie backs or struts are not preloaded. This Section does not apply to any excavation whereby one side of the excavation is cross-strutted to the opposite side (trench type), nor to tie back or raker strut systems wherein the ties or struts are preloaded.

3.2 Level Earth: 36 Lb per foot EFP (Equivalent Fluid Pressure) is the minimum value to be used in designing shoring. This corresponds to Type 2 soil as defined in AREA Chapter 8, Part 5, Articles 5.2.5 and 5.3.2. EFP values for soils in Types 3, 4 and 5 shall be based on the values tabulated in AREA, Chapter 8, Part 5, Article 5.2.5.

3.3 Where the ϕ and C values of the soils have been ascertained by borings and tests and the values for the EFP have been established by a Registered Professional Civil Engineer specializing in geotechnical engineering, then these values may be used in lieu of the tabulated values providing the ϕ and C values determined by test have been reduced by 15% to allow for the dynamic effect of train loadings on the retained materials.

3.4 Surcharge: Minimum surcharge to be applied is 2 feet height of 1l0 p.c.f. earth.

3.5 Sloping Surcharge: Minimum EFP values shall be those from the curves for Type 2 soils, Chapter 8, Part 5, Appendix C; Section 3.3 is also applicable to sloping surcharge.

3.6 Cooper's E80 Railroad Surcharge Loading. Use values from Chart 3.6 (see next page).

3.7 Alternate Computation: The load system of Sections 3.2 through 3.5 can be calculated on the basis of AREA Chapter 8, Part 5, Appendix B, Trial Wedge Method of Earth Pressure Computation. Values from 3.6 must be added to the trial wedge computation to obtain values for total loads.

3.7.1 The minimum values for retained soils shall be those stated for Type 2 soil, namely, unit weight = 110 pcf, angle of internal friction ϕ = 30°, cohesion = 0. Section 3-3 is also applicable to this method, however, the ϕ and C values determined by borings and tests shall be reduced 15% to allow for the dynamic effect of train loadings. This method will handle soils with both ϕ and C characteristics, as well as structures in excess of 20 feet in height.

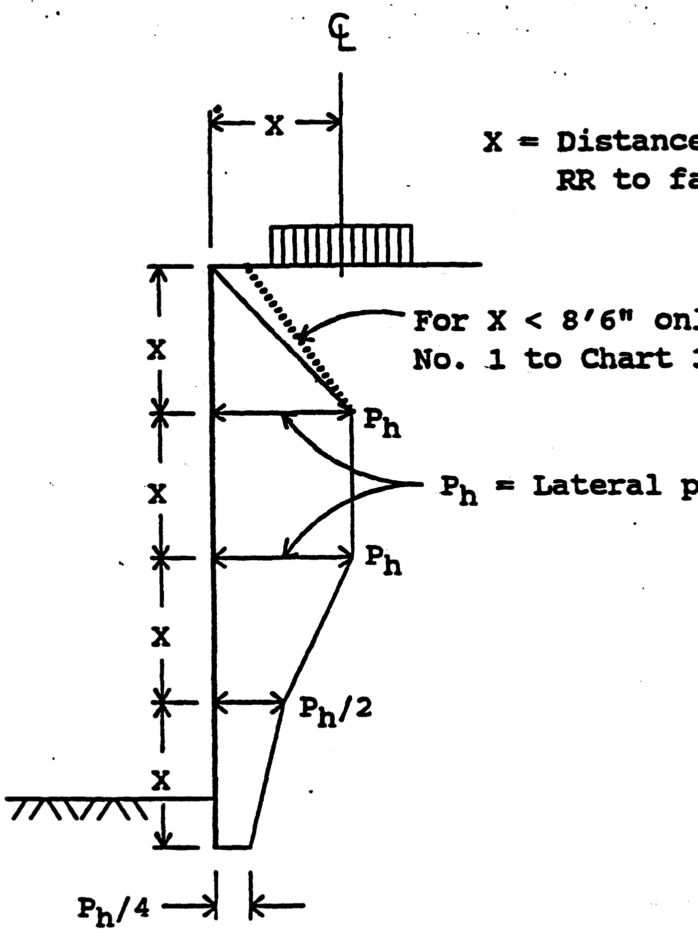
3.8 All retaining structures shall be safe against slip circle type failure.

20a-4 L<u>OADS ON SHORINGS</u>

4.1 The load systems in this section apply to excavations whereby one side of the excavation is strutted against the opposite side (trench type) and tie back and raker strut systems wherein the ties or struts are preloaded.

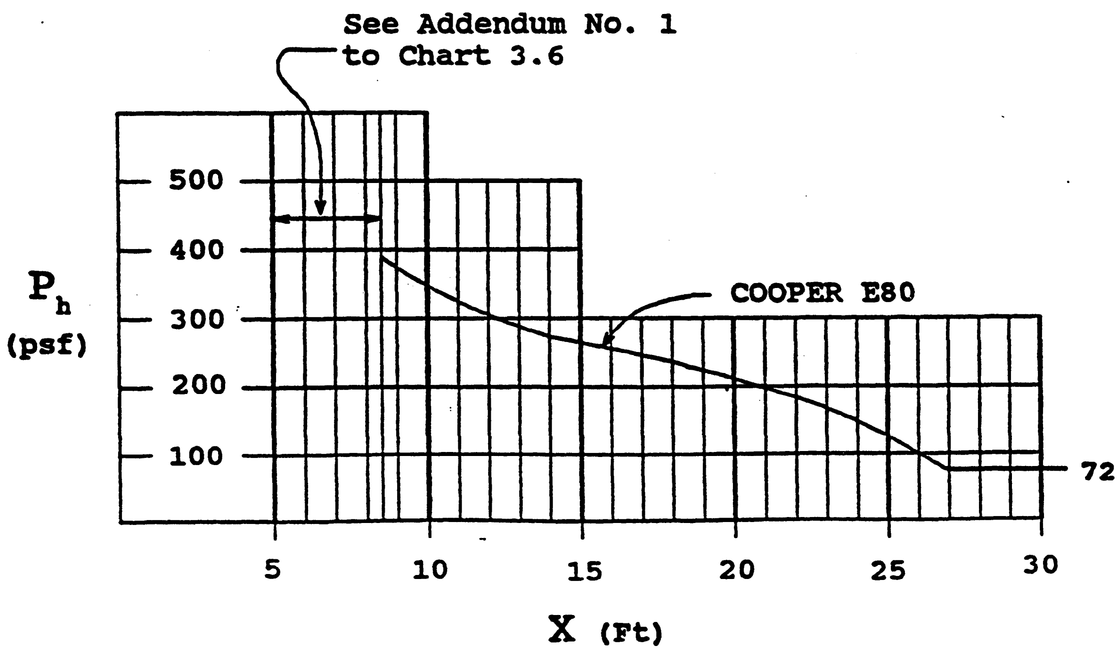
Refer to OSC procedure with RR surcharge load.

CHART 3.6 LATERAL PRESSURE FOR COOPER RAILROAD LIVE LOAD



NOTES:

1. A minimum of 72 psf lateral pressure must be used as specified in 3.4 (equivalent to 2' of earth).
2. For each additional depth increase of X, decrease intensity of the pressure by 50%.



APPENDIX C

Addendum No.1 to Chart 3.6

For lateral pressure values of excavations closer than 8'-6" to center line of track use the following table in lieu of the dashed line of the upper pressure curve of Chart 3.6. The upper pressure curve in Chart 3.6 is to be used for all excavations. All values of X equal the distance from center line of track to the face of excavations.

LATERAL Pressure : P_h (psf)

X -> Ft. / DEPTH Ft.	8.5	7.5	6.5	5.5	4.5
0	100	100	100	100	100
1	374	500	770	1310	1530
1.5	511	692	1105	1310	1420
2	648	899	1105	1310	1310
2.5	785	899	l105	1208	1208
3	785	899	1105	1105	1105
4	785	899	899	899	899
4.5	785	842	842	842	842
5	785	785	785	785	
5.5	729	729	729	729	
6.5	617	617	617		
7.5	504	504			
8.5	390				

20a-5 ALLOWABLE STRESSES AND FACTORS OF SAFETY

5.1 Structural Steel

Axial Tension = $F_y/1.5$ (24,000 psi for A36 Steel)

Steel Sheet Pile Section, AREA Chapter 8
Anchor Rods, AREA Chapter 8, Part 20 Sect. E, Article 7

Flexural Tension = $F_y/1.5$ (24,000 psi for A36 Steel)

Axial Compression
New Steel - 1st usage on subject job:

$$F_a = 20,000 - (0.4)(L/r_y)^2$$

Other than above: $F_a = 16,000 - (0.38)\ (L/r_y)^2$

Flexural Compression: $F_b = (14,400,000)/(Ld/bt) \leq 24,000$ psi

5.2 Prestress Strand or Rod

Allowable working stress (other than tie back):
(0.6) (Ultimate Strength)

Allowable working stress (used as tie back):
(0.4) (Ultimate Strength)

(If use as structural element exceeds 30 days, then the strand shall be protected .-from rust).

5.3 Steel Wire Cable

Allowable working load in Lb: Rated Breaking Strength/2.5

(If used as structural element exceeds 30 days then the cable shall be protected from rust.)

5.4 Concrete

All stress allowables to comply with AREA Chapter 8, Part 2.

5.5 Timber

Compression perpendicular to the grain: 450 psi

Compression parallel to the grain: $480,000/(L/d)^2$ psi

Maximum = 1,600 psi

Flexural stress: 1,700 psi

reduced to 1,500 psi for members with a nominal depth of 8 inches or less.

Horizontal shear 140 psi

5.6 Factors of Safety

For anchor blocks, deadmen, etc.	2.0
In the use of passive pressure for stability	2.0
In the use of soil shear strength and friction based on vertical loads.	1.5
Slip circle failure of structure as a whole, or any part exceptanchor blocks, deadmen, etc.	1.5
Slip circle failure of anchor bolts, deadmen, etc.	2.0
Soil bearing pressures, U.B.C. Section 29	

5.7 No increase in stresses or reduction in safety factors as tabulated is permitted.

20a-6 S<u>HORING PLANS</u>

6.1 Shoring plans shall consist of the following:

A: drawing showing dimensioned locations with respect to track, plan view, elevations, sections and details. Drawing elements shall be fully dimensioned, materials specified and end connections detailed.

Structural calculations shall accompany the plans and shall show the design basis for the shoring and all elements.
Drawings and calculations shall be prepared by or under the immediate supervision of a Registered Professional Civil Engineer Licensed to use the title "Structural Engineer" or by a Registered Civil Engineer with a minimum of 5 years of experience specializing in the design of shorings. Both drawings and calculations shall be signed by the Registered Professional Engineer.

Three (3) sets of plans shall be submitted for review.

Overpass Clearance and Drainage Requirements follow.

NOTES

1. DRAINAGE AND EROSION CONTROL:

(a) To prevent embankment material from sloughing, and drainage water from undermining track subgrade, embankment slopes adjacent to tracks shall be paved with concrete around the curved face to a line opposite the abutment, and self cleaning paved ditches shall be provided to carry waters through the overpass area and disperse them away from the track. These paved slopes and ditches shall be provided at all overpass structures in desert areas, those subject to blowing and drifting sand, and all others where the terrain is such that a build-up of drainage water and flows along the track may be anticipated. Where slopes are not paved, they shall be no steeper than 2 to 1.

(b) Concrete ditches to be Class B, PCC, 4" min. thick. The dimensions of ditches shown are minimum for locations where expected flows along track are not heavy. The size of ditches will vary depending upon the terrain, and should be designed in accordance with good drainage engineering practices.

(c) Drainage plans shall be included with general plans for overpass structures submitted by public agencies to the railroad company for approval. If paved slopes and ditches are not included, the covering letter shall explain why they are considered unnecessary.

2 . PERMANENT CLEARANCES:

(a) Whenever practicable, overpass structures shall be designed to locate all piers and abutments outside of railroad right of way. Permanent clearances as shown are minimum, and greater clearances are preferred when they can be obtained without undue additional expense.

(b) All piers shall be located beyond outer edges of ditches at top of slopes; however, where special, conditions make this impracticable, the side clearances to piers may be reduced provided adequate drainage facilities are furnished outside of piers) and explanation of such special conditions is submitted along with drainage plans for approval by the railroad company.

(c) Because of the required minimum temporary construction clearance; to forms, falsework, etc. as specified in Note 3 below, it is normally not possible to construct piers with a permanent side clearance of 10'-O" or 11'-0" from center line of track to face of pier, as shown in "Section Away from Slope". A greater clearance is usually necessary, the distance depending upon the method of construction, unless the track is out of service or can be shifted away from pier during construction.

3. TEMPORARY CONSTRUCTION CLEARANCES:

(a) The following criteria for the use of falsework supporting deck construction adjacent to operating rail lines of the various categories shall be used as general guidelines, subject to deviation only upon the approval of the railroad's Chief Engineer-System:

Category A: Lines with heavy passenger traffic (San Francisco-San Jose is the only line presently in this category).

No falsework of any kind. No cast in place girders. Precast girders erected on permanent bent caps preferred.

Category B: Heavy traffic freight lines, with or without passenger trains.

Minimum horizontal clearance of 14 feet from center line of nearest track to falsework. Temporary collision posts set in 6 feet of concrete and extending not less than 16 feet above top of rail shall be installed on both sides of the bent and located 10 feet clear of the center line of track approximately 100 feet in advance of falsework. Falsework to be sheathed solid on the side adjacent to track between 3 and 17 feet above top of rail elevation. Collision posts and sheathing shall not be required for lines in this category if horizontal clearances to falsework is 18 feet or greater.

Category C: Lightly used freight lines, drill, yard tracks.

Minimum horizontal clearance of 10 feet from center line of nearest track to falsework. Other criteria same as Category B except that collision posts and sheathing shall not be required if horizontal clearance is 14 feet or greater.

(b) The following minimum temporary construction clearances shall be maintained for placement above top of rail of materials other than falsework (covered in Note 3(a) above), such as piled or stored materials, parked equipment, placement or driving of piles, placement of forms, bracing or. other construction supports:

10'-0" horizontally from center line of nearest tangent track or
11'-0" horizontally from center line of curved track.
22'-0" vertically above top of highest rail.

Any proposed temporary clearances less than these must be submitted to the railroad company for review and approval prior toconstruction, and must be authorized by the utility regulatory body of the state if less than the clearances legally prescribed.

(c) Excavations for pier footings without shoring of track roadbed shall not be made closer than the limit line shown with slope of 1.6 to l starting at subgrade 13'-0" from center line of track. Excavations closer than this sloped limit shall not be undertaken without prior approval by railroad of plans for shoring of track roadbed, and in any instance shall not be approved closer than 8'-6" horizontally from center line of track to near edge of excavation.

(d) Walkways with railings shall be constructed in accordance with plans approved by railroad over open excavation areas adjacent to tracks within normal road bed, and railings shall not be closer than 8'- 6" horizontally from center line of tangent track or 9’ - 6" from curved track.

(e) Plans and calculations covering all falsework, shoring, excavation supports, etc. adjacent to railroad tracks, which have been certified to be complete and satisfactory by the public agency, shall be submitted to the railroad's Chief Engineer-System for approval before construction is begun.

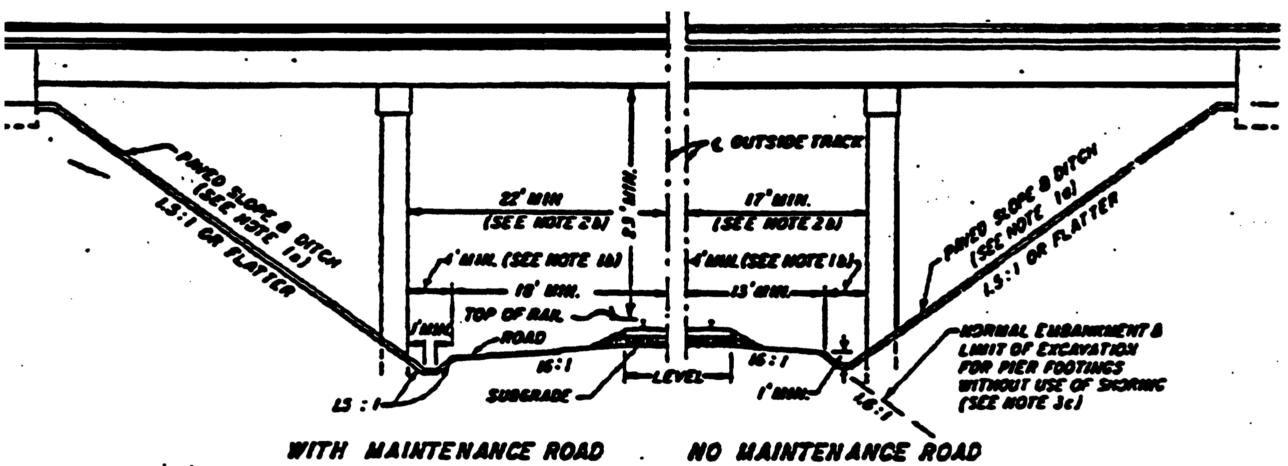
4. DRAWING REFERENCES:

(a) For details of standard track roadbed and ballast see drawing C.S. 5OO for mainline single track, C.S. 520 for main line double track, D.S. 513 for branch lines and sidings, and C.S. 515 for drill, spur and yard tracks. Note references on these drawings to special ballast and roadbed sections required to provide walkways adjacent to switches and tracks where trainmen are required to work on the ground.

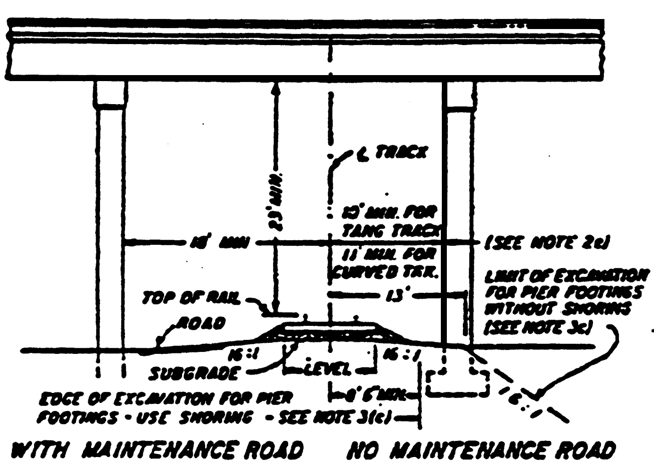
SOUTHERN PACIFIC LINES
COMMON STANDARD

OVERPASS CLEARANCE AND DRAINAGE REQUIREMENTS

NO SCALE ISSUED AUG.30, 1974

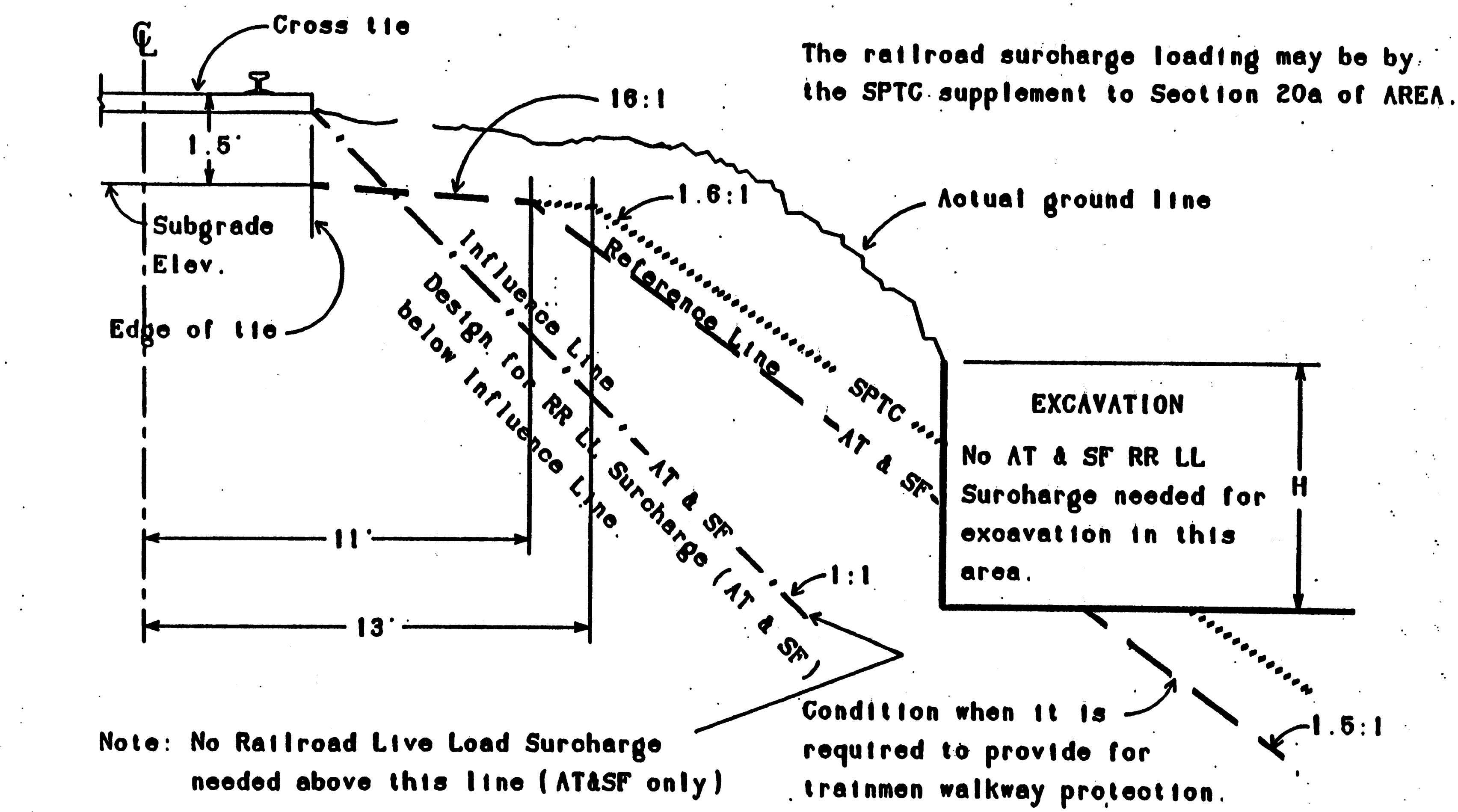


MINIMUM SECTION NEXT TO SLOPES



MINIMUM SECTION AWAY FROM SLOPES

REQUIREMENTS FOR EXCAVATION SHORING - THE ATCHISON TOPEKA, AND SANTA FE RAILWAY COMPANY



Generally no shoring will be required if the bottom of excavation is above a railroad reference line and the ground will stand on it's own. Also, for a trench condition where H > 5', shoring is required per Cal-OSHA. The dotted line shows reference line for SPTC (Rev 8-30-74). The SPTC criteria applies to UPRR, WPRR, NWP, SD & AE. Railroads should be notified prior to start of work. Railroad approval will be required for any work within 15'- 0" of the center line of track. Shoring that intersects the AT & SF Influence Line must be designed to withstand railroad live load lateral forces. The Influence Line is not to be used for unsupported slope excavations.

APPENDIX C

The AREA Manual for Railway Engineering defines soil types in the following manner:

5.2.5 Character of Backfill

Backfill is defined as all material behind the wall, whether undisturbed ground or fill, that contributes to the pressure against the wall. The backfill shall be investigated and classified with reference to the following soil types:

TYPES OF BACKFILL FOR RETAINING WALLS

Type

1. Coarse-grained soil without admixture of fine soil particles, very free-draining (clean sand, gravel or broken stone).
2. Coarse-grained soil of low permeability due to admixture of particles of silt size.
3. Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones.
4. Soft or very soft clay: organic silt: or soft silty clay.
5. Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.

5.3.2 Computation of Backfill Pressure

Values of the unit weight, cohesion, and angle of internal friction of the back fill material shall be determined directly by means of soil tests or, if the expense of such tests is not justifiable, by means of the following table referring to the soil types defined in Section 5.2.5. Unless theminimum cohesive strength of the backfill material can be evaluated reliably the cohesion shall be neglected and only the internal friction considered.

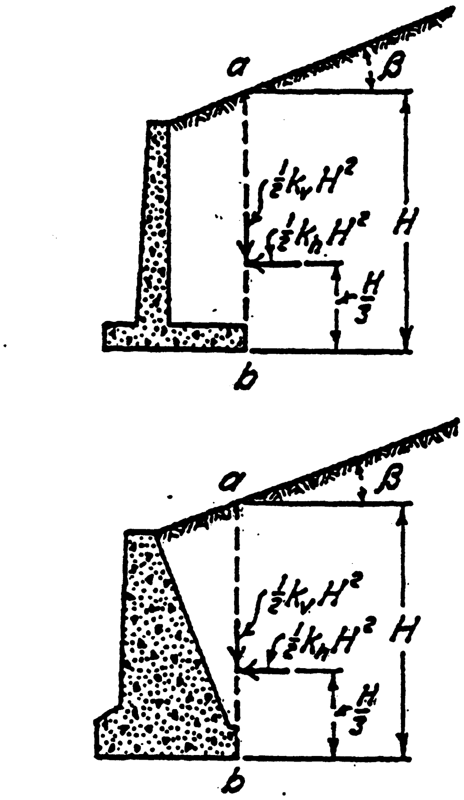
Soil Type	Unit Weight Lb per Ft^3	Cohesion Lb per Ft^2	Angle of Internal Friction
1	105	0	33° 42' *
2	110	0	30°
3	125	0	28°
4	100	0	0
5	120	240	0

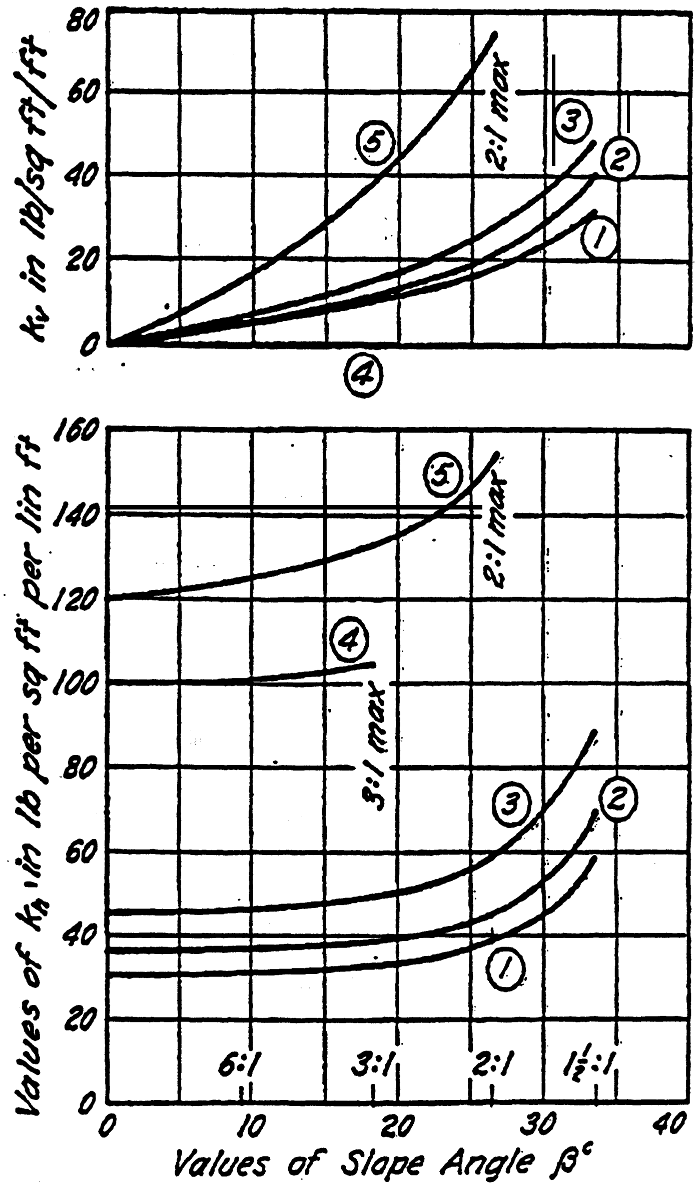
* (38° for broken stone)

American Railway Engineering Association
C & M Section - Engineering Division - AAR

EARTH PRESSURE CHARTS FOR WALLS LESS THAN 20 FT HIGH

Charts A and B may be used for estimating the backfill pressure if the backf ill: material has been classified in accordance with Sec. B, Art. 4.



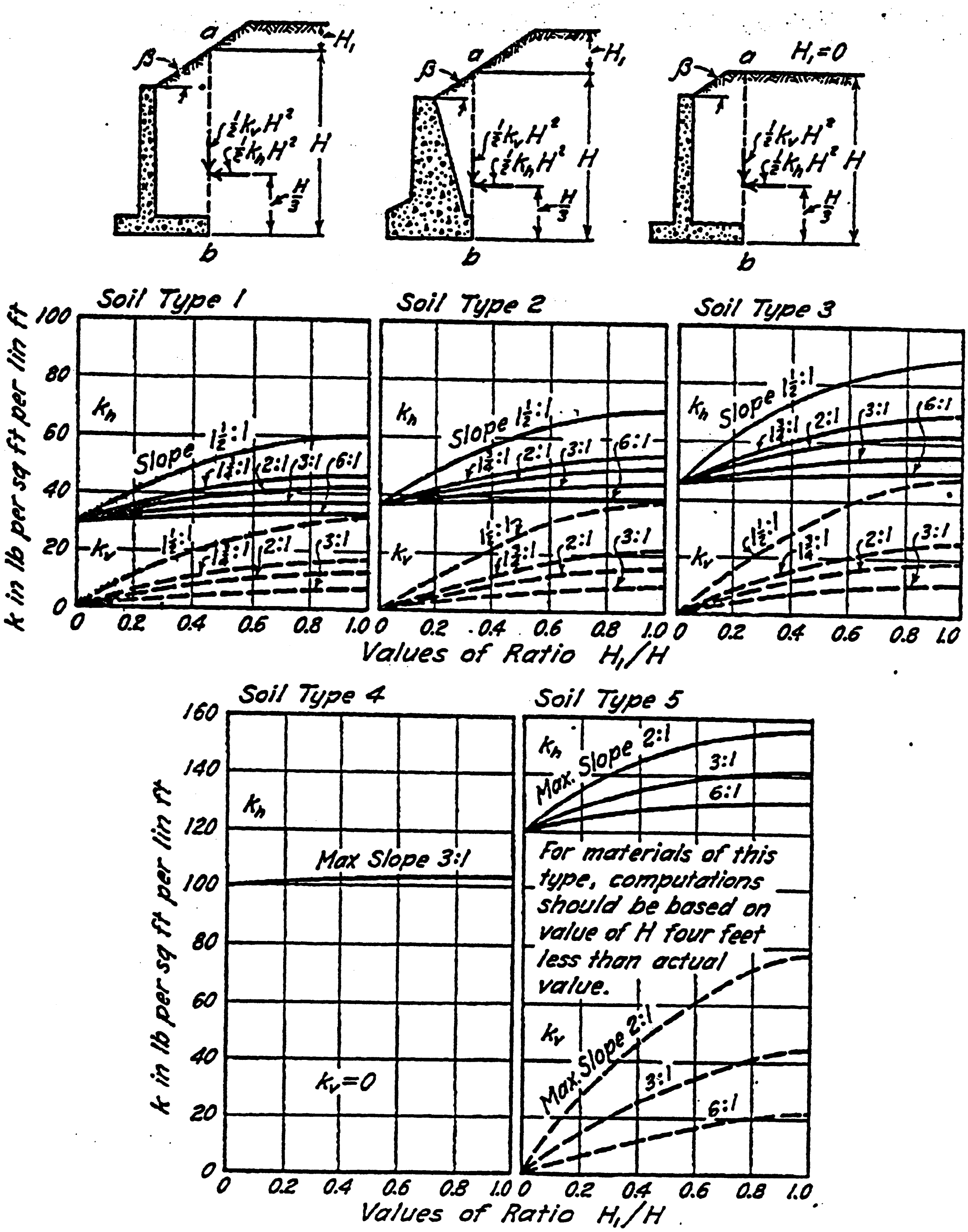


NOTES:

Numerals on curves indicate soil types as described in Sec. B, Art. 4.

For materials of Type 5 computations should be based on value of H four feet less than actual value.

Chart A



a
b
β
H_1
$H_1=0$
H
$\frac{1}{2}k_vH^2$
$\frac{1}{2}k_hH^2$
$\frac{H}{3}$
Soil Type 1
Soil Type 2
Soil Type 3
Soil Type 4
Soil Type 5
k in lb per sq ft per lin ft
k_h
k_v
Slope $1\frac{1}{2}:1$
$1\frac{3}{4}:1$
2:1
3:1
6:1
Max Slope 3:1
$k_v=0$
Max. Slope 2:1
For materials of this type, computations should be based on value of H four feet less than actual value.
Values of Ratio H_1/H

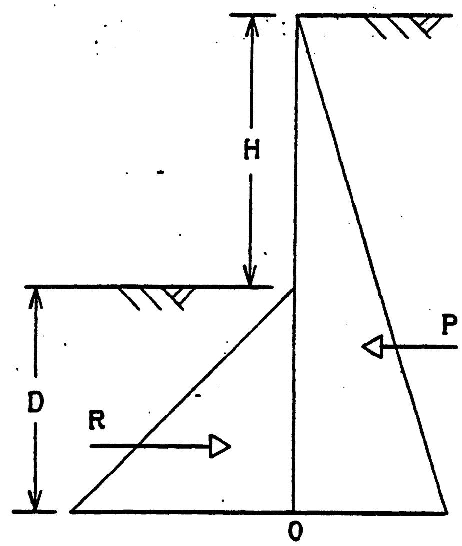
Chart B

APPENDIX D

EASY "MINIMUM" DEPTH SOLUTION FOR CANTILEVER SHEET PILING

Solution for minimum calculated depth of penetration.

This method does not include minimum surcharge of 72 psf.



For level surface:

$$k_a = \tan^2(45° - \phi/2)$$

$$k_p = \tan^2(45° + \phi/2)$$

Determine moments about point o:

$$R\left(\frac{D}{3}\right) = k_p \gamma D\left(\frac{D}{2}\right)\left(\frac{D}{3}\right) = k_p \gamma \frac{D^3}{6}$$

$$P\left(\frac{H+D}{3}\right) = k_a \gamma (H+D)\left(\frac{H+D}{2}\right)\left(\frac{H+D}{3}\right) = k_a \gamma \frac{(H+D)^3}{6}$$

Equate moments about point 0:

$$k_p \gamma \frac{D^3}{6} = k_a \gamma \frac{(H+D)^3}{6}$$

$$\frac{k_p}{k_a} = \frac{(H+D)^3}{D^3}$$

$$\left(\frac{k_p}{k_a}\right)^{1/3} = \frac{H}{D} + 1$$

From which:

$$\boxed{D_{MIN} \approx \frac{H}{\left[\left(\frac{k_p}{k_a}\right)^{1/3} - 1\right]}}$$

APPENDIX E

SUPPLEMENTARY TIEBACK INFORMATION

The following were copied from the Federal Highway Administration Report No. FHWA-RD-75-128, "LATERAL SUPPORT SYSTEMS AND UNDERPINNING"; Vol. I. Design and Construction, pages 199 - 227

The following was copied from Federal Highway Administration Report No. FHWA-RD-75-130, "LATERAL SUPPORT SYSTEMS AND UNDERPINNING"; Vol. III. Construction Methods, pages 212 - 213

TIEBACK TYPES BY PRESSURE

Summary of tieback types and applicable soil types.

Method	Diameter(inches) Shaft Type	Bell Type	Gravity Concrete	Grout Pressure (psi) (1)	Suitable Soils for Anchorage	Load Transfer Mechanism
1. LOW PRESSURE						
Straight Shaft Friction (Solid stem auger)	12-24" (30 - 60cm)	NA	A	NA	Very stiff to hard clays Dense cohesive sands	Friction
Straight Shaft Friction (Hollow stem auger)	6-18" (15 - 45cm) (12-14" most common)	NA	NA	30 - 150 (200 - 1035 kN/m2)	Very stiff to har clays Dense cohesive sands Loose to dense sands	Friction
Underreamed Single Bell at Bottom	12-18" (30 - 45cm)	30-42" (75 - 105cm)	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
Underreamed Multi-Bell	4-8" (10- 20cm)	8-24" (20- 60cm)	A	NA	Very stiff to hard cohesive soils Dense cohesive sands Soft rock	Friction and bearing
2. HIGH PRESSURE-SMALL DIAMETER						
Non-regroutable (2)	3-8" (7.5 - 20cm)	NA	NA	150 (1035 kN/m2)	Hard clays Sands Sand gravel form-ations Glacial till or hardpan	Friction or friction and bearing in permeable soils
Regroutable (3)	3-8" (7.5 - 20cm)	NA	NA	200-500 (1380 - 3450 kN/m2)	Same as for non-regroutable anchors plus: a) stiff to very stiff clay b) varied and difficult soils	Friction and bearing

(1) Grout pressures are typical
(2) Friction from compacted zone having locked in stress. Mass penetration of grout in highly pervious sand/gravel forms "bulb anchor".
(3) Local penetration of grout will form bulbs which act in bear-ing or increase effective anchor diameter.

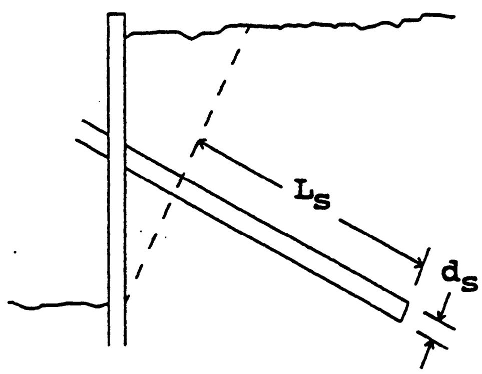
A - applicable
NA - not applicable

APPENDIX E

ANCHOR CAPACITY FORMULAS - LARGE DIAMETER

Large diameter anchors can be either straight shafted, single-belled, or multi-belled. These anchors are most commonly used in stiff to hard cohesive soils that are capable of remaining open when unsupported: however, hollow flight augers can be used to install straight-shafted anchors in less competent soils.

The methods used to estimate the ultimate pullout capacity of large diameter anchors are largely based on the observed performance of anchors and are, therefore, empirical in nature. The following equations can be used to estimate anchor load capacity; fieldtesting of anchors is required to determine true anchor capacity.



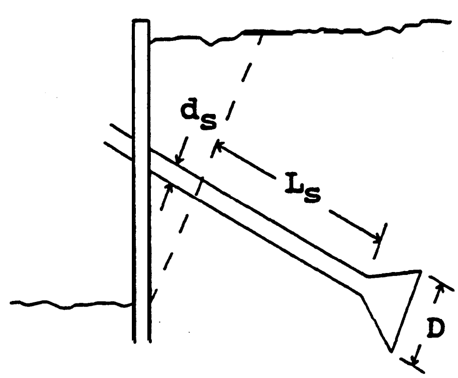
(a)
Friction Anchor

a. Straight-shafted Anchor

$$P_u = \alpha S_u \pi d_s L_s$$

where:

d_s = diameter of anchor shaft
L_s = length of anchor shaft
S_u = undrained shear strength of soil
α = Reduction factor in S_u due to disturbance, etc. α = 0.3 - 0.5 (Hanna, 1973a; Broms, 1968: Littlejohn, 1970a; Neely and Montague-Jones, 1974).



(b)
Belled Anchor

b. Belled Anchor

$$P_u = \alpha S_u \pi d_s L_s + \pi/4(D^2 - d_s^2) N_c S_u$$

(Littlejohn, 1970a)

where:

d_s, L_s, S_u and α are as before
D = diameter of anchor bell
N_c = bearing capacity factor = 9

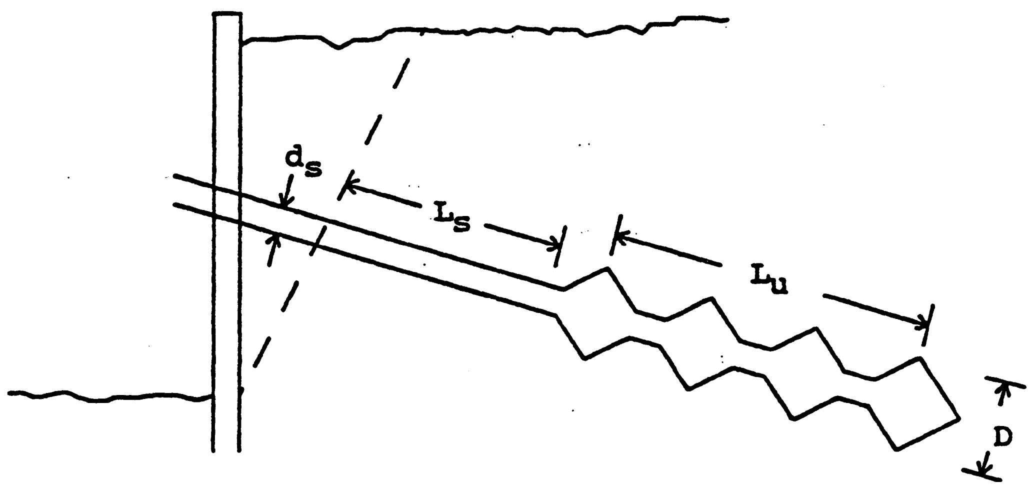
c. Multi-belled Anchor

$$P_u = \alpha S_u \pi d_s L_s + \pi/4(D^2 - d_s^2) N_c S_u + \beta S_u \pi D L_u$$

where:

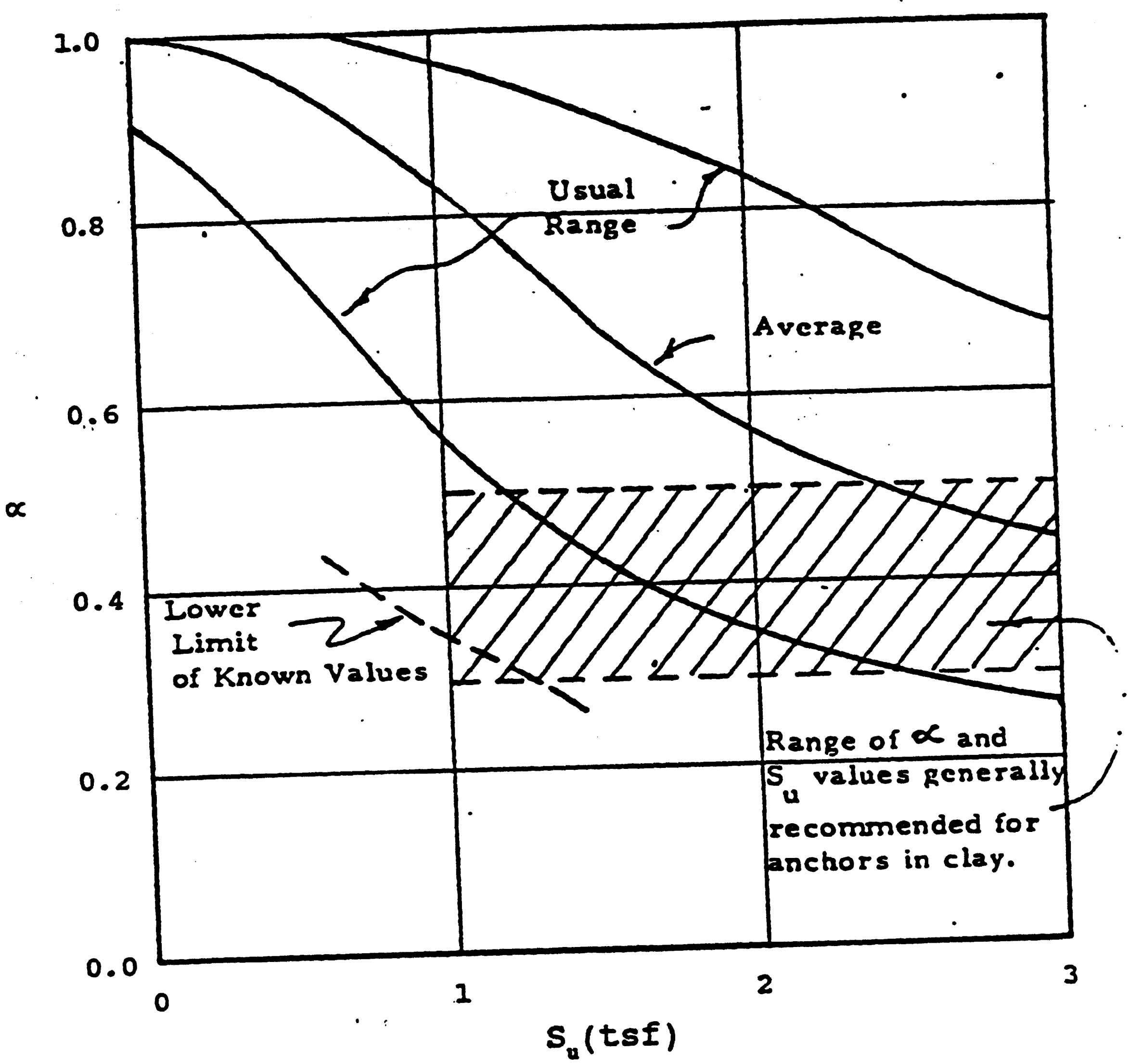
d_s, L_s, S_u, α, N_c, D are as before
L_u = length of underreamed portion of anchor
β = reduction factor in S_u for soil between underream tips
= 0.75 - 1.0 (Littlejohn, 1970a; Bassett, 1970; Neely and Montague-Jones, 1974)

In order for failure to occur between the underream tips, the tips must be spaced at 1.5 - 2.0 times the belled diameter with the bell diameter equal to 2.0 to 3.0 times the shaft diameter.



(c)
Multi-belled Anchor

REDUCTION FACTORS FOR CLAY



From Peck, Hasnson & Thornburn (1974)

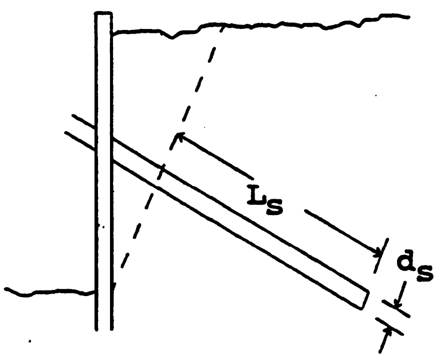
NOTE: 1 tsf = 95.8 kN/m^2
Reduction factor in S_u from observed capacity of friction piles.

ANCHOR CAPACITY FORMULAS - SMALL DIAMETER

Small diameter anchors are generally installed in granular soilswith grouting taking place under high pressure (usually greater than 150 psi (1035 kN/m^2). The anchor capacity will depend upon the soil type, grouting pressure, anchor length, and anchor diameter. The way in which these factors combine to determine anchor loads is not clear; therefore, the load predicting techniques are often quite crude. The theoretical relationships in combination with the empirical data can be used to estimate ultimate anchor load.

a. Theoretical Relationships

1. No grout penetration in anchor zones



(a)
Friction Anchor
(No Grout Penetration)

$p_U = p_i \pi d_s L_s \tan \phi_e$

where:

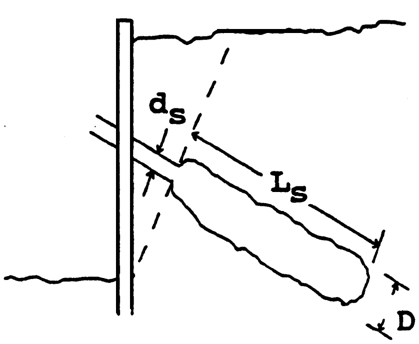
d_s = diameter of anchor shaft
L_S = length of anchor shaft
ϕ_e = effective friction angle between soil and grout
p_i = grout pressure

or $p_U = L_s n_1 \tan \phi_e$ (Littlejohn, 1970a)

where:

n_1 = 8.7 - 11.1 k/ft (127 - 162 kN/m)

2. Grout penetration in anchor zone (very pervious soils)



(b)
Bulb Anchor
(Grout Penetration)

$$P_u = A\bar{\sigma}_v \pi D L_s \tan\phi_e + B\bar{\sigma}_{v@end} \pi/4 (D^2 - d_s^2)$$

(Littlejohn, 1970a)

where:

d_s, D, L_s, and ϕ_e are as before
$\bar{\sigma}_v$ = average vertical effective stress at anchor entire anchor length

$\bar{\sigma}_{v@end}$ = vertical effective stress at anchor end closest to wall

A = (Contact pressure at anchor soil interface)/(effective vertical stress, $\bar{\sigma}_v$)

Littlejohn reports typical values of A ranging between 1 and 2

B = bearing capacity factor similar to N_q but smaller in magnitude. A value of B = N_q/(l.3 - 1.4) is recommended provided $\geq$ 25D; where h is the depth to anchor.

Since the values of D, A and B are difficult to predict, Littlejohn (197Oa) also suggests:

$$P_u = L_s n_2 \tan \phi_e$$

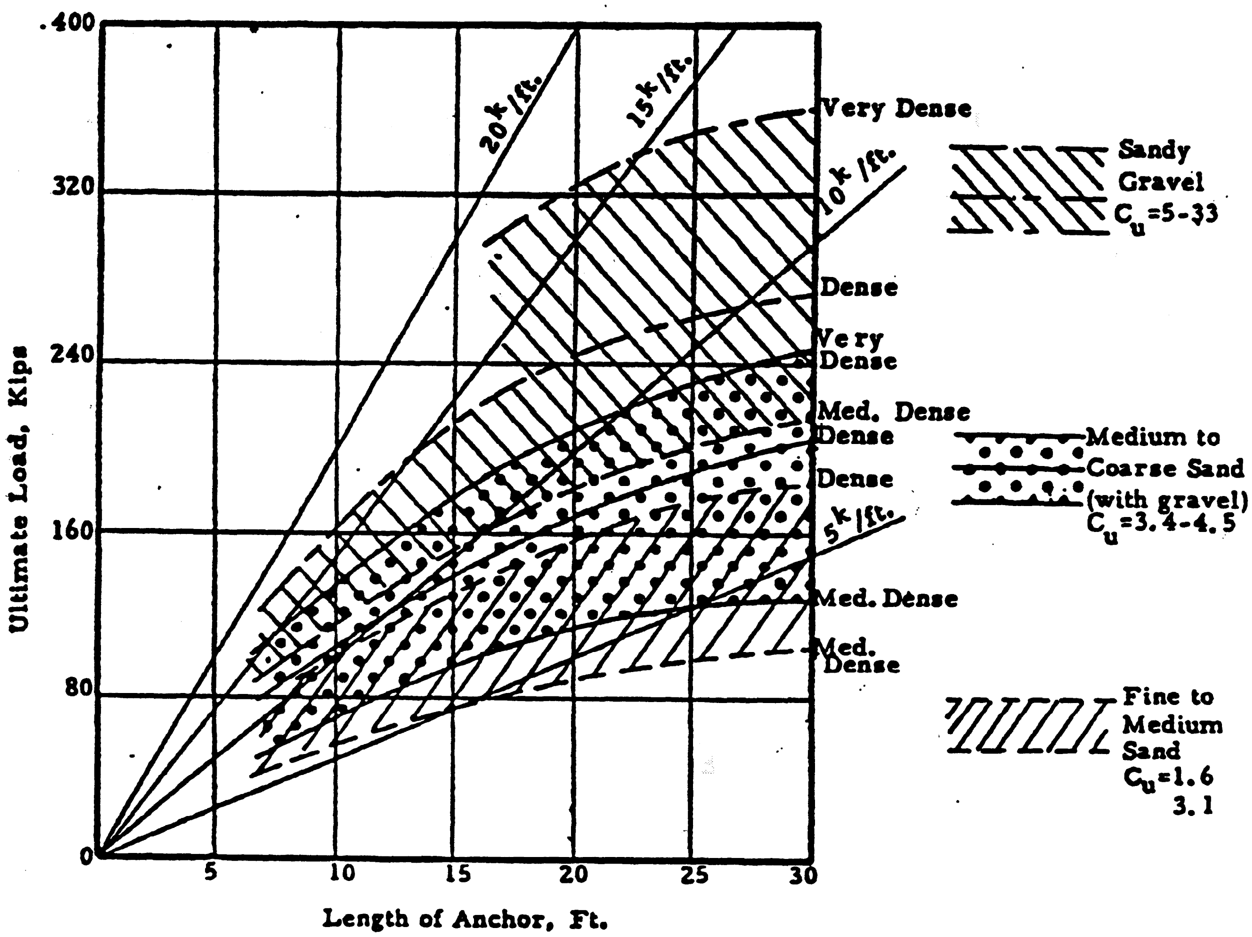
where:

n_2 = 26 - 40 kips/ft	or	(380 - 580 kN/m)
L_s = 3 - 12 ft	or	(0.9 - 3.7 m)
D = 15 - 24 inches	or	(400 - 610 m)
depth to anchor = 40 - 50 ft	or	(12.2 - 15.1 m)

<u>b. Empirical Relationships</u>

The figure on the following page presents an empirical plot of the load capacity of anchors founded in cohesionless soils. This figure was developedby Ostermayer (1974) and represents the range of anchor capacities that may develop in soils of varying densities and gradiations.

ANCHOR CAPACITIES IN COHESIONLESS SOILS



Note: 1 ft = 0.305 m
1 in = 2.54 cm
1 k/ft = 14.6 kN/m

Diameter of Anchor 4" - 6"
Depth of Overburden ≥ 13'

Load capacity of anchors in cohesionless soils showing effects of relative density, gradation, uniformity and anchor length.

APPENDIX E

ANCHOR CAPACITY FORMULA - ROCK

Most rock anchors are straight-shafted friction anchors of 4 inches to 6 inches in diameter. In the past it has been assumed that the load is transmitted uniformly along the grout-rock interface, and most anchor design has been based upon this assumption. Littlejohn (1975) reports the results of studies performed by several authors that indicate that this assumption may not be valid. However, in the absence of more detailed information the establishedmethods should still be used. The designer should be aware of the potential problems of local debonding. Rigid field testing should establish anchor adequacy.

The equation used to estimate anchor capacity is:

$$P_u = \pi d_s L_s \delta_{skin}$$

where:

d_s = diameter of anchor shaft
L_s= length of anchor shaft
δ_{skin} = grout-rock bond strength

The values of skin friction δ_{skin}, for various rock types are summarized in the table on the following page.

In soft rock, it is also possible to form belled or multi-underreamed anchors. The equations governing the ultimate loads in these rocks are given in previous equations. In these cases, the cohesive strength of the rock becomes the controlling quantity.

BOND STRESS VALUES FOR ROCK TYPES

Typical values of bond stress for selected rock types.

Rock Type (Sound, Non-Decayed)	Ultimate Bond Stresses Between Rock and Anchor Plug (δ_{skin})
Granite & Basalt	250 - 800 psi
Limestone (competent)	300 - 400 psi
Dolomitic Limestone	200 - 300 psi
Soft Limestone	150 - 220 psi
Slates and Hard Shales	120 - 200 psi
Soft Shales	30 - l20 psi
Sandstones	120 - 250 psi
Chalk (variable properties)	30 - 150 psi
Marl (stiff, friable, fissured)	25 - 36 psi

1 psi = 6.90 kN/m^2

NOTE: It is not generally recommended that design bond stresses exceed 200 psi even in the most competent rocks.

Data is summary of results presented in:

1. Inland-Ryerson (1974 - ACI Ad Hoc Committee)
2. Littlejohn (1970)
3. Littlejohn (1970)

APPENDIX E

ANCHOR CAPACITY FORMULA - GRAVEL PACKED ANCHORS

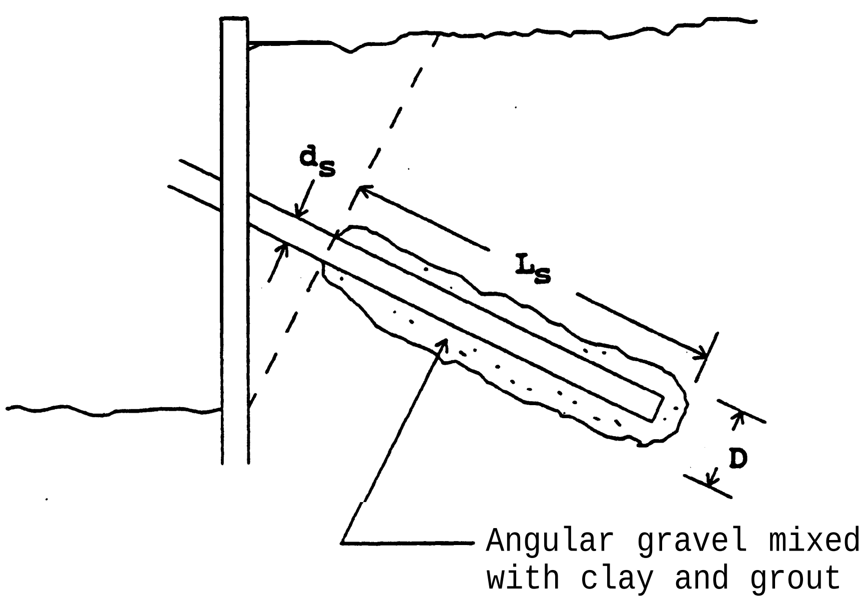
A gravel packed anchor is used on cohesive soils primarily to increase the value of the undrained shear strength coefficent, α. The original anchor hole is filled with angular gravel. A small closed-end casing is then driven into the hole displacing the gravel into the surrounding clay. Grout is then injected as the casing is withdrawn. The grout penetrates the gravel and increases the effective anchor diameter. The irregular surface also improves the strength along the grout-soil interface.

Littlejohn (1970a) proposes that the following equation be used for determining the ultimate load of a gravel packed anchor. There are terms for both frictional resistance and end bearing. A substantial increase in the value of the undrained shear strength coefficient is recommended, and the anchor diameter is larger.

$$P_u = \alpha S_u \pi D L_s + \pi/4 (D^2 - d_s^2) N_c S_u$$

where:

d_s, D, L_s, S_u are as before and $N_c = 9$
a = 0.6 - 0.75 = undrained shear strength coefficent



APPENDIX F

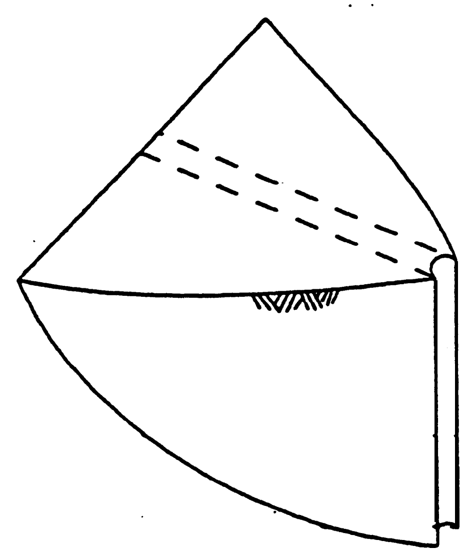
Occasionally it will be necessary to review soldier pile shoring designs based on a combination of cohesionless and cohesive soils. To aid in the review of systems with a combination of soils the California Transportation Materials and Research Laboratory has furnished guidelines for the analysis of soldier piles.

When soldier piles are spaced 4 or more pile diameters apart they can be treated as individual piles. Piles spaced closer than 2 pile diameters should be treated as a pile group (or as a sheet pile wall). For soldier piles spaced between 2 and 4 pile diameters prorate between the analysis for a sheet pile wall condition (which represents 1 soldier pile diameter spacing) and the condition for 4 pile diameter spacing single pile analysis.

Above the plane of the excavation depth the active pressures and surcharge loadings act on a length of wall equal to the soldier pile spacing. Below the excavation depth the active and surcharge pressures act on an expanded effective width (d) of the soldier pile. The expanded effective soldier pile width, incorporates the advantages of increased passive soils resistance for cohesionless or cohesive soils. When adequate soils reports are furnished with shoring plans indicating properties for cohesion in combination with an internal friction angle both properties maybe utilized to increase the effective passive soil resistance.

Passive resistance for individual soldier piles may exceed Rankine theoretical values by a significant amount. For temporary shoring considerations a minimum safety factor is required, so the ultimate load capacity values are to be reducedaccordingly. A minimum safety factor of 1.5 is recommended.

Numerous tests have indicated that soil resistance to horizontal pile loading is greater than that predicted by Rankine equations. For clays the ultimate passive resistance can be as large as 9 to 12 times the shear strength (C), and for cohesionless soils the ultimate resistance can be up to 3 times largerthan Rankine values. The soil resistance acting on isolated piles may be considered to act somewhat as depicted in the sketch at the right.



A combined lateral resisting passive pressure (P_p) equation includes a unitless frictional earth pressure coefficient K_q and a unitless cohesive earth pressure, coefficient k_c, either of which will effectively increase the passive resistance. Any combined lateral resisting passive pressure (P_p) may be defined in equation form as:

$$P_p = d[qK_q + CK_c]$$

Where:
- d = The effective soldier pile diameter
- q = Effective overburden pressure
- K_q = $(\phi/10)K_p \leq 3K_p$
- K_p = Coefficient of passive pressure
- C = Soil cohesion = $q_u/2$
- K_c = 0; for $0 \leq Z_2 \leq 1.5d$ feet
- K_c = $9 + \phi/10 \leq 12$ for $Z_2 > 1.5d$
- Z_2 = Depth below excavation

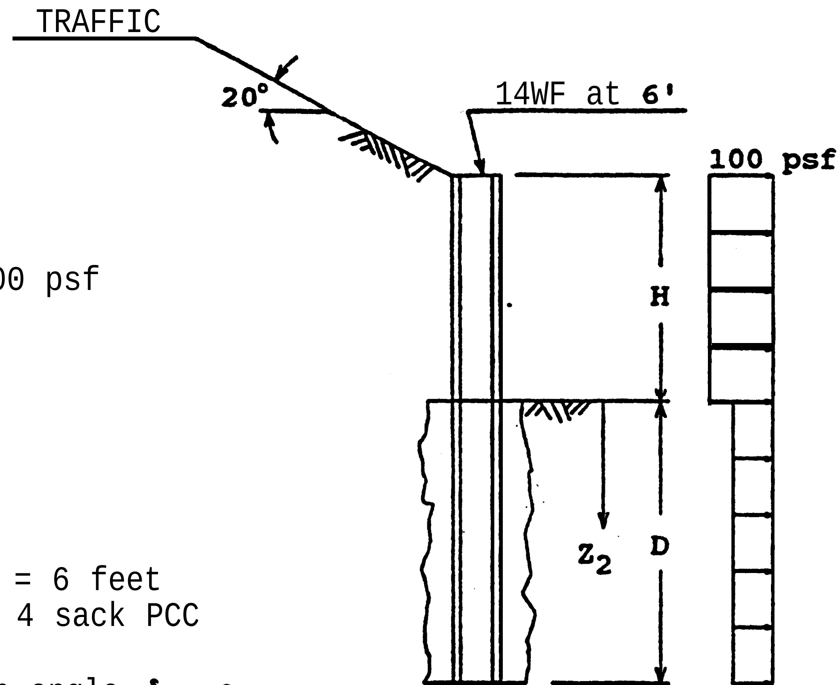
Note That no passive resistance is to be considered for clay above a penetration depth of 1.5d feet.

Arching capability values are included in the Pp equation, no additional arching capability considerations are warranted. The principles defined above can best be demonstrated with the example problem which follows. The alternative surcharge loading of 100 psf (for traffic) is used in the sample problem to reduce its complexity.

APPENDIX F

SAMPLE PROBLEM NO. 22 - COMBINED GRANULAR AND COHESIVE SOIL

GIVEN;



Retained Soil:

$H = 12'$
$\gamma_1 = 120$ pcf
$\phi_1 = 32°$
Surcharge = 100 psf

Resisting Soil:

$\gamma_2 = 115$ pcf
$\phi_2 = 20°$
$C_2 = 300$ psf

S = Pile Spacing = 6 feet
d = 2'-0" ϕ with 4 sack PCC

Use wall friction angle $\delta = 0$.

SOLUTION:

From FIGURE 8:

For $\phi = 32°$ $\beta/\phi = 0.6$ $K_{A1} = 0.40$
For $\phi = 20°$ $\beta/\phi = 1.0$ $K_{A2} = 0.97$
For $\phi = 20°$ $\beta/\phi = 0.0$ $K_{P2} = 3.0(0.678) = 2.03$
(Use level surface condition at excavation depth)

Active soil pressure above depth of excavationi

$P_A = SK_{A1}\gamma_1 H = 6(0.40)(120)(12) = 3,456$ Lb/LF

Surcharge = S(100)(H) = 6(100)(H) = 600(H) Lbs

Active soil pressure below depth of excavation:

$P_A = d(K_{A2}\gamma_1 H + K_{A2}\gamma_2 Z_2 - 2C[K_{A2}]^{1/2})$

$P_A = 2\{0.97(120)(12) + 0.97(115)Z_2 - 2(300)[0.97]^{1/2}\}$

$P_A = 2(1,397 + 112Z_2 - 591) = 1,612 + 224Z_2$ Lb/LF

Surcharge = 100(d)(D) = 100(2)(D) = 200(D) Lb

Passive Soil Pressures: (Combined granular and cohesive soils)

$P_p = d(qK_q + CK_c)$ (For 4 or more pile diameter spacing)

Where:

Effective overburden pressure $q = \gamma_2 Z_2 = 115Z_2$ psf
Ultimate $K_q = (20°/10)(2.03) = 4.06 < 3K_p = 9.75$
Working value of $K_q = K_q$/(Safety Factor) = 4.06/1.5 = 2.71
C = 300 psf
$K_c = 0$ for $0 < Z_2 \leq 1.5d = 3$ feet.
Ultimate $K_c = 9 + \phi/10 \leq 12 = 9 + 2 = 11 < 12$ for $Z_2 > 1.5d$.
Working value of $K_c = K_c$/S.F. $= K_c/1.5 = 11/1.5$

$P_p = d[\gamma_2 Z_2 K_q + CK_c]$

$P_p = 2'[115Z_2(2.71)] = 623.3z_2$ plf for $0 < Z_2 \leq 3'$

$P_p = 2'[115z_2(2.71) + 300(11/1.5)] = 623.3Z_2 + 4400$ plf for $Z_2 \geq 3'$

(No clay can be utilized for passive resistance above Z_2=1.5d=3')

Since the pile spacing for this example problem is 6 feet and the pile diameter is 2 feet the 4 or more pile diameter pile spacing criteria is not met. It will be necessary to prorate between a 4 pile diameter spacing and a condition similar to that of a sheet pile wall.

When the pile spacing effectively equals zero the passive pressure is derived from the following general equation:

$$P_p = \gamma Z_2 K_p + 2C(K_p)^{1/2}$$

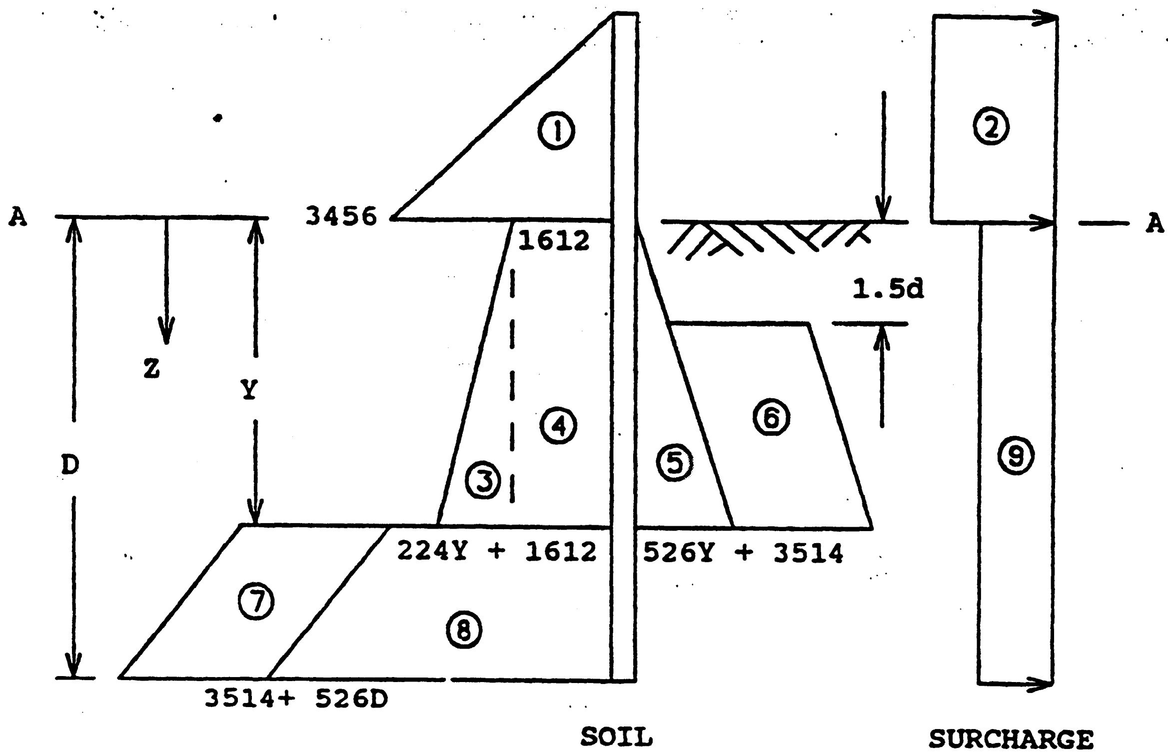
For $0 < Z_2 < 3'$: $P_p = \gamma Z_2 K_p = 115Z_2(2.03)$
$= 233.5Z_2$ Lb/LF

For $Z_2 > 3'$: $P_p = 233.5Z_2 + 2(300)(2.03)^{1/2}$
$= 233.5Z_2 + 854.9$ Lb/LF

Proration value: S/4d where S = 6' and d = 2' ∴ Ratio = 6/8 = 3/4

For $0 < Z_2 < 3'$: $P_p = 233.5z_2 + (3/4)(623.3Z_2 - 233.5Z_2)$
$\approx 526Z_2$ Lb/LF

For $Z_2 > 3'$: $P_p = 526Z_2 + 854.9 + (3/4)(4400 - 854.9)$
$\approx 526Z_2 + 3514$ Lb/LF



DESIGN PRESSURE (Not to scale)

Areas:

1) (l/2) (3456) (12)	= 20,736
2) 600(12)	= 7,200
3) $(1/2)(224)Y^2$	= $l12Y^2$
4) 1,612Y	= 1,612Y
5) $(1/2)(526)Y^2$	= $263Y^2$
6) 3,514(Y - 3)	= 3,514Y - 10,542
7) 3,514(D - Y)	= 3,514D - 3,514Y
8) $(1/2)(526)D^2 - 1/2(526)Y^2$	= $263D^2 - 263Y^2$
9) 200D	= 200D

The sum of the horizontal force must equal zero, $F_H = 0$.

$$F_H = 20,736 + 7,200 + 112Y^2 + 1,612Y - 263Y^2 - 3,514Y + 10,542 + 3,514D - 3,514Y + 263D^2 - 263Y^2 + 200D = 0$$

$$= 263D^2 + 3,714D - 5,416Y - 414Y^2 + 38,478 = 0$$

$$= D^2 + 14.12D - 20.59Y - 1.57Y^2 + 146.30 = 0$$

The sum of the moments about A-A must equal zero, $M_{(A-A)} = 0$.

$$\begin{aligned} M_{(A-A)} = {} & 20,736[12/3] + 7,200[12/2] - 112Y^2[2Y/3] - 1,612Y[Y/2] \\ & + 263Y^2[2Y/3] + 3514(Y-3)[3 + (1/2)(Y-3)] \\ & - 3514(D - Y)[Y + (1/2)(D - Y)] - 263D^2[2D/3] \\ & + 263Y^2[2Y/3] - 200D[D/2] = 0 \end{aligned}$$

$$\begin{aligned} 0 = {} & 82,944 + 43,200 - 74.67Y^3 - 806Y^2 + 175.33Y^3 + 1,757Y^2 \\ & - 15,813 - 1,757D^2 + 1,757Y^2 - 175.33D^3 + 175.33Y^3 - 100D^2 \end{aligned}$$

$$0 = -175.33D^3 - 1,857D^2 + 2,708Y^2 + 275.99Y^3 + 110,331 = 0$$

$$D^3 + 10.59D^2 - 15.45Y^2 - 1.57Y^3 - 629.28 = 0$$

By trial and error D = 24.26 feet, and Y = 20.45 feet.

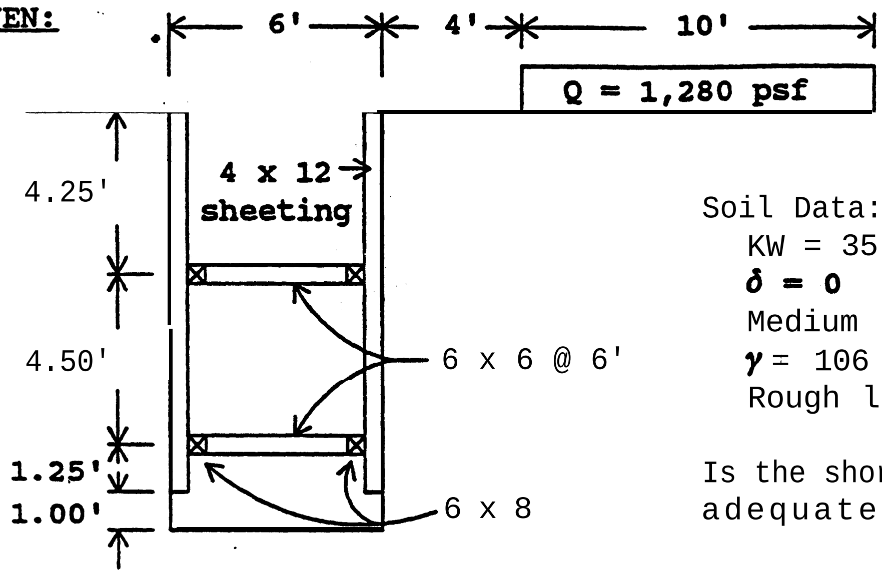
Safety factor of 20% to 40% for additional D is not required.

In view of the softness of the clay as indicated in TABLE 13 this answer is not unreasonable.

APPENDIX G

SAMPLE PROBLEM NO. 23 - STRUTTED TRENCH (Medium Compact Sand)

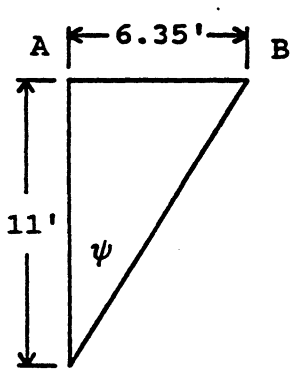
GIVEN:



Soil Data:
KW = 35 pcf
$\delta = 0$
Medium compact sand:
γ = 106 pcf
Rough lumber

Is the shoring system shown adequate?

SOLUTION:



$K_a = Kw/\gamma = 35/106$
$= 0.33$
$K_a = \tan^2(45° - \phi/2)$
$\therefore \phi = 30°$

$\psi = 45° - \phi/2 = 30°$

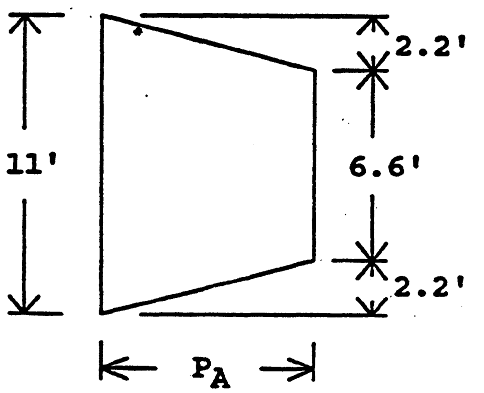
$AB \doteq (11)(\tan 30°)$
$= 6.35'$

Note: Pressure diagram will be a trapezoid.

Use the tabular values listed in the section on surcharges or the Boussinesq Strip formula to find the lateral surcharge pressures. The tabular method will be used here.
Multiply the values by 1280/300 = 4.27.

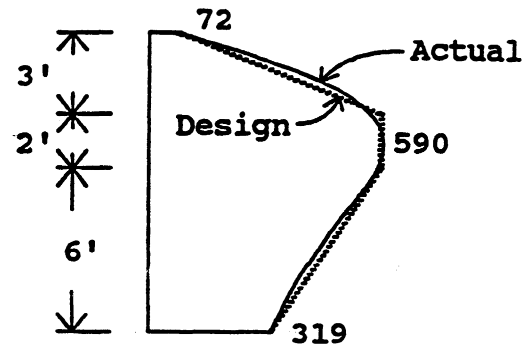
Depth	Q value (0 - 14')	- Q value (0 - 4')	x 4.27	= σ
1	272.81	208.27		275.6
2	246.16	135.06		474.4
4	196.31	54.51		605.5
6	153.52	24.15'		552.4
8	118.58	12.16		454.4
10	91.21	6.81		360.4
11	80.03	5.27		319.2

Soil: Restrained System, H > 10

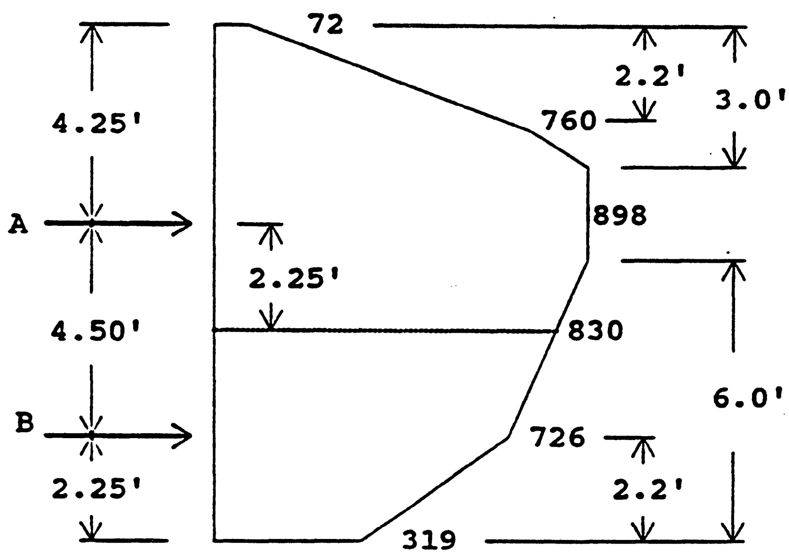


$P_A = 0.8\gamma H K_a$
$= (0.8)(106)(11)(0.33)$
$= 308$ psf

Surcharge:



Use values shown for design



Combined Diagram

Note: For clay soils the approach would be the same: the soil pressure diagram would be modified by using the Stability Number Method.

Total force = (2.2)(72 + 760)/2 + (0.8)(760 + 898)/2 + (2.0)(898)
+ (3.8)(898 + 726)/2 + (2.2)(726 + 319)/2
= 915 + 663 + 1,796 + 3,086 + 1,150 = 7,610 Lb/LF

A = 915 + 663 + 1,796 + (1.5)(898 + 830)/2 = 4,670 Lb/LF
B = (2.3)(830 + 726)/2 + 1,150 = 2,939 Lb/LF

CHECK SHEETING (upper cantilever)

Find M_{max}:

AREA	ARM	MOMENT
(72)(2.2) = 158.4	2.2/2 + 2.05 = 3.15	499
(760 - 72)(2.2)/2 = 756.8	2.2/3 + 2.05 = 2.78	2,104
(760)(0.8) = 608.0	0.8/2 + 1.25 = 1.65	1,003
(898 - 760)(0.8)/2 = 55.2	0.8/3 + 1.25 = 1.52	84
(898)(1.25) = l,122.5	1.25/2 = 0.63	707
2,700.9		4,397

S Req'd = M/f = (4,397)(12)/(1,500)(1.33) = 26.4 in^3
S Furnished = $bh^2/6$ = $(12)(4)^2/6$ = 32.0 in^3

Find v_{max}: l.5V/A = (1.5)(2,701)/(4)(12) = 84 psi < 140

CHECK SHEETING (middle section)

Assume w equals a constant 898 Lb/LF
M = $wL^2/10$ = $(898)(4.5)^2/10$ = 1,818 < 4,397

V= (4.5/2 - 0.5/2 - 0.33)(898) = 1,500 Lb
v = 1.5V/A = (1.5)(1,500)/48 = 46.9 psi < l40

SHEETING O.K.

CHECK WALES (upper wale controls)

M = $wL^2/10$ = $(4,670)(6)^2/10$ = 16,812 Ft-Lb
S Req'd = M/f = (16,812) (12)/(1,500) (1.33) = 101.1 in^3
S Furnished = $bh^2/6$ = $(6)(8)^2/6$ = 64.0 in^3

Reduce strut spacing.
Maximum L = $[(64)(1,500)(1.33)(10)/(4,670)(12)]^{1/2}$ = 4.77'
Use L = 4.75' (4'- 9")

V = (4.75/2 - 0.5/2 - 0.67)(4,670) = 6,795 Lb
v= 1.5V/A = (l.5)(6,795)/(6)(8) = 212.3 psi > 140 ∴ **n.g.**

Try 8 x 8 wale

V = (212.3)(6)/8 = 159.2 > 140 ∴ **n.g.**

Try 8 x 8 wale with strut spacing of 4'-3"

V = (4.25/2 - 0.5/2 - 0.67)(4,670) = 5,627 Lb
v = (1.5)(5,627)/64 = 131.9 psi < 140

REVISE STRUT SPACING TO 4'- 3"

CHECK STRUT

P/A = (4,670)(4.25)/(6)(6) = 551.3 psi
allowable f_c = 1300 - 20L/d where L/d ≤ 50
= 1300 - (20)(4)(12)/6 = 1,140 psi
allowable f_c = $480{,}000/[L/d]^2$ = $480{,}000/[48/6]^2$
= 7,500 psi > 1,300 max, use 1,300 psi
1,140 controls
(1,140)(1.33) = 1,516.2 > 551.3

CHECK COMPRESSION ON WALE

allowable f_p = (350)(1.33) = 465.5 < 551.3 ∴ n.g.

Try 8 x 8 strut, P/A = (4,670)(4.25)/(8)(8) = 310 < 465.5

SUMMARY

Sheeting is satisfactory for the wale spacing shown.
Wales need to be 8 x 8 Rough with 4'- 3" strut spacing.
Struts need to be spaced at 4'- 3" sized 8 x 8 Rough (or 6 x 6 Rough with steel plates at ends).

Notes :

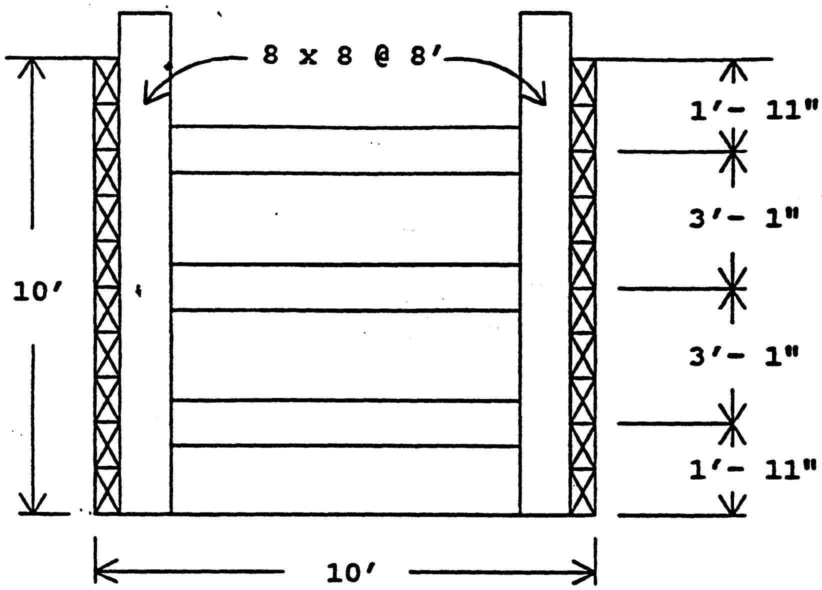
If the surcharge shown were to represent a building or other critical load, the load duration factor of 1.33 would not have been used and in this case the section modulus of the sheeting would not have been adequate.

The top cantilever of 4'- 3" will permit settlement concurrent with the sheeting deflection which would be detrimentalto any structure adjacent to the excavation.

A better design would provide for three sets of wales and struts with the wales appropriately spaced to carry similar loadings.

APPENDIX G

SAMPLE PROBLEM NO. 24 - STRUTTED TRENCH (Bay Mud)



GIVEN:

Stiff wet bay mud (60% clay).
γ = 120 pcf

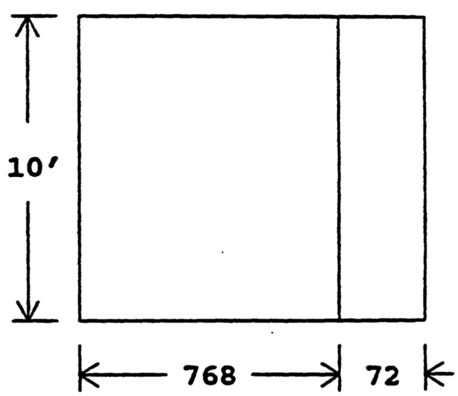
Rough Lumber
Lagging: 4 x 12
Struts: 8 x 8 @ 8'

SOLUTION:

For soft wet conditions Kw = 120 pcf.

For restrained system and H $\leq$ 10',
may use rectangular pressure diagram.

P_A = 0.64KwH = (0.64)(120)(10)
= 768 psf.



There was no surcharge given. Use the minimum (72 psf).

Total = 768 + 72 = 840 psf

CHECK LAGGING

Consider arching effect on lagging: Multiply all loads by 0.6.

M = [wL^2/10]0.6 = [(840)$(8)^2$/10]0.6 = 3,226 Ft-Lb
S Req'd = M/f = (3,226)(12)/(1,500)(1.33) = 19.4 in^3
S furnished = bh^2/6 = (12)$(4)^2$/6 = 32.0 in^3 > 19.4

Y= [(8/2 - 0.33 - 0.33)(840)](0.6) = 1,683 Lb
v = 1.5V/A = (1.5)(1,683)/(4)(12) = 52.6 psi < 140

CHECK 8 x 8 UPRIGHTS.

Middle Section
$M = (8)(840)(3.08)^2/10 = 6{,}375$ Ft-Lb
$V = (3.08/2 - 0.33 - 0.67)(840)(8) = 3{,}629$ Lb

Cantilever section
$M = (8)(840)(1.92)^2/2 = 12{,}386$ Ft-Lb
$V = (1.92 - 0.33 - 0.67)(840)(8) = 6{,}182$ Lb

S Req'd = $(12{,}386)(12)/(1{,}500)(1.33) = 74.5\ in^3$
S Furnished = $(8)(8)^2/6 = 85.3\ in^3 > 74.5$

$v = (1.5)(6{,}182)/64 = 144.9 > 140 \therefore$ n.g.

Increase strut spacing from 3'- 1" to 3' 3"

$V = (1.75 - 0.33 - 0.67)(840)(8) = 5{,}040$ Lb
$v = (1.5)(5{,}040)/64 = 118.1 < 140$

CHECK STRUTS

$P/A = (840)(8)(1.75 + 3.25/2)/(8)(8) = 354.4$ psi

Allowable $f_c = 480{,}000/(L/d)^2 = 480{,}000/(96/8)^2$
= 3,333 psi > 1,600 max, use 1,600 psi
1,600 controls > 354.4 psi o.k.

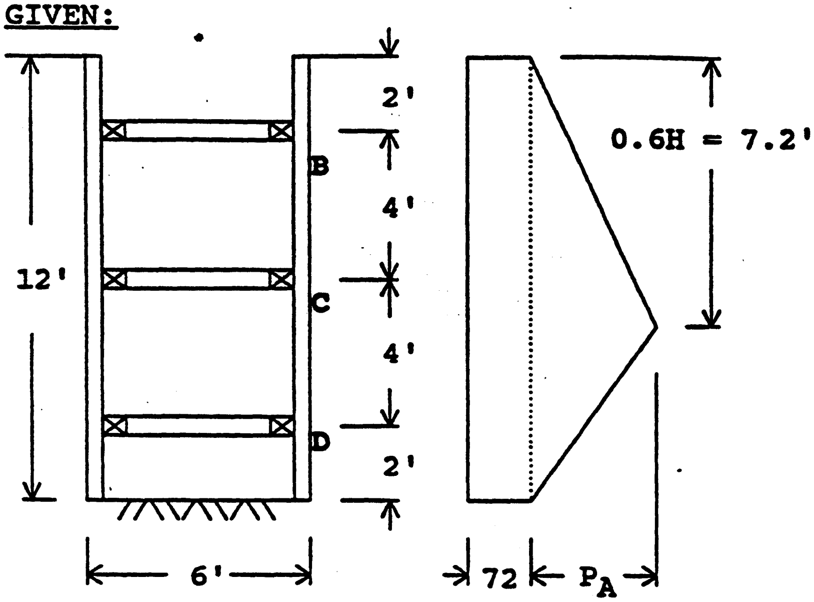
CHECK COMPRESSION ON UPRIGHT

Allowable $f_p = (350)(1.33) = 465.5 > 354.4$

System will be satisfactory if top and bottom cantilever dimensions are changed from 1' -11" to 1' - 9", and the vertical spacing between strut is accordingly revised from 3'- 1" to 3'- 3".

SAMPLE PROBLEM NO. 25 - STRUTTED TRENCH (Medium Clay)

GIVEN:



Method by
Tschebotarioff

From Soils Report

γ= 115 pcf
q_u = 2,000 psf

Materials

Sheeting: 2 x 12
Wales: 6 x 8
Struts: 6 x 8 @ 6'
All rough lumber

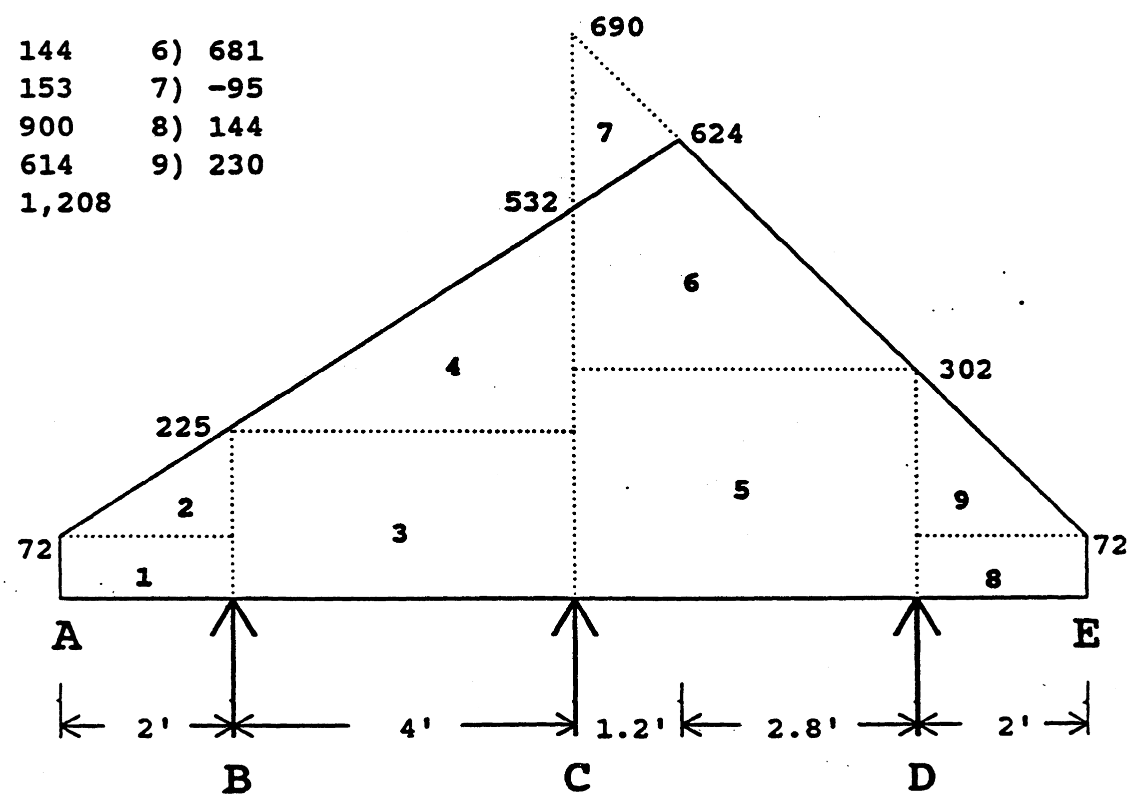
$P_A = 0.4\gamma H$
$= (0.4)(115)(12)$
$= 552$ psf

No surcharge given
Use minimum 72 psf

SOLUTION:

AREAS (Lb/LF)

1) 144
2) 153
3) 900
4) 614
5) 1,208
6) 681
7) -95
8) 144
9) 230



Find Maximum Moment (use moment distribution):

Fixed end moments (Ft-Lb/LF)

1) (72) (2) [2/2] = 144
2) {(153)(2)/2}[2/3] = 102
BA = 246

3) $(225)(4)^2/12$ = 300 = 300
4) $(327)(4)^2/30$ = <u>174</u>
$(327)(4)^2/20$ = 262
BC = 474 CB = 562

5) $(302)(4)^2/12$ = 403 = 403
6) $(388)(4)^2/20$ = 310
$(388)(4)^2/30$ = 207
7) $[(-158)(4)^2(0.3)^2/60][10 - (10)(0.3) + (3)(0.3)^2]$ = <u>-28</u>
$[(-158)(4)^2(0.3)^2/60][5 - (3)(0.3)]$ = <u>-16</u>
CD = 685
DC = 594

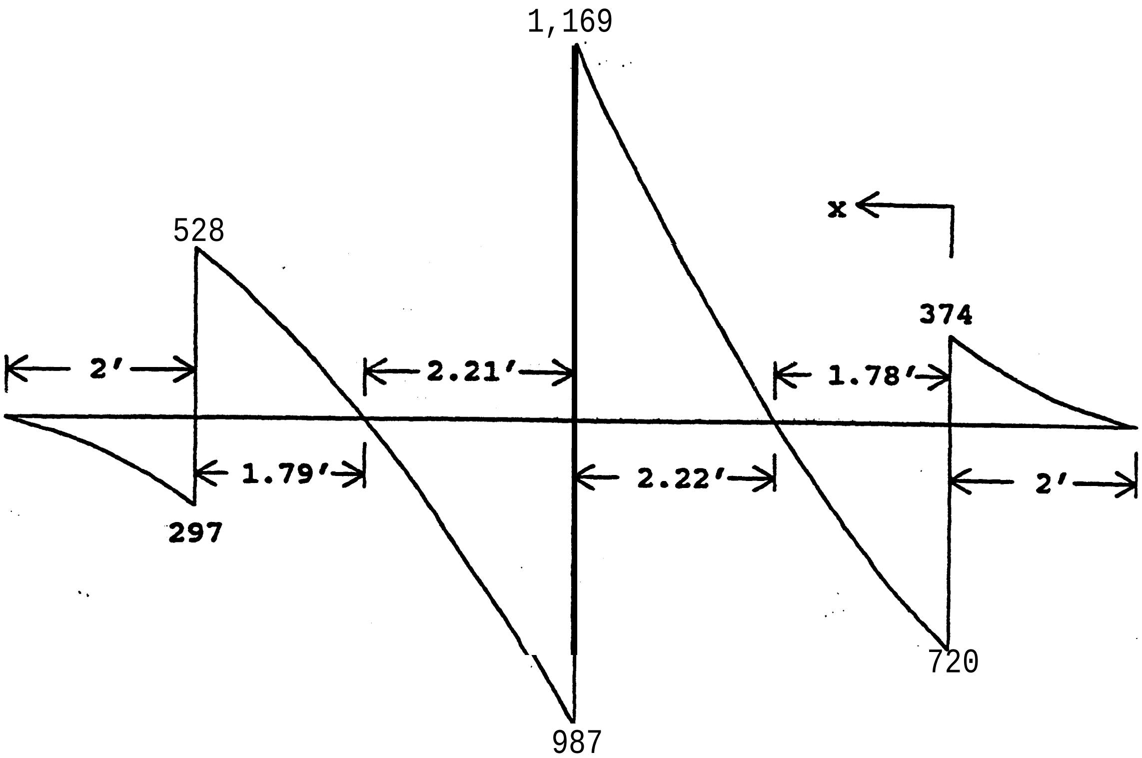
8) (72) (2) [2/2] = 144
9) {(230) (2)} [2/3] = 153
DE = 297

	B		C		D	
	0	1.0	0.5	0.5	1.0	0
	-246	+474	-562	+685	-594	+297
		-228	-61.5	-61.5	+297	
		-30.7	-114	+148.5	-30.7	
		+30.7	-17.3	-17.3	+30.7	
	246	+246	-755	+755	-297	+297
MV		-127.2	+127.2	+114.5	-114.5	
	+144	+450	+450	+604	+604	+144
	+153	+204.7	+409.3	+517.3	+258.7	+230
				-66.4	-28.4	
	825		2,156		1,094	

CHECK SHEETING:

By inspection, the maximum moment will either be at C or between c and D.

M_C = -755 Ft-Lb/LF



Find point of zero shear between C and D.

$720 - 302X - (624 - 302)(x/2.8)(x)/2 = 0$
$57.5x^2 + 302X_1 - 720 = 0$
X- 1.78'

$M = -(302)\ (1.78)[1.78/2] - \{(322)(1.78/2.8)(1.78)/2\}[1.78/3]$
$\quad + 1{,}094[1.78] - 144[1.78 + 2/2] - 230[1.78 + 2/3]$
$\quad = 398$ Ft-Lb/LF

M_C controls (M = -755 Ft-Lb)

S req'd = M/f = (755)(12)/(1,500)(1.33) = 4.54 in^3

S furnished = $(12)(2)^2/6 = 8.0$ in^3

v= 1.5V/A = (1.5)(1,169)/(2)(12) = 73.1 < 140 psi

CHECK WALES (center controls):

M = $(2{,}156)(6)^2/10 = 7{,}762$ Ft-Lb

S req'd = (7,762)(12)/(1,500)(1.33) = 46.7 in^3

S furnished = $(6)(8)^2/6$ = 64 in^3
V = (6/2 - 0.33 - 0.33)(2,156) = 5,045 Lb

v = (1.5)(5,045)/(6)(8) = 157.7 psi > 140

Use 8 x 8's v = (157.7)(48/64) = 118 < 140

CHECK STRUTS (center controls):

P/A = (2,156)(6)/(6)(8) = 270 psi

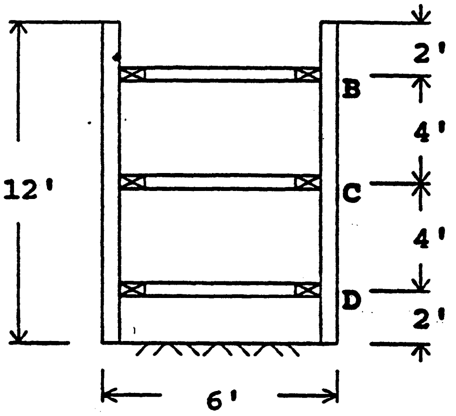
allowable f_c = $480{,}000/[L/d]^2$ = $480{,}000/[(4.33)(12)/6]^2$
= 6,400 psi > 1,600 max, use 1,600 psi
1,600 controls > 270 psi o.k.

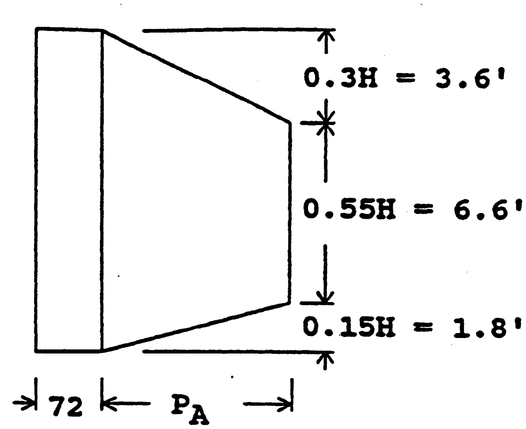
SUMMARY

Sheeting is satisfactory for the wale spacing shown.
Wales need to be 8 x 8.
Struts okay as shown

A better approach would be to use the more conventional trapezoidal earth pressure diagram as it is easier to work with. This is demonstrated on the following page.

SAMPLE PROBLEM NO. 25 - STRUTTED TRENCH: ALTERNATE ANALYSIS





$P_A = \gamma H - 2q_u = (115)(12) - (2)(2{,}000) \leq 0$

Use Stability Number Method:
$N_0 = \gamma H/C$ (115)(12)/600 = 2.3
$P_A = (C/150)(7N_0^{\,2} + 10N_0) = 240$ psf
P_a = Minimum Surcharge = 240 + 72 = 312

Top strut = (240){(4 + 0.4)/2}(6) + (72)(4)(6) = 4,896 Lb
Center strut load = (312)(4)(6) = 7,488 Lb
Bottom strut = (240){(4 + 2.2)/2}(6) + (72)(4)(6) = 6,192 Lb
(All of which are less than the Tschebotarioff analysis)

Determine maximum moment of sheeting.

M_C = -(240)(4.2) [4.2/2] - {(240)(1.8)/2}[4.2 + 1.8/3]
-(72)(6)[6/2] + (6,192/6)[4] = -322

M_{CD} moment at midpoint between C and D
= -(240)(2.2)[2.2/2] - {(240)(1.8)/2}[2.2 + 1.8/3]
- (72)(4)[4/2] + (6,192/6)[2] = 302

M_{DE} = -(240)(0.2) (0.1) - {(240)(1.8)/2}[0.2 + 1.8/3]
- (72)(2)[2/2] = -322

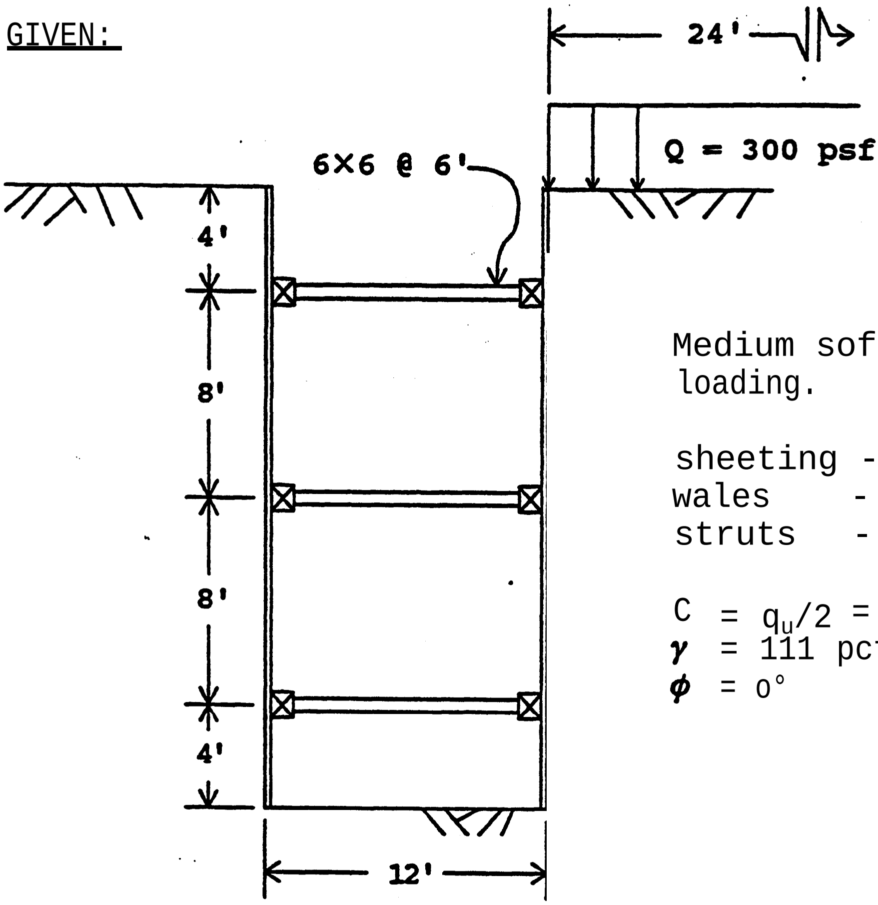
Use M = -322 Ft-Lb/LF < -755 (from previous analysis). ∴ o.k.

By inspection, all other members check.

Center of soil pressure is smaller by this conventional method, but the top and bottom struts are more appropriately loaded.

SAMPLE PROBLEM NO. 26 - STRUTTED TRENCH: (Medium Soft Clay)

GIVEN:



Medium soft clay, short term loading.

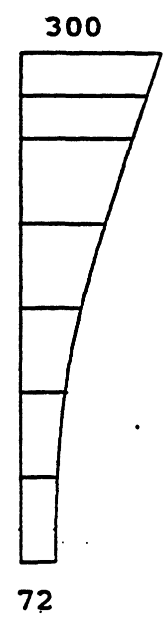
sheeting - 4 X 12 (Rough)
wales - 12 X 12 (Rough)
struts - 6 X 6 (Rough)

$C = q_u/2 = 400$ psf
$\gamma = 111$ pcf
$\phi = 0°$

SOLUTION:

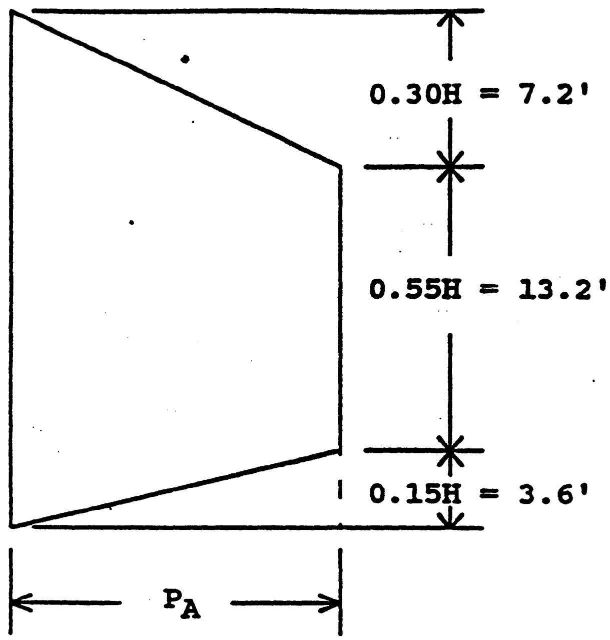
Boussinesq strip surcharge values:

Depth	Lateral Pressure
2	268
4	237
8	181
12	135
16	100
20	73
24	72 (minimum)



APPENDIX G

Soil Pressure Diagram



Soil Pressure Diagram (Terzaghi & Peck)

$$
\begin{aligned}
P_A &= \gamma H - 4C \\
&= 111(24) - 4(400) \\
&= 1064 \text{ PSF}
\end{aligned}
$$

OR

$$
\begin{aligned}
P_A &= 0.375\gamma H \\
&= 0.375(111)(24) \\
&= 999 \text{ PSF}
\end{aligned}
$$

use P_A = 1064 PSF

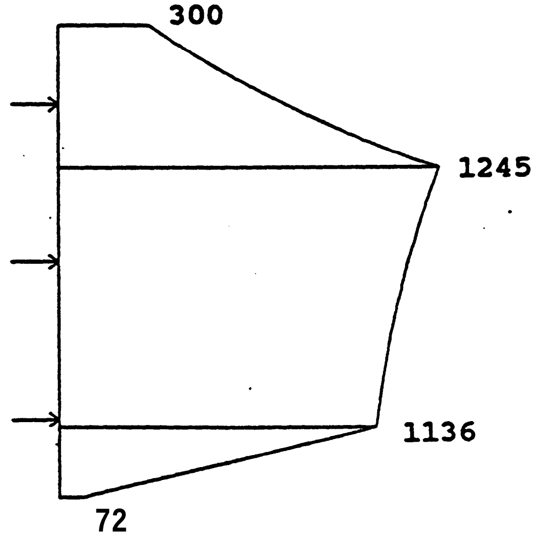
Kw may be computed once the shape of the pressure diagram is established.

Total Area = Total Pressure

$$
\begin{aligned}
(0.3H)P_A/2 &= 0.150P_AH \\
(0.55H)P_A &= 0.550P_AH \\
(0.15H)P_A/2 &= \underline{0.075P_AH} \\
\text{Total Force} &= 0.775P_AH
\end{aligned}
$$

Normalizing = 0.8/(0.775)(0.8) = 0.8258
P_A = 0.8258KwH = 1064
Kw = 53.7 PCF

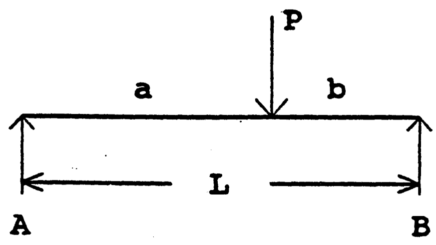
Adding soil and surcharge pressures:



With the loading diagram complete, the strut reactions can be determined using moment shears calculated from the fixed end moments of each span.

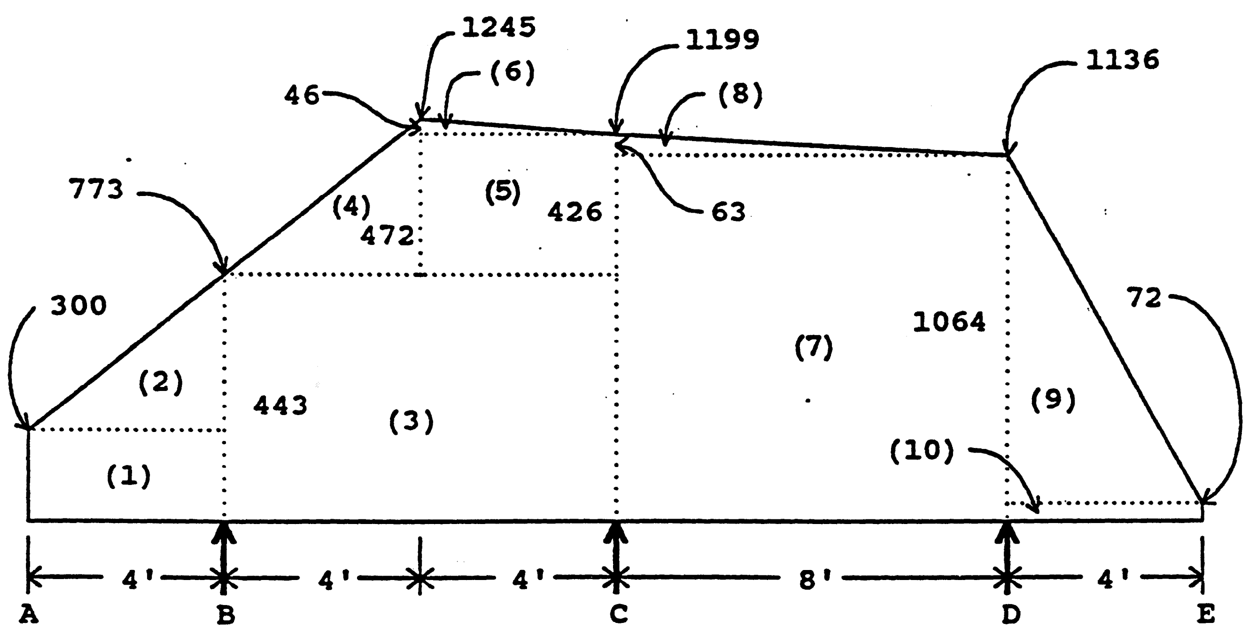
First determine the fixed endmoments and use moment distribution to balance the loads.

For simple moment distribution, small triangles may be converted to concentrated loads placed at the triangle centroids.



P(b/L) = A (left reaction)
A(a) = SBM (simple beam moment at load)
SBM(b/L) = FEM at left Pab^2/L^2
SBM - FEM Lt. = FEM at right

Compute the simple beam fixed end moments.



(1) 300(4) = 1200	$300(4)^2/12$	= 2400		
(2) 473(4)/2 = 946	946(4)/3	= 1261		
	FEM BA	= 3661		
(3) 773 (8) = 6184	$733(8)^2/12$	= 4123		4123
(4) 472(4)/2 = 944	944(4 + (4/3))/8	= 629		
	629 (2/3) (4)	= 1678	SBM	
	1678(4 + (4/3))/8	= 1119		559

(5) 426(4) = 1704	1704(4/2)/8	= 426	
	426(4 + (4/2))	= 556	
	2556(4)/(2)(8)	= 639	1917
(6) 46(d)• = 92	92(2/3)(4)/8	= 31	
	31(4 + (4/3)	= 165 SBM	
	165(2/3) (4)/8	= 55	110
	FEM BC	= 5936	FEM CB = 6709
(7) 1136(8) = 9088	$1136(8)^2/12$	= 6059	6059
(8) 63(8)/2 = 252	252(2/3) (8)/8	= 168	
	168(8)/3	= 448 SBM	
	448(2/3)(8)/8	= <u>299</u>	<u>149</u>
	FEM CD	= 6358	FEM DC = 6208
(9) 1064(4)/2 = 2128	2128(4/3)	= 2837	
(10) 72(4) = <u>228</u>	$72(4)^2/2$	= <u>576</u>	
	FEM DE	= 3413	

Sum of Loads = 22,826 LB/LF

Moment Distribution:

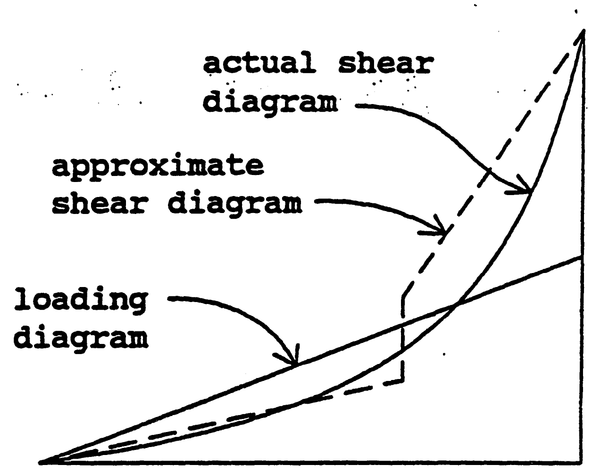
0	1	0.5	0.5	1	0
-3661	+5936	-6706	+6358	-6208	+3413
	<u>-2275</u>	+ 176	+ 176	<u>+2795</u>	
	+ 88	-1138	+1398	+ 88	
	<u>- 88</u>	- 130	- 130	- 88	
	- 65	- 44	- 44	- 65	
	+ 65	+ 44	+ 44	+ 65	
-3661	+3661	-7802	+7802	-3413	+3413

Calculate the strut reactions. This is done by summing the moment shears and the simple beam shears. Moment shears are the difference between the simple beam FEMs divided by the span length. This distributes the load to account for the actual condition of a continuous beam.

Moment Shears		- 518:	+ 518	+ 549	- 549	
Simple Beam Shears		3092	3092	4544	4544	
	1200	629	315	168	84	2128
	946	426	1278			228
		31	61			
Reactions	5806		10525		6495	

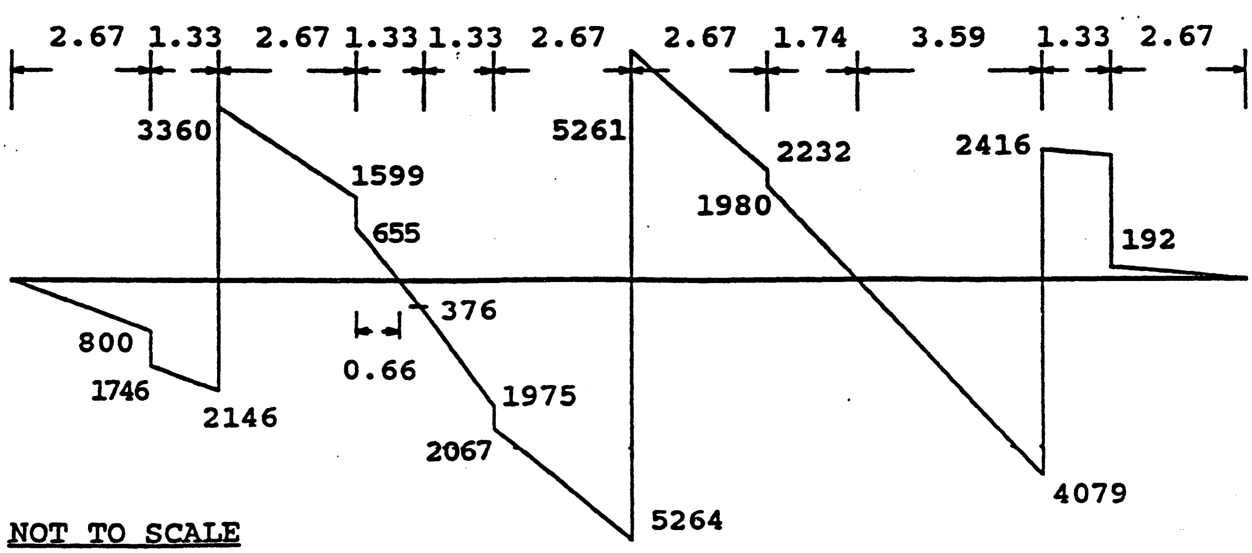
Sum of Reactions = 22,826 = Sum of Loads. CHECK

Determine the shear diagram. The portion contributed by the triangular load can be approximated by breaking it into two straight line slopes at a third point.



Approximate Shears

300(2/3) (4)	=- 800			=- 2067
(2)	=- 946		1199(2/3)(4)	= - 3197
	=- 1746			=- 5264
300(4)/3	= - 400	C		= + 10525
	= - 2146			= + 5261
B	= + 5806		1136(1/3)(4)	= - 3029
	= + 3360			= + 2232
773(2/3)(4)	=- 2061	**(8)**		=- 252
	= + 1599			= + 1980
(4)	= - 944		1136(2/3) (8)	= - 6059
	=+ 655			= - 4079
773 (l/3) (4)	= - 1031	D		=+ 6495
	=- 376			= + 2416
1199(1/3)(4)	= - 1599	(9)		=- 2128
	=- 1975			=+ 288
(6)	= - 92	**(10)**		=- 288
	= - 2067		CHECK	= 0



From the values in the shear diagram calculate the values for l - the approximate moment diagram.

Approximate Moment Calculations:

800(2.67)/2	=	-1068	Ft-Lb/LF
1746(1.33)/2	=	-1116	
2146(1.33)/2	=	-1427	
	=	-3656	
3660(2.67)/2	=	+4886	
1599(2.67)/2	=	+2135	
	=	+3365	
655(0.66)/2	=	+ 216	
	=	+3581	
376(0.67)/2	=	- 126	
	=	+3455	
376(1.33)/2	=	- 250	
1975(1.33)/2	=	-1313	
	=	+1892	
2067(2.67)/2	=	-2759	
5264(2.67)/2	=	-7027	
	=	-7894	MAXIMUM
5261(2.67)/2	=	+7023	
2232(2.67)/2	=	+2980	
	=	+2109	
1980(1.74)/2	=	+1723	
	=	+3832	
4079(3.59)/2	=	-7322	
	=	-3490	
2416(1.33)/2	=	+1607	
2320(1.33)/2	=	+1534	
	=	- 340	
192(2.67)/2	=	+ 256	
	=	- 84	Close enough - CHECK

Check the shoring system members.

Sheeting 4 X 12 (Rough)

$S = bh^2/6 = 12(4)^2/6 = 32 \text{ in}^3$
S required = M/F = 7894(12)/(1500)(1.33) = 47 in^3

Try 6 X 12 (Rough)

$S = 12(6)^2/6 = 72 \text{ in}^3$ OK

Note that the 0.6 lagging arching factor is not used with sheeting or with soft clay.

Wales 12 X 12 (Rough)

$S = 12(12)^2/6 = 288\ in^3$

Center controls, use $M = w1^2/10$

$S\ required = 10525(6)^2(12)/(10)(l.33)(1500) = 228\ in^3$

Length for shear = 6/2 - 0.5 - 1 = 1.5 Ft.

v = 3(1.5)(10525)/2(12)(12) = 164 psi > llO(1.33) = 146 psi

Try a 12 X 16 (Rough)

v = 128 psi < 146 psi OK

Strut 6 X 6 (Rough)

Center controls.

Strut length = 12 - 2(1.33 + 0.33) = 8.68 Ft.

P/A = 10525(6)/(6)(6) = 1754 psi

Allowable:

480000/(8.68(12)/6)2 = 1592 psi < 1600 psi Maximum

Try a 8 X 8 (Rough)

P/A = 10525(6)/(8)(8) = 987 psi < 1592 psi o.k.

Bearing value on wale = 987 > 350 psi Allowable.
Provide steel plates at ends of struts.

SUMMARY

All materials are too small for 6'-O" spacing of struts.

May use: Sheeting = 6 X 12 (Rough)
Wales = 12 X 26 (Rough)
Struts = 8 X 8 (Rough)
Steel bearing plates will be needed at ends of the struts to provide adequate bearing area to prevent overstess in compression perpendicular to the grain of the wales.

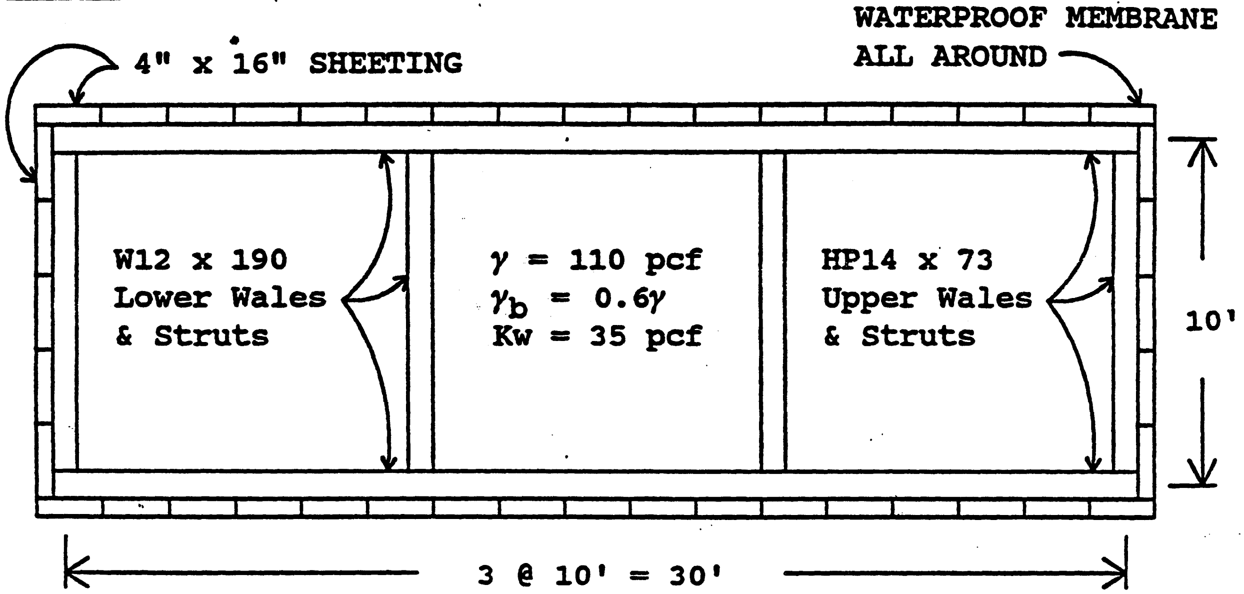
Wale and strut requirements could be reduced by decreasing the spacing of the struts.

APPROXIMATE METHOD

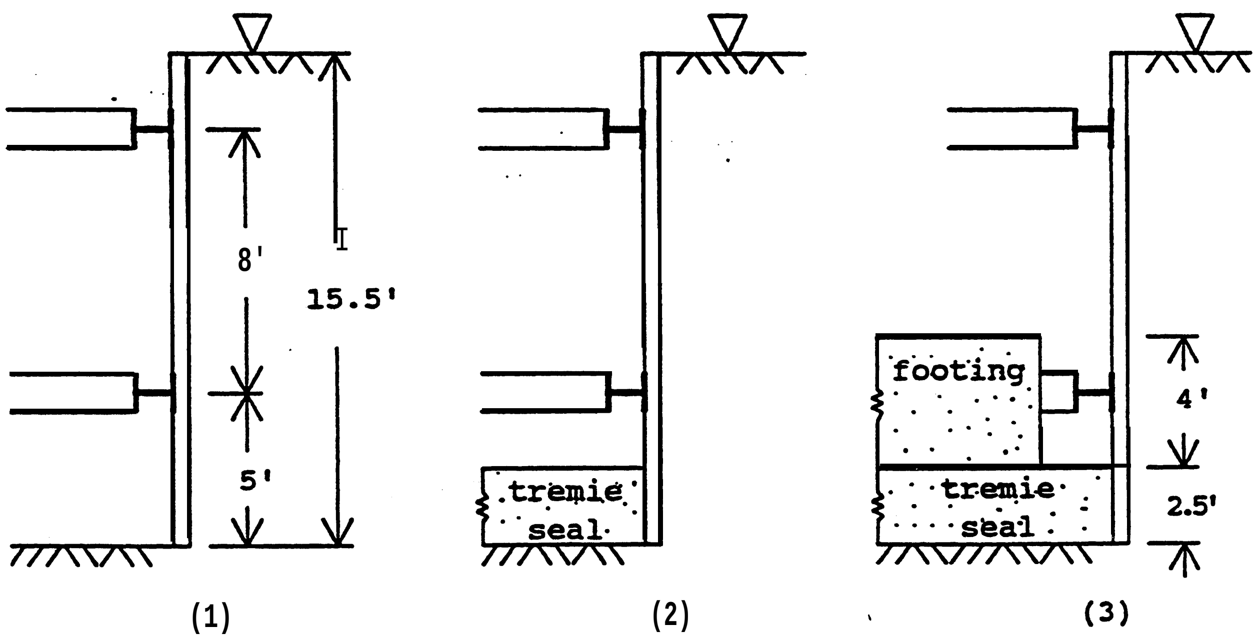
In lieu of doing moment distribution, assume average load on center strut = 1200 PSF X 8 foot strut spacing X l.1 for moment and shear continuity = 1200(8)(1.1) = 10560 (versus 10,525). Check member adequacies.

SAMPLE PROBLEM No. 27 - COFFERDAM

GIVEN:



PLAN - STRUTS 6 WALES



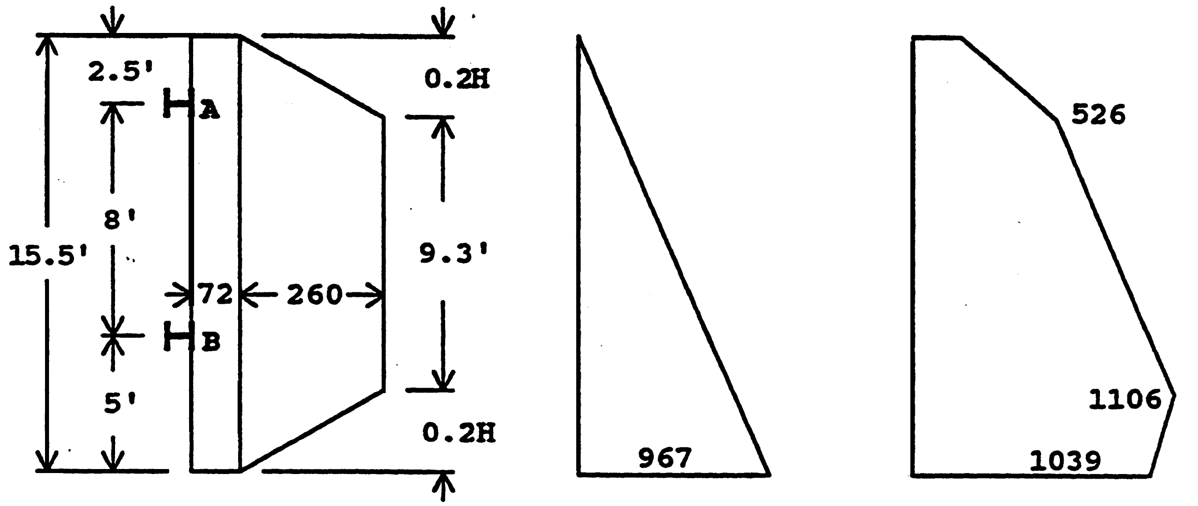
ELEVATION

SEQUENCE:
(1) Excavate and construct cofferdam, set cofferdam and backfill.
(2) Tremie, cure, dewater, and remove lower interior struts.
(3) Construct footing, cure, strut to footing, and remove top interior struts.

CASE (1):

Pressure Diagrams

P_A = (0.8)(Kw)(0.6)(15.5) = 260 psf
Minimum surchargeload = 72.0 psf
Water = (62.4) (15.5) = 967 psf



SOIL WATER COMBINED LOADING

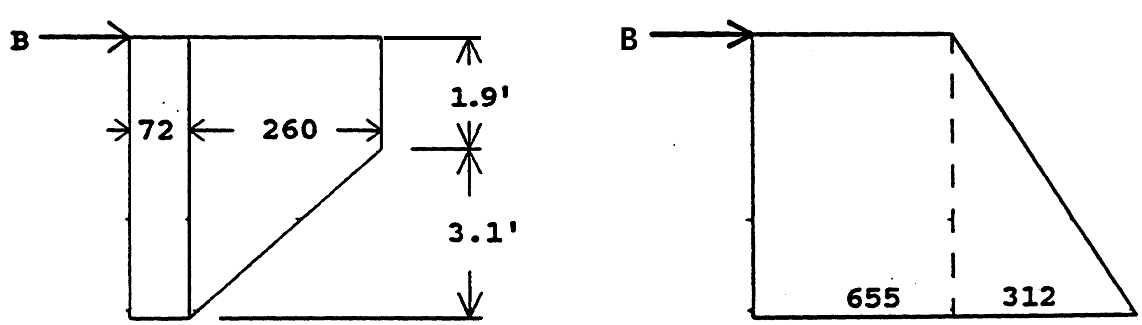
Check tremie vs. combined pressure (without surcharge load)

(2.5)(155) = 388 < 1,039 - 72 = 967 ∴ o.k.

Can the cofferdam be dewatered to permit construction of a footing lowered to the elevation of eliminated tremie seal? All struts to remain in place.

SHEETING

Five foot cantilever will be the most critically loaded location.



ΣM_B = (655) (5) [5/2] + (312) (5)[(5) (2/3)]/2 + (72) (5) [5/2]
+ (260) (1.9)[1.9/2] + (260) (3.1)[1.9 + (1/3) (3.1)]/2
= 13,339 Ft-Lb/LF

For 16" wide timber M_B = (1.33) (13,339) Ft-Lb/Timber
Section Modulus Required = M/f (Use Load Duration Factor = 1.33)
S = (1.33)(13,339)(12)/(1,500)(1.33) = 106.7 in^3
S Furnished = $(16)(4)^2/6$ = 42.67 in^3

SHEETING WILL BE OVERSTRESSED IF COFFERDAM IS DEWATERED!

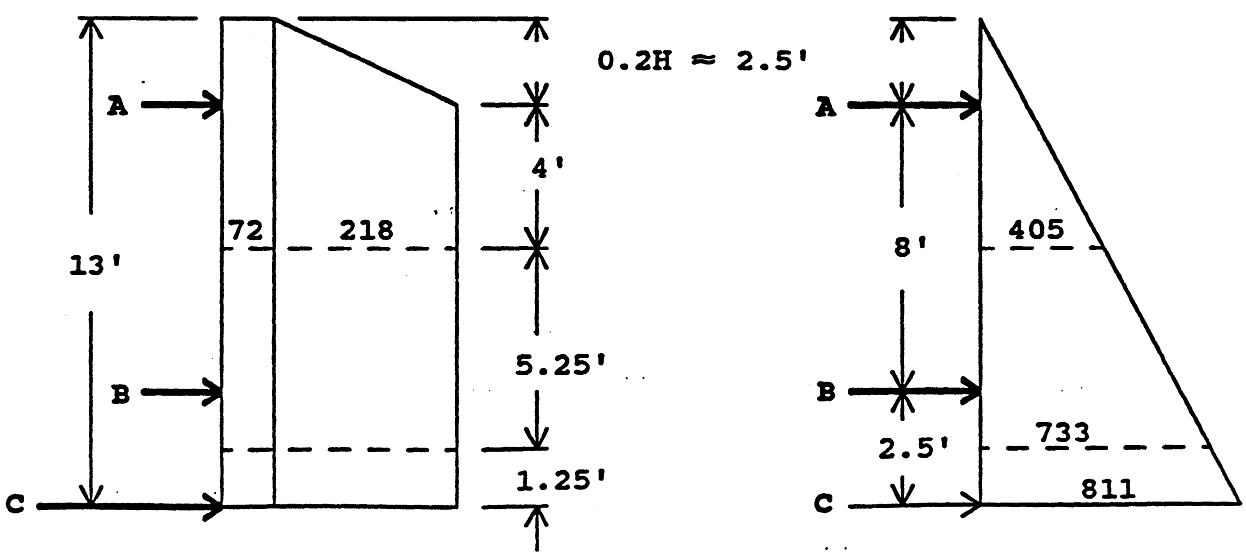
If sheeting had been sufficent at this point, the struts and walers would have been checked next.

If not dewatered:
ΣM_B = (72)(5)[5/2] + (260)(1.9)[1.9/2]
+ (260)(3.1)[1.9 + (l/3)(3.1)]/2 = 2,551 Ft-Lb/LF
S required = (2,551/13,339)(106.7) = 20.4 < 42.67 in^3

CASE (2):

Place tremie concrete , dewater, and remove lower struts.

Pressure Diagrams (above tremie concrete)
P_A = (0.8)(3.5)(0.6)(13) = 218 psf
Water = (62.4)(l3) = 811 psf



Determine reactions by approximate method of area division, then to approximate moment distribution arbitrarily increase B by 10% and prorate reaction difference to A and C.

APPENDIX G

REACTIONS

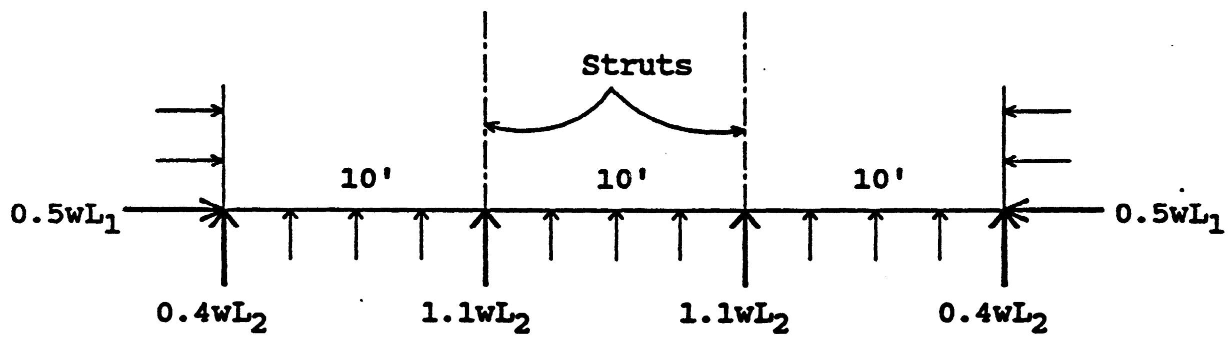
A = (218 + 72)(6.5) - (218)(2.5)/2 + (6.5)(406)/2
= 2,932 Lb/LF
B = (218 + 72)(5.25) + (405 + 733)(5.25)/2
= 4,510 Lb/LF
C = (218 + 72)(2.5)/2 + (733 + 811)(1.25)/2
= 1,327 Lb/LF

Total Load = (218 + 72)(13) - (218)(2.5)/2 + (811)(13)/2
= 8,769 Lb/LF

Increase B by 10%: (4,510)(1.10) = 4,961 Lb/LF
Difference from previous value = 4,961 - 4,510 = 451
Prorate difference to A and C. 2,932/(2,932 + 1,327) = 0.69%

A = 2,932 - (0.69)(451) = 2,621 Lb/LF
C = 1,327 - (0.31)(451) = l,187 Lb/LF

LOADING CONFIGURATION



LOWER STRUT

Properties of a W12 x 190: $I = 1,890\ in^4$, $S = 263\ in^4$, $A = 55.9\ in^2$, $r_x = 5.82$ in, $r_y = 3.25$ in

End load = (1.1)(4,961)(10) = 54,571 Lb
P/A = 54,571/55.9 = 976 psi
$f_{all} = 16,000 - 0.38(L/r)^2 = 16,000 - 0.38[(10)(12)/3.25]^2$
$= 15,482$ psi

LOWER WALE

use $(P/A)/f_{all} + (M/S)/FB \leq 1.0$
$f_{all} = 16,000 - 0.38[(10)[12)/5.82]^2 = 15,838$ psi

Check short side:

End load = (0.4) (4,961)(10) = 19,844 Lb
$M = (4961)(l0)^2/8$ = 62,013 Ft-Lb = 744,156 in-Lb
(19,844/55.9)/15,838 + (744,156/263)/22,000 = O.15 < 1.0 ∴ o.k.

Check long side:

End load = (0.5) (4,961)(10) = 24,805 Lb
$M = (4,961)(10)^2/10$ = 49,610 Ft-Lb = 595,320 in-Lb
(24,805/55.9)/15,838 + (595,320/263)/22,000 = 0.13 < 1.0 ∴ o.k.

UPPER STRUT

Properties of a HP14 x 73: $I = 734\ in^4$, $S = 108\ in^3$,
$A = 21.5\ in^2$, $r_x = 5.85$ in, $r_Y = 3.49$ in

End load = (1.1)(10)(2,621) = 28,831 Lb
P/A = 28,831/21.5 = 1,341 psi
$f_{all} = 16,000 - 0.38[(10)(12)/3.49]^2 = 15,551$ psi

UPPER WALE

$f_{all} = 16,000 - 0.381[(10)(12)/5.85]^2 = 15,840$ psi

By inspection short side will be most critical:

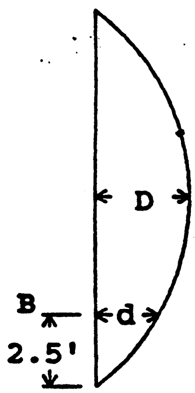
End load = 0.4wL = (0.4)(2,621)(10) = 10,484 Lb
$M = (2,621)(10)^2/8$ = 32,763 Ft-Lb = 393,156 in-Lb
(10,484/21.5)/15,840 + (393,156/108)/22,000 = 0.20 < 1.0 ∴ o.k.

NOTE: Overstresses in steel members should not be allowed because:

- Adequate soil data was not furnished.
- Contractor may not be an expert in cofferdam construction (normally the case).
- It is unlikely that the steel stresses will be frequently checked by means of strain gages.

REMOVE LOWER STRUTS

Moment will be based on proportion of load carried by wale.
Limit deflection of sheeting to L/240 and assume simple span of 10.5 feet.



center Δ = L/240 = (10.5)(12)/240 = 0.5 in
d = (l-x/L) (4) (D) (x)/L)
= (1-2.5/10.5)(4)(0.5)(2.5)/10.5
= 0.36"

Load to be carried by 30' wale assuming simple span for Δ = 0.36"

$\Delta = 0.36 = (5)(w)(30)^4(1{,}728)/[(384)(30\text{x}10^6)\ (1{,}890)]$
$\therefore$w = 1,120 Lb/LF

$M = (1{,}120)(30)^2/8 = 126{,}000$ Ft-Lb = 1,512,000 in-Lb
$(24{,}805/55.9)/15{,}838 + (1.5\text{x}10^6/263)/22{,}000 = 0.29 < 1.0$ $\therefore$ o.k.

SHEETING

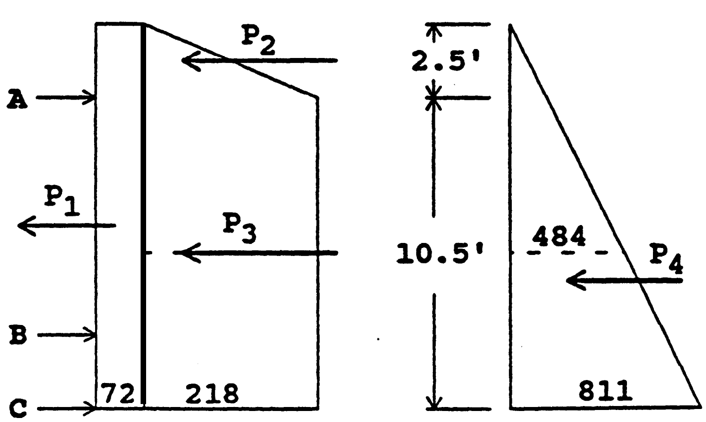
Disregard any reaction at B temporarily and solve for A and C reactions (with sheeting assumed fixed at C) by approximate method of area division.

A = (72)(2.5) + (290)(5.25) + (218)(2.5)/2 + (484)(7.75)/2
= 3,851 Lb/LF
C = (290)(5.25) + [(484 + 811)/2]5.25
= 4,922 Lb/LF

Make reaction at B = 1,120 Lb/LF and decrease A and C proportionally.

A = 3,851 - (2.5/10.5)1,120 = 3,584 Lb/LF
C = 4,922 - (8/10.5)1,120 = 4,069 Lb/LF

Maximum moment will either be at C or somewhere between A and B.



P_1 = (72)(13) = 936

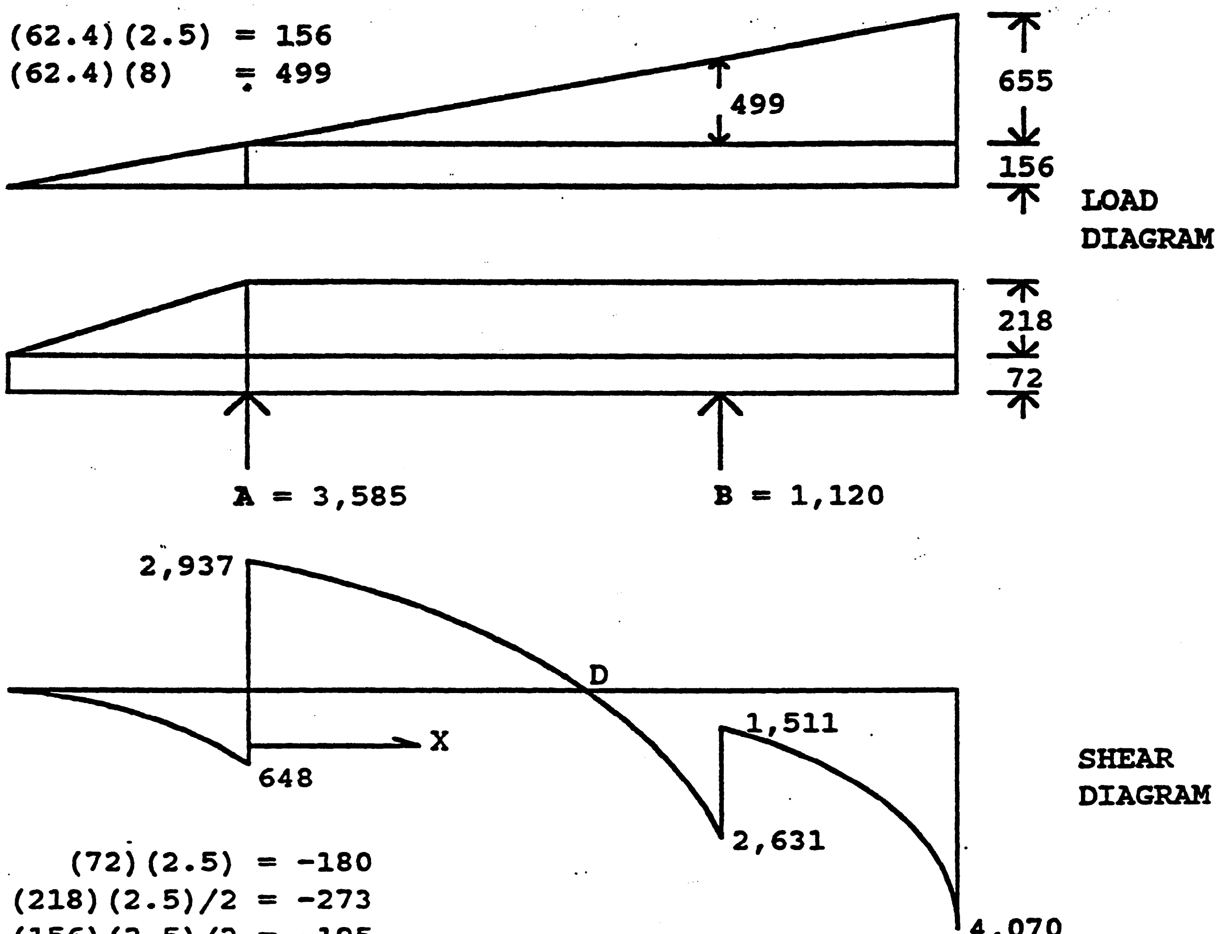
P_2 = (218)(2.5)/2 = 273

P_3 = (218)(10.5) = 2,289

P_4 = (811)(13)/2 = <u>5,272</u>
8,770

$\Sigma M_C = P_1(13/2) + P_2(10.5 + 2.5/3) + P_3(10.5/2) + P_4(13/3)$
$- A(10.5) - B(2.5) = 3{,}609$ Ft-Lb/LF

To find maximum moment between A and B find point of zero shear.



(72)(2.5) = −180
(218)(2.5)/2 = −273
(156)(2.5)/2 = <u>−195</u>
−648

-648
<u>3,585</u>
2,937

2,937
(290 + 156)8 = -3,571
(499)(8)/2 = <u>-1,997</u>
-2,631

-2,631
<u>1,120</u>
-1,511

-1,511
(290 + 156)(2.5) = -1,116
[(499 + 655)/2](2.5) = <u>-1,443</u>
-4,070

-4,070
<u>4,076</u>

$2937 - (290 + 156)X - (62.4)\ (X)^2/2 = 0$
$\therefore X = 4.9'$

$\Sigma M_D = (72)(7.4)[7.4/2] + (218)(2.5)[4.9 + 2.5/33/2$
$+ (218)(4.9)[4.9/2] + (62.4)(7.4)^2[7.4/3]/2 - (3{,}584)[4.9]$
$= -7{,}197$ Ft-Lb/LF > 3,609 $\therefore$ Controls

S required = (16/12)(7,197)(12)/(1500)(1.33) = 57.6 > 42.67 in^3

LOWER STRUTS SHOULD NOT BE REMOVED !!

If the lower struts could have been removed, the wale load at the short ends would have increased from 0.4wL to 1.5wL and the struts would be rechecked for structural adequacy.

CASE (3):

Can the top struts be removed if the lower struts remain in place?

Pressure diagrams will be the same as for CASE(2). Initial reactions are A = 2,621, B = 4,961, and C = 1,187 Lb/LF. Without the top struts, the upper wales and sheeting will deflect inward increasing the reaction at B and decreasing the reaction at C.

Deflection of the sheeting and upper wale at A must be the same. Determine the load that can be carried by the upper wale. Limiting load to be determined by the combinationof axial and bending stress, or by the load which limits deflection to L/240. Determine deflection for limiting load. Determine the maximum load the sheeting will take to have the same deflection as the wale at the location of the A reaction. If the sum of the wale and sheeting limiting loads are less than the reaction of A (2,621 Lb/LF), the wale or the sheeting will be overloaded and/or overstressed. Assume the sheeting to be a cantilever fixed at B.

UPPER WALE

Determine allowable load for 30' span

End load = 0.5wL = (0.5)(2,621)(10) = 13,105 Lb
$M = w(30)^2/8 = 112.5w$ Ft-Lb = 1350w in-Lb
$f_{all} = 15{,}840$ psi

$(13{,}105/21.5)/15{,}840 + (1350w/108)/22{,}000 = 1.0$
$\therefore$ limiting wale load w = 1,692 Lb/LF

Determine maximum deflection.

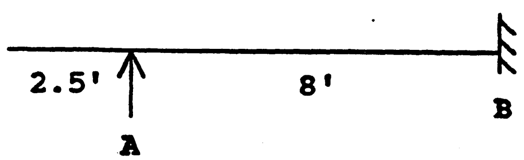
Limiting load deflection A = $5wL^4/384EI$
$= (5)(1,692)(30)^4(12)^3/(384)(30\times10^6)(734) = 1.4"$

$\Delta \le L/240 = (30)(12)/240 = 1.5"$
∴Limiting load on wale is governed by stresses rather than deflection.

SHEETING

Limit sheeting at reaction A to wale deflection of 1.4"

Δ at A = $[(w)(b)^3(12)^3/3EI)$
where b = 8'



$I = (16)(4)^3/12 = 83.33 \text{ in}^4$
$= 64 \text{ in}^4/\text{LF}$

$1.4 = [(w)(8)^3(12)^3/(3)(1.6\times10^6)(64)$
∴w = 486 Lb/LF

1,692 + 486 = 2,178 < 2,623

WALE WILL BE OVERLOADED - DO NOT REMOVE TOP STRUTS !!

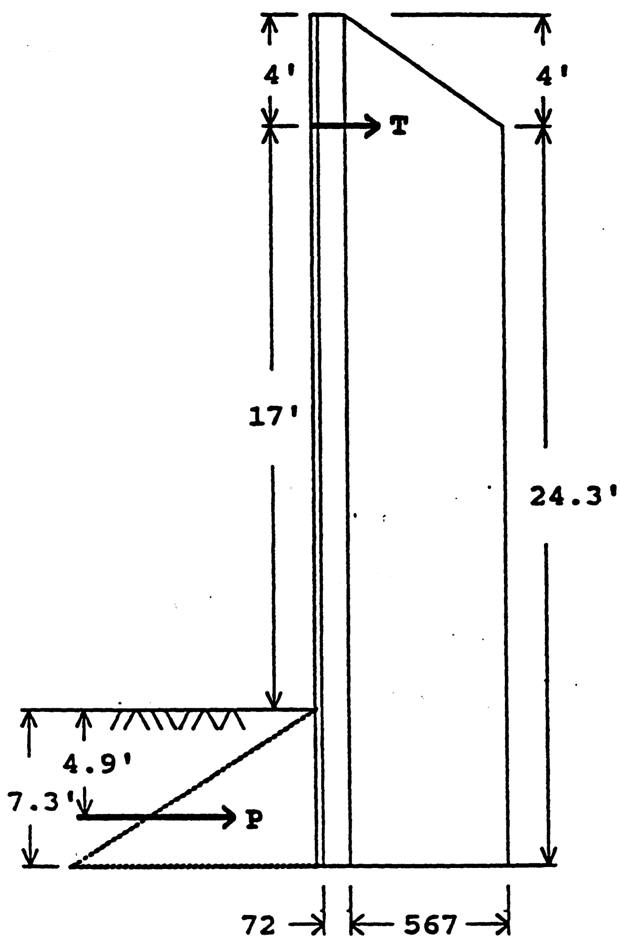
Not checked:

Wale web crippling
Welding
Sheeting to wale connections
Buoyancy

SAMPLE PROBLEM NO. 28 - DEFLECTION

Horizontal movement, or deflection, of shoring systems can only be roughly approximated because soils do not apply pressures as a true equivalent fluid, even in the totally active state. A deflection calculation can be made by structural mechanics procedures (moment area - M/EI) and then some soundengineering judgment should be used. In general, cohesionless soils will give about l/2 the calculated values, & cohesive even less. The time that the shoring is in place will also affect movement. Monitoring, or performance testing, is the final answer.

Following is an example of a deflection calculation for a sheet pile with a single tieback. It is assumed that the lock-off load of the tie is sufficient to preclude any movement at the tie support. The moment-area method will be used to calculate deflections.



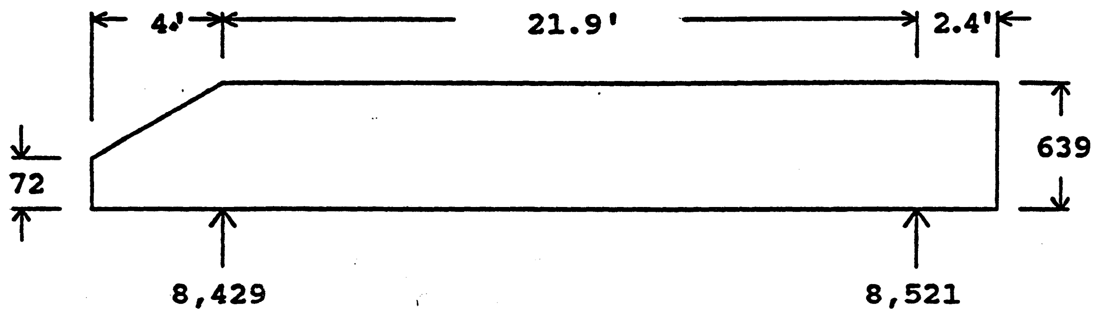
P = the total passive reaction
= 8,521 Lb/LF

T = 8,429 Lb/LF

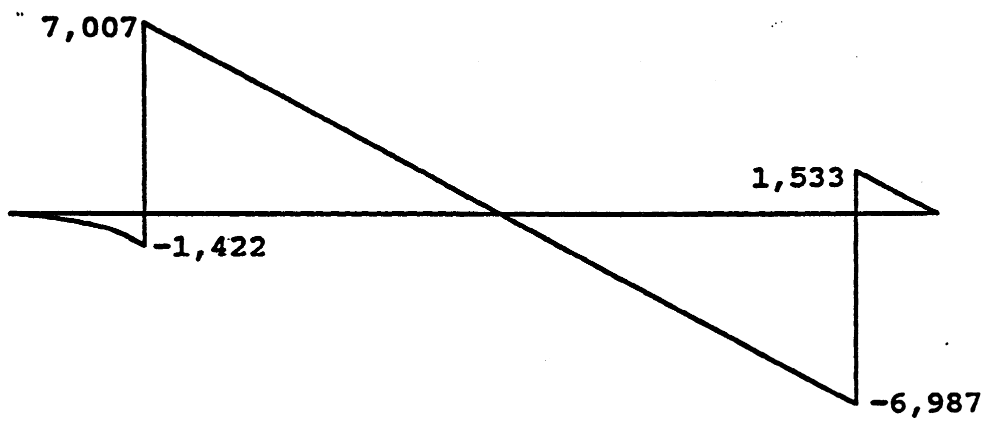
E = 30 x 10^6 psi

I = 109 in^4

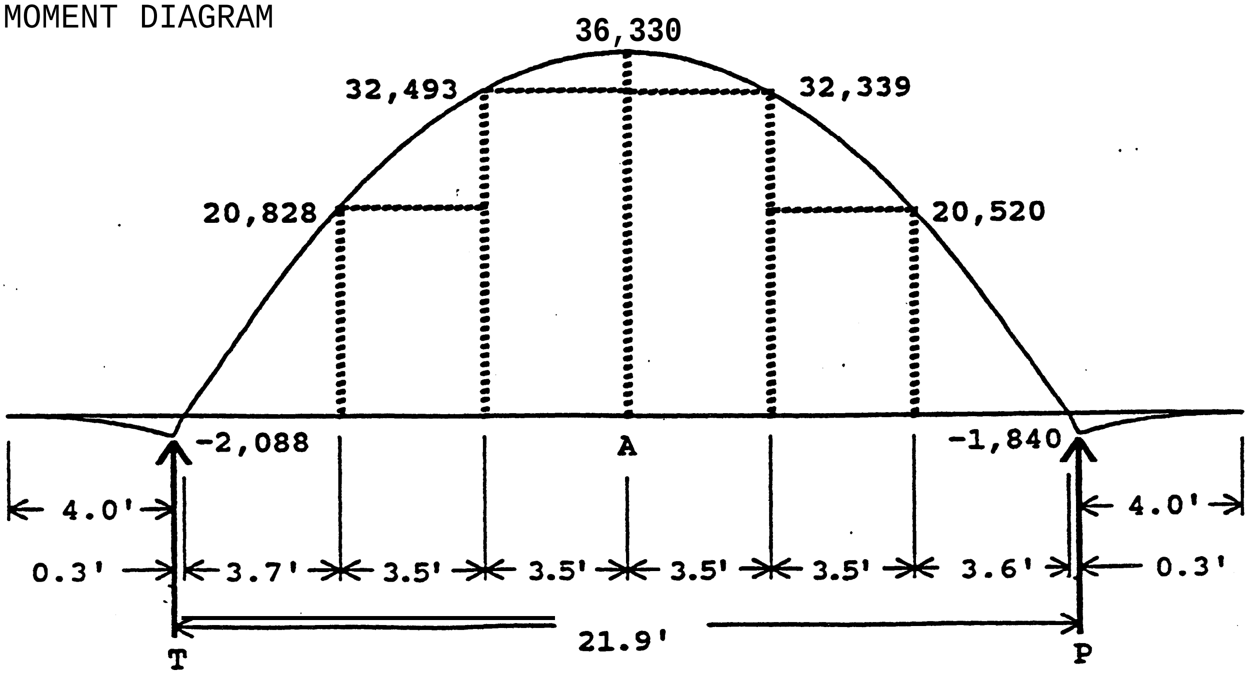
LOAD DIAGRAM



SHEAR DIAGRAM



MOMENT DIAGRAM

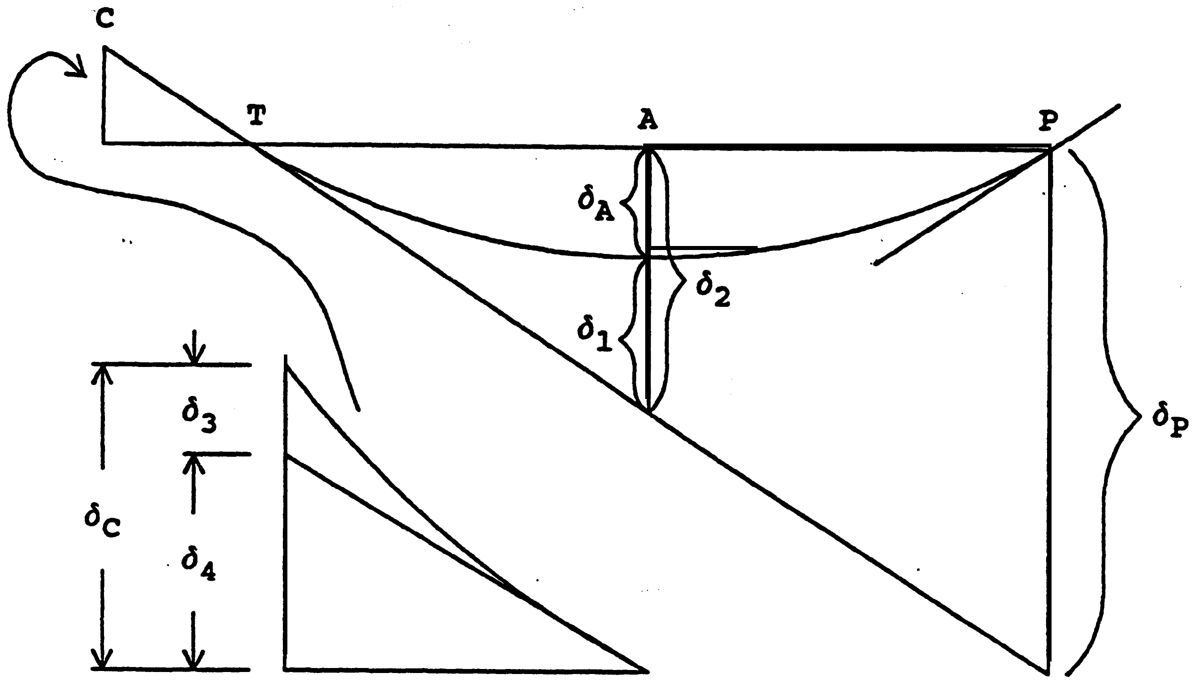


Determine the deflection δ_P

$\delta_2 = (\delta_P)(11.0/21.9)$

Determine the deflection of the tangent to the elastic curve at the centerline from the tangent at T (δ_1).

The true deflection at A $= \delta_2 - \delta_1 = \delta_A$



$$\delta_P = ((-2{,}088)(0.3/3)[21.6 + (3/4)(0.3)] + (36{,}330)(10.7)(2/3)[10.9 + (3/8)(10.7)] + (36{,}330)(10.6)(2/3)[0.3 + (5/8)(10.6)] - (1{,}840)(0.3/3)[0.3/4])(1{,}728)/(30 \times 10^6)(109) = 2.98 \text{ in}$$

$$\delta_2 = (2.98)(11.0/21.9) = 1.50 \text{ in}$$

$$\delta_1 = (-2{,}088)(0.3/3)[10.7 + (3/4)(0.3)] + (36{,}330)(10.7)(2/3)[(3/8)(10.7)](1{,}728)/(30 \times 10^6)(109) = 0.55 \text{ in}$$

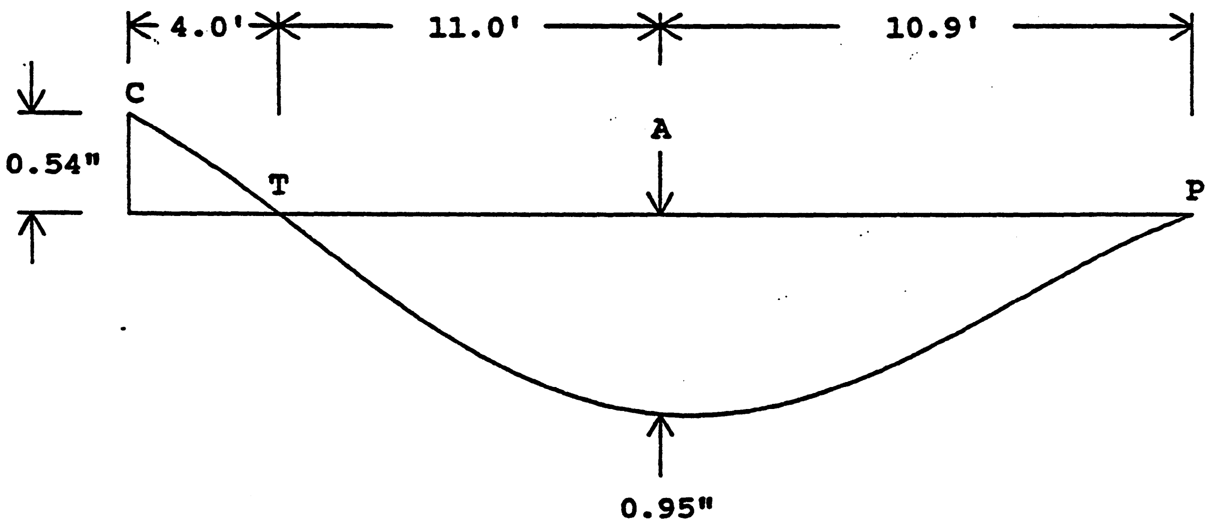
$$\delta_A = 1.50 - 0.55 = 0.95 \text{ in}$$

The deflection at the cantilever section will be the sum of the difference between the tangents to the elastic curves at point T and C: (δ_3) and δ_4.

δ_3 = ((-2,088)(4.0)/4)[(4,5)(4)](1,728)/(30 x 10^6)(109)
= -0.004 in

By similar triangles, δ_4 = (2.98)(4.0/21.9) = 0.544 in

δ_C = $\delta_3 + \delta_4$ = -0.004 + 0.544 - 0.54 in



Index to Appendix H – Trenching and Shoring Memos

APPENDIX H

SLOPING SURCHARGES

ANALYTICAL TRIAL WEDGE FOR SHORING SYSTEMS

UTILIZING COHESION IN COMBINATION WITH GRANULAR SOIL

An alternative to the semi-graphical trial wedge is the analytical trial and error approach. The analytical method involves equating the forces acting on a straight line failure plane, at an assumed failure angle (α), then solving for the maximum resisting active resultant pressure (P_h).

This analytical method will be useful for irregular slopes and for slopes where Rankine, Coulomb or Log-spiral applications are not appropriate. In addition, when the soil contains clay the additional clay shear resistance acting along the. active failure plane surface effectively reduces the active wedge driving force.

The analytical procedure consists of equating forces, assuming a failure angle, then computing a resulting pressure. Computation is repeated until the failure wedge angle selected produces the maximum resulting active pressure. The analytical procedure is ideal for computer application.

The maximum resulting active pressure can be used to determine an equivalent fluid pressure (Kw) which would simulate a soil having a level surface acting on the height of the shoring system. From the equivalent fluid (Kw) value a fictitious K_A value may be computed. The method for converting the maximum resulting pressure to Kw and then to a hypothetical K_A times the soil unit weight is demonstrated in Part 2.

Active wedge movement or clay shrinkage will permit the development of clay tension cracks. Soil can lose apparent cohesion, due to loss of capillary tension for example. The tension cracks will eventually extend to the wedge failure plane with the maximum crack height equaling the critical height of the clay.

When tension cracks can develop, it will only be necessary to consider the soil wedge weight, including any surcharge, between the limits of the shoring and the nearest tension crack that intercepts the failure plane. Otherwise, the entire wedge soil weight, including surcharge, should be deemed to be acting as the driving force on the failure plane.

Cohesive resistance acting along the active failure plane should only be considered to be effective between the limits of the shoring and any tension crack that intercepts the failure plane.

If tension cracking will not occur, the entire length of the failure plane will provide cohesive resistance. Limiting the length of the clay resistance along the hypothetical failure line is demonstrated in Part 3 and Part 4.

When it is possible for water to stand in tension cracks the additional horizontal reaction must be added to the other force components. The horizontal component of water $(P_w = 0.5\gamma_w h_c^2)$ will act at one-third the height of the crack and will oppose the horizontal component of the shoring resisting force (P_h).

When soilshear strength has both frictional and cohesive components these values must be confirmed and be appropriate. Log-of-Test Borings accompanying contract plans do not indicate both cohesion and friction angle for the same boring. It will generally be necessary for the Contractor to furnish soils information from a recognized soils lab or similar source.

Verification of soil shear strength with values derived from the Log-of-Test Borings should be based on a conservative interpretation of the Log-of-Test Borings and the relationships between Standard Penetration Test (SPT) N-values and soil shear strength values. Generally speaking, unconfined compressive strength derived from SPT N-values for a cohesive soilshould be based on a more conservative interpretation than that used to derive a soil friction angle from SPT N-value for a granular soil. This also means that when TABLES 12 and 13 are used conservative values should be selected. Conformance with the foregoing will obviate the need to apply safety factors to the soil strength values.

Wall friction (6) may be considered when the shoring will not be subjected to dynamic loading, or when shoring construction does not include lagging. For the active condition the wall friction acts vertically upward and effectively reduces the force of the weight acting down. It is recommended that no wall friction be used for the passive case.

It should be noted that weak seams in the soil can predispose the soil mass to fail along the seam rather than along the failure plane, particularly if water is present in the seam. Weak seam slippage would prevent the full wedge from becoming active. If weak seams exist in the passive wedge undesirable events will most probably result.

The analytical development for trial wedges assumes homogeneous isotropic soil. Surcharges on the active wedge merely add weight to the wedge, and with the analytical method any surcharges situated beyond the failure plane, or behind tension cracks, are neglected.

APPENDIX H

Analytical trial wedge development is presented in the following sequence:

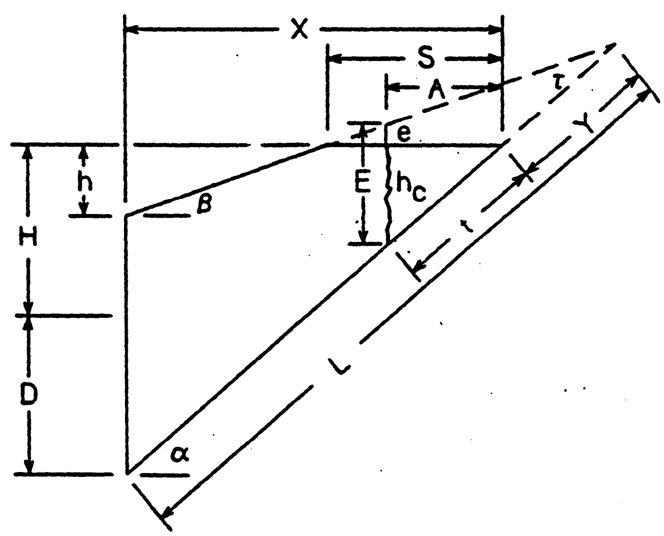
PART 1: Effect of tension crack height and location on the active trial wedge.

PART 2: Analytic&l active-trial wedge for strutted type systems assuming no clay tension crack has developed in the wedge. An example problem is included.

PART 3: Analytical active trial wedge development for a continuous cantilever shoring wall with a surcharge situated over a clay tension crack.

PART 4: Example problem demonstrating passive resistance for the active wedge of PART 3 for a level and for a downward sloping surface. Recommended safety factors against sliding and overturning are included.

PART 1: EFFECT OF TENSION CRACK ON THE ACTIVE WEDGE



LOCATION OF TENSION CRACK

Three different conditions can exist for the location of the maximum potential clay tension crack: (1) it can be located within the semi-level top of embankment area as shown in the adjacent figure where S > A, (2) it can be located below the hinge point so that A >S, or (3) it may be located on a slope (S = 0) when the slope line intersects the failure plane. With no surcharge on the embankment the essential equations for the length (t) and for the contributing weight (W) are as follows:

1) S > A

$$S = X - h/\tan\beta$$
$$Y = S(\sin\beta)(\sin\tau)$$
$$A = h_c/\tan\alpha = t\cos\alpha$$

$t = h_c/\sin\alpha = A/\cos\alpha$
$E = (t + Y)(\sin\tau)/\sin(90° + \beta)$
$e = E - h_c$
$S - A = e/\tan\beta$
$W = \gamma/2[(H + D - h + E)(X - A) - e^2/\tan\beta]$

2) $S < A$

$S = X - h/\tan\beta$
$t = [h_c\sin(90° + \beta) - S(\sin\beta)]/\sin\tau$
$A = t\cos\alpha$
$W = \gamma/2[(H + D - h + h_c)(X - A)]$

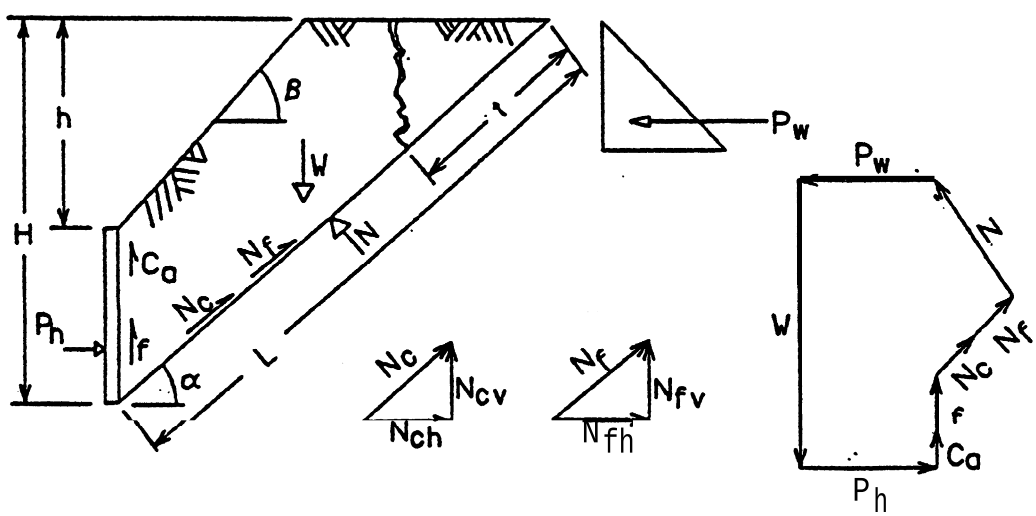
3) $S = 0$

$t = h_c\sin(90° + \beta)/\sin\tau$
$A = t\cos\alpha$
$W = \gamma/2[(H + D - h + h_c)(X - A)]$

When a surcharge of a permanent nature (not traffic) is situated on the embankment the depth of the tension crack (b) which might. be located under the surcharge (Q) is reduced as indicated by the following equation:

Under surcharge (Q): $h_c = (2C/[K_A]^{1/2} - Q)/\gamma$

PART 2. ANALYTICAL ACTIVE TRIAL WEDGE FOR STRUTTED TYPE SYSTEMS



APPENDIX H

The following is an analytical development of an active wedge for a strutted type system with an intermix of cohesionless and cohesive soils. Assume insufficient time for a significant clay tension crack to develop. Assume also that no water will get into tension cracks and that no friction or other cohesive forces will develop on the soil face of the shoring. An example problem is included.

N_f = Frictional resistance (Granular)

N_c = Cohesive resistance

γ = Unit weight of the soil
ϕ = Soil internal friction angle
C = Clay cohesion
α = Assumed angle of wedge failure
W = Wedge weight plus surcharge

Shear on failure plane: $S = C + N\tan\phi = N_C + N_f$

The relatively small wall adhesion (CJ forces will not be considered to simplify the calculations. Wall friction (δ) and water pressure (P_W) are not included in this example.

$N_f = N\tan\phi$ $\qquad$ $N_{fH} = N\tan\phi\cos\alpha$

$N_{fV} = N\tan\phi\sin\alpha$

$N_C = LC = HC/\sin\alpha$

$N_{CH} = N_C\cos\alpha = HC(\cos\alpha)/\sin\alpha$ $\qquad$ $N_{CH} = HC/\tan\alpha$

$N_{CV} = N_C\sin\alpha = HC(\sin\alpha)/\sin\alpha$ $\qquad$ $N_{CV} = HC$

EQUATE $\Sigma F_H = 0 \quad \rightarrow +$

$$0 = P_h + N_{CH} + N_{fH} - N\sin\alpha$$
$$0 = P_h + HC/\tan\alpha + N\tan\phi\cos\alpha - N\sin\alpha$$
$$0 = P_h + HC/\tan\alpha + N(\tan\phi\cos\alpha - \sin\alpha)$$

EQUATE $\Sigma F_V = 0 \quad \uparrow +$

$$0 = -W + N_{CV} + N_{fV} + N\cos\alpha$$
$$0 = W - HC - N\tan\phi\sin\alpha - N\cos\alpha$$
$$N = (W - HC)/(\tan\phi\sin\alpha + \cos\alpha)$$

Or transposing: $N = (W - HC)\cos\phi/\cos(\alpha - \phi$

From $\Sigma F_H = 0$:

$$P_h = N(\sin\alpha - \tan\phi\cos\alpha) - HC/\tan\alpha$$

Substitute the N value from the $\Sigma F_V = 0$ **equation**

$$P_h = [(W - HC)/(\tan\phi\sin\alpha + \cos\alpha)][\sin\alpha - \tan\phi\cos\alpha] - HC/\tan\alpha$$

$$P_h = (\sin\alpha - \tan\phi\cos\alpha)(W - HC)/(\tan\phi\sin\alpha + \cos\alpha) - HC/\tan\alpha$$

Transposing: $P_h = (W - HC)\tan(\alpha - \phi) - HC/\tan\alpha$

If C = 0 then $P_h = W\tan(\alpha - \phi)$

Where $W = \gamma/2(H^2/\tan\alpha - h^2/\tan\beta)$, and $\beta \leq \phi$

Note: If there is no bench on the embankment portion of the wedge the equations above are still valid.

Assume: (Generally $45° - \phi/2 < \alpha < 45° + \phi/2$)

Search values of α to determine maximum P_h.

Once maximum P_h is determined an equivalent fluid weight may be calculated from the equation $Kw = 2P_h(H-h)^2$.

If the system will be in place long enough for clay tension cracks to develop, the clay resisting length (L) must be reduced by the length (t) along the failure plane behind the tension crack. The height of the tension crack h_C, determined by the critical height of the clay, is equal to $(2C/[K_A]^{1/2} - Q)/\gamma$, where Q is the surcharge and K_A is for a level surface. The length t may be determined from the triangular relationships $t/h_c = L/H$. Water pressure may now need to be considered in addition to all other forces acting on the system.

EXAMPLE:

Assume foregoing configuration for a strutted trench with H equal to 28 feet, h = 16 feet (which includes 2 feet of soil for an equivalent minimum surcharge), $\beta = 45°$, $\gamma = 115$ pcf, $\phi = 32°$, with $\delta = 0$, and C = 200 psf. This condition is for short term (48 hours) loading, and appropriate safety factors are included in the soil parameters.

Determine maximum active resultant pressure (P_h) and an equivalent fluid weight (Kw). From Kw determine a hypothetical K_A using the given unit weight. Draw a proper pressure diagram for the given height of the shoring system.

SOLUTION:

For short term loading the potential clay cracking will be ignored.

$\tan\phi$ $\tan 32^\circ$ $= 0.625$ $\qquad$ $\tan\beta = \tan 45^\circ = 1.0$

$W = 115/2[(28)^2/\ \tan\alpha - (16)^2/1.00] = 57.5[784/\ \tan\alpha - 256)$

$HC = 28(200) = 5{,}600$

$$P_h = (\sin\alpha - 0.625\cos\alpha)[57.5(784/\tan\alpha - 256) - 5{,}600]/(0.625\sin\alpha + \cos\alpha) - HC/\tan\alpha$$

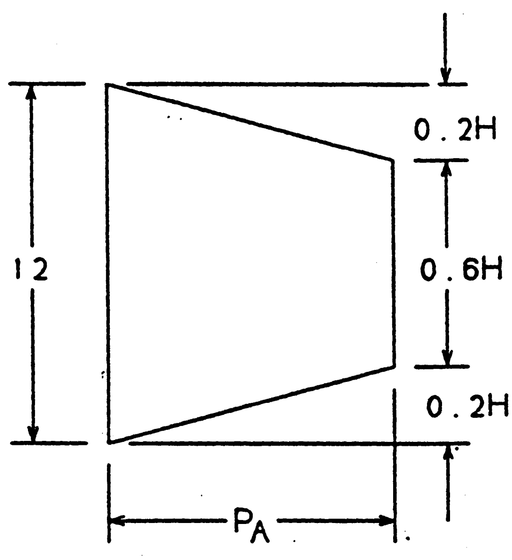
Successively try various angles in the above equation to search for the maximum active pressure:

Maximum P_h = 1,048 Lb/LF (When angle $\alpha = 52^\circ$)

$Kw = 2P/(H - h)^2 = 2(1048)/(28-16)^2$ 15 pcf

Then a hypothetical K_A = Kw/γ = 15/115 = 0.13

And one appropriate pressure diagram to use is the trapezoid for which $P_A = 0.8Kw(H - h) = 0.8(15)(12) = 144$ psf



If C = 0 the exposed slope angle cannot be assumed to be greater than the internal friction angle of the soil $(\beta \le \phi)$

Revising so that C = 0 and $\beta = \phi$:

$W = 115/2[(28)^2/\tan\alpha - (16)^2/\tan\phi]$

Maximum $P_h = W\tan(\alpha - \phi)$ = 4981 Lb/L.F. (When $\alpha = 46^o$)

Then $K_w = 2(4981)/(28-16)^2$ = 69 pcf

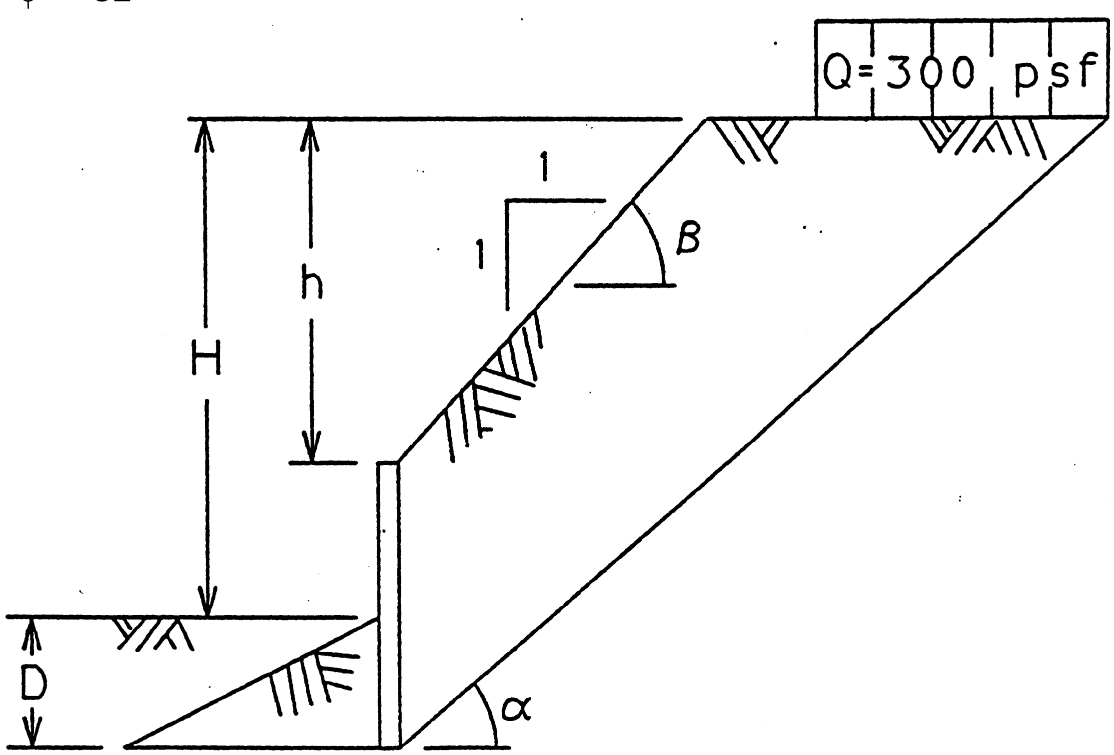
The hyothetical $K_A = K_w/\gamma$ = 69/115 = 0.6

and $P_A = 0.8K_w(H - h) = 0.8(69)(12)$ = 662 psf > 144 psf

PART 3. ANALYTICAL WEDGE FOR A CANTILEVER WALL

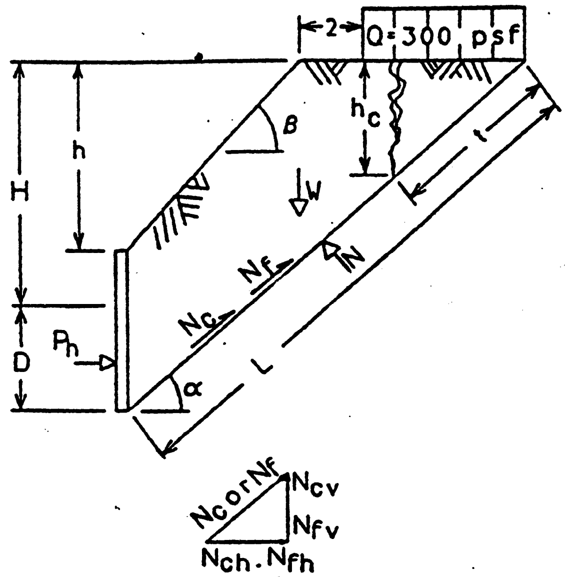
Following is a development of an analytical active wedge for a continuous cantilever type shoring wall with a surcharge and a clay tension crack using the following parameters:

H = 24'	$\beta = 45°$
h = 12'	$\delta = 0$
D = 20'	C = 200 psf
γ = 115 pcf	Surcharge = 300 psf
$\phi = 32^o$	



APPENDIX H

ANALYTICAL ACTIVE WEDGE



With tension crack:

$N_c = C(L-t) = C(H+D)/\sin\alpha$
$-Ch_c/\sin\alpha$

$N_c = C(H+D-h_c)/\sin\alpha$

$N_{CH} = C(H+D-h_c)/\tan\alpha$

$N_{CV} = C(H+D-h_c)$

$N_{fH} = N\tan\phi\cos\alpha$

$N_{fV} = N\tan\phi\sin\alpha$

A) For the case when a tension crack will develop:

$\Sigma F_H = 0$

$0 = P_H + N_{CH} + N_{fH} - N\sin\alpha$

$0 = P_h + C(H+D-h_c)/\tan\alpha + N\tan\phi\cos\alpha - N\sin\alpha$

$P_h = N(\sin\alpha - \tan\phi\cos\alpha) - C(H+D-h_c)/\tan\alpha$

Transposing: $P_h = N\sin(\alpha - \phi)/\cos\phi - C(H + D - h_c)/\tan\alpha$

$\Sigma F_V = 0$

$0 = N_{cv} + N_{fv} + N\cos\alpha - W$ (Where W includes Q)

$0 = C(H+D-h_c) + N\tan\phi\sin\alpha + N\cos\alpha - W$

Solve for N and substitute into the $\Sigma F_H = 0$ equation.

$N = [W - C(H+D-h_c)]/(\tan\phi\sin\alpha + \cos\alpha)$

Transposing: $N = [W - C(H + D - h_c)/(\cos(\alpha - \phi)/\cos\phi]$

$p_h = \{[W - C(H+D-h_c)]/(\tan\phi\sin\alpha + \cos\alpha)\}\{\sin\alpha - \tan\phi\cos\alpha\}$
$- C(H+D-h_c/\tan\alpha$

$P_h = [\sin\alpha - \tan\phi\cos\alpha]\{[W - C(H+D-h_c)]/(\tan\phi\sin\alpha + \cos\alpha)\}$
$\quad - C(H+D-h_c)/\tan\alpha$

Transposing: $P_h = [W - C(H+D-h_c)]\tan(\alpha - \phi) - C(H+D-h_c/\tan\alpha$

$L = (H+D)/\sin\alpha$

Surcharge between shoring and tension crack:

Partial Surcharge = $300\{[(H+D)/\tan\alpha] - h - 2' - h_c/\tan\alpha\}$

$W = \gamma/2[(H + D)^2/\tan\alpha - h^2/\tan\beta - (h_c)^2/\tan\alpha]$ for the soil plus $300[(H+D-h_c)/\tan\alpha - h - 2']$ for the surcharge.

$h_c = (2C/[K_A]^{1/2} - Q)/\gamma = (400/[0.31]^{1/2} - 300)/115 = 3.64'$
(Reminder: tension crack K_A is for level surface.)

P_h = 22,32l Lb/LF (maximum when $\alpha = 57°$)

Confirm that the tension crack is under the surcharge; if it is not, recompute h_c (using $Q = 0$), W, and P_h. In this example the tension crack is under the surcharge.

B) If water might fill the tension crack the transposed P_h equation becomes:

$P_h = [W - C(H + D - h_c)]\tan(\alpha - \phi)$
$\quad - C(H + D - h_c)/\tan\alpha + 0.5\gamma_w(h_c)^2$

For this condition P_A = 22,734 Lb/LF (Maximum when $\alpha = 57°$)

C. If there will be no tension crack:

$W = \gamma/2[(H + D)^2/\tan\alpha - h^2/\tan\beta]$ for the soil plus $300\{[(H + D)/\tan\alpha] - h - 2'\}$ for the surcharge

$P_h = [W - C(H + D)]\tan(\alpha - \phi) - C(H + D)/\tan\alpha$

P_h = 22,072 Lb/L.F. (Maximum when $\alpha = 58°$)

The above equations can be used when C = 0, but in that case the slope angle (β) cannot exceed the interior friction angle (ϕ) of the soil.

Use the maximum active condition (B) to determine safety factors for a passive condition where the slope is not steep but may be continuous, and compare to the condition where the excavated surface may be considered level. This example is demonstrated after the passive wedge development which follows.

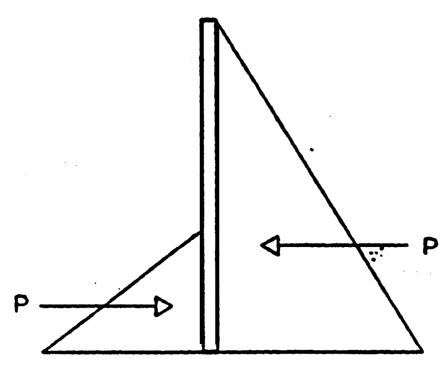
PART 4: PASSIVE WEDGE RESISTANCE

Demonstrate passive wedge resistance for two conditions; one in which the resisting ground surface is level in front of the shoring, and one in which the ground in front of the shoring slopes down and away on approximately a 3 to 1 continuous slope. The appropriate passive wedge resisting pressure is determined from the log-spiral curves of FIGURE 8. A semi-graphical or analytical passive trial wedge solution would not be appropriate since either of these two methods would approximate the Coulomb solution; which has been demonstrated to always be an unsafe approach.

SOLUTION:

	FOR LEVEL SURFACE	SLOPING AT $\approx$ 3:1
β/ϕ	0	$-18.44^\circ/32^\circ \approx -0.6$
K_p Coefficient	7.9	3.2
Reduction Factor	0.425	0.425
K_p	3.36	1.36
D	20'	20'
γ	115 pcf	115 pcf
$P_p = 1/2 K_p \gamma D^2$	77,280 Lb/LF	31,280 Lb/LF

Wall pressures for cantilever systems are normally assumed to act at one-third of the height of a triangle (whose base is located at the bottom of the pressure wedge).



SUMMARY OF PRESSURES:

ACTIVE PRESSURES (Maximum):

With tension crack: P_A = 22,321 Lb/LF
With water in the crack: P_A = 22,734 Lb/LF (Controls)
With no tension crack: P_A= 22,072 Lb/LF

PASSIVE PRESSURES (Minimum):

Sloping embankment below H: P_p = 31,280 Lb/LF
With level surface below H: P_p = 77,280 Lb/LF

Compute the safety factor (S.F.) against sliding:

Passive sloping embankment condition:

$S.F. = P_p/P_A = 31{,}280/22{,}734 = 1.38 < 2.0 \therefore$ n.g.

Passive level surface condition:

$S.F. = P_p/P_A = 77{,}280/22{,}734 = 3.4 > 2.0 \therefore$ o.k.

Compute the safety factor (S.F.) against overturning using the moments taken about the bottom of the shoring:

Passive sloping embankment condition:

M_p for P_p = 31,280(20)/3 = 208,533 Ft-Lb/LF

M_A for P_A = 22,734(12 + 20)/3 = 242,496 Ft-Lb/LF

$S.F. = M_p/M_A = 208{,}533/242{,}496 = 0.86 < 1.5 \therefore$ n.g.

Passive level surface condition:

M_p for P_p = 77,280(20)/3 = 515,200 Ft-Lb/LF

M_A for P_A = 242,496 Ft-Lb/LF

$S.F. = M_p/M_A = 515{,}200/242{,}496 = 2.12 > 2.0 \therefore$ o.k.

The shoring should be satisfactory for the condition where the passive wedge consists of a level surface: providing the ground conditions do not change significantly, including the possibility of the active wedge becoming heavily saturated.

TRENCHING AND SHORING REMINDER LIST

I. WHERE TO FIND TRENCHING SHORING INFORMATION

Standard Specifications
- 5-1.02 Plans & Working Drawings
- 5-l.02a Trench Excavation Safety Plans
- 7-l.0lE Trench Safety
- 19-3.03 Cofferdams

Cal OSHA Excavation & Trench SAFETY ORDERS
Contract Plans - Log Of Test Borings
Contract Special Provisions
Materials Reports - Geotechnical Report For Foundation Work
Trenching & Shoring Manual, Office Of Structure Construction
USS Steel Sheet Piling Design Manual (July 1994)
Bridge Construction Records and Procedures Manual
- Vol 1 8-0.0 Railroads & Related Facilities
- Vol 2 145-10.0 Submitting Shoring Plans

Office Of Structure Construction
- Trenching & Shoring Engineer
  - John Babcock @ (916) 227-8835
- Other experienced Structure Representatives
- Your immediate supervisor

Office Of Structural Foundations - Geotechnical Support

II. INITIAL REVIEW CRITERIA

Deficiencies relative to this section of the check list could cause delays in starting the 'design review' clock.

1. Drawings are to be sufficiently complete to permit review.
2. Complete set of calculations required if not in accordance with Cal OSHA Standards.
3. All sheets of the excavation plan Submittal, if not in accordance with Cal-OSHA Standards, are to be stamped and signed by a Registered California Civil Engineer.
4. Soil classification and other related information provided **(ϕ, δ, γ, k_p & k_a or k_v).**
5. Method and sequence of shoring installation and excavation specified, including temporary bracing.
6. All controlling design dimensions must be shown (H, D, H_S, L, etc.).
7. Sufficient detail and supporting data is included to permit a complete stress analysis, including duration of excavation and start time.
8. Verify constructablity at location specified. Will the shoring plan infringe upon existing facilities or utilities.
9. Manufacturer's brochure and engineering data submitted if appropriate.

III. DESIGN REVIEW REQUIREMENTS

1. Monitor review time
   - 3 weeks for engineered trenches[1].
   - 5 days for trenches in accordance with Cal-OSHA Standards[1].
   - 3 weeks for cofferdams[2].
   - review Special Provisions for project specific requirements.
   - 12 weeks for locations requiring plan review by a railroad[2].
2. Maintain a correspondence log for each shoring plan submitted Include entries for stopping and starting the design review time.
3. Verify all surcharges are considered (railroad, traffic, K-rail, existing facilities, etc). Use a minimum 72 psf (3510 N/m^2) surcharge. SPTC (Southern Pacific Transportation Company) requires use of the Boussinesq pressure distribution to determine Lateral pressure on shoring.
4. Determine the effect of construction sequencing and activities on the excavation.
5. Review the contractor's submittal for confined space requirements in accordance with Cal-OSHA requirements.
6. Verify contractor has accounted for water in the design if it is present, or has specified dewatering.
7. Manufactured assemblies shall be used and installed in accordance with the manufacturer's recommendation and certification shall be furnished by the Contractor.

IV. STRUCTURAL REVIEW

1. Steel - refer to current AISC specifications. If grade of steel is unknown use A36 (F_y=36 ksi, E=30x10^6) or A36M (F_y = 250 MPa, E=20.7 x 10^4 MPa)
2. Welding - refer to current AWS design requirements.
3. Aluminum - refer to current aluminum design requirements.
4. Concrete - refer to current structural concrete design requirements.

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[1] Standard Specifications Section 5-l.O2A, Trench Excavation Safety Plans
[2] Standard Specifications Section 19-3.03, Cofferdams
[3] Refer to Trenching and Shoring Manual Memo 3
[4] Refer to Trenching and Shoring Manual Section 12

5. Timber
   - Construction Safety Orders define lumber and allowable stresses in Appendix C to section 1541.1
   - shoring plans designed by a qualified Engineer which do not specify stress limitations or list type of lumber may be assumed to be Douglas Fir Larch (North) Group II with the following stress limitations:
     - $F_c = 480{,}000/(L/D)^2$ psi ($3310/(L/d)^2$ MPa
     - $F_b = 1{,}500$ psi (10.3 MPa)
     - $F_t = 1{,}200$ psi (8.3 MPa)
     - $F_\perp = 450$ psi (3.1 MPa)

6. Short term increase to allowable stresses are allowed (to maximum of 133%) except in the following situations:
   a. excavations in place more than 90 days
   b. when dynamic loadings are present(pile driving, traffic, etc)
   c. when excavations are adjacent to railroads
   d. all horizontal struts

V . EXCAVATIONS ADJACENT TO TRAFFIC AND EXISTING STRUCTURES

1. Review the effect of possible settlements in the roadway. Require a system to monitor changes in roadway grades if this is a concern.
2. Require temporary railing to be anchored if within 2' (0.6 m) of the excavation.
3. Additional protection provided adjacent to pedestrian walkways in accordance with Cal OSHA requirements.
4. Shoring systems adjacent to roadway should not allow material loss from behind the shoring.
5. Consider using K_o rather than K_a.
6. Ensure shoring system is ridid enough&prevent movement of the shoring. Use of prestressed tiebacks or struts jacked into place.

VI. RAILROAD REVIEW REQUIREMENTS

1. Refer to Chapter 7, Appendix C and Memo 3 of the Trenching & Shoring Manual.
2. Project specific information may be found in Sections 10 & 13 of the Special Provisions.
3. Shoring plans adjacent to railroads are to be approved by the Railroad involved prior to approval by the Structure Representative.

4. After structure representative's review, drawings and calculations should be forwarded for review to Sacramento. The following information should be included:
   - Name of railroad company and contract number
   - Bridge name and number, county, route, and post mile
   - Distance from centerline of track to face of excavation.
5. Contact the Railroad Agreements Section for project specific problems.

VII. APPROVAL PROCESS

1. Engineered System - "Plan Approval" stamp on each sheet, signature and P.E number of structure representative or. staff member who has reviewed the design and is a Registered California Civil Engineer.
2. Proprietary System - "Plan Approval" stamp (blockout P.E. number line) on each sheet, signed by structure representative.
3. In accordance with Cal OSHA Standards - "Plan Approval" stamp (blockout P.E. number line) on each sheet, signed by structure representative.
4. Railroad Company involved - same as above except no approval will be given by structure representative until notified by Sacramento Office Of Structure Construction Shoring Engineer that the plan is satisfactory to the railroad company involved.
5. Contractor should be notified by phone when plan is approved, followed with an approval letter and approved set of plans.
6. Forward one copy of approved shoring plan and structure representatives calculations to Sacramento Office of Structure Construction.

APPENDIX H

VIII. CONSTRUCTION CONSIDERATIONS (BEFORE INSTALLATION)

1. Request APPROVED copy of shoring plan from RE/Structure Rep or Permit Engineer and verify contractor has a identical set of APPROVED plans.
2. Request notification of the contractor's competent person on job site.
3. Verify contractor has requested, and the utility mark out has been completed. Regional notification centers include but are not limited to the following:
   - Underground Service Alert
     - Northern California (USA) l-800-642-2444
     - Southern California (USA) l-800-422-4133
   - South Shore Utility
     - Coordinating Council (DIGS) l-800-541-3447
   - -- Western Utilities Underground Alert, Inc l-800-424-3447
4. Overhead utilities should be signed in accordance with Cal OSHA requirements.
5. If utilities are to be relocated, verify that they are outside the limits of the contractor's proposed work area.
6. Photograph the area to be excavated.
7. Verify required work can be completed with approved shoring plan and will not affect adjacent work taking place.
8. Verify Contractor has obtained a Cal-OSHA Excavation Permit.
9. Verify that the materials required by the design are present on site.
10. Review with the contractor the allowable locations. for spoil piles.
11. Setup settlement monitoring system for excavations adjacent to existing facilities.

IX. CONSTRUCTION CONSIDERATIONS (DURING INSTALLATION AND EXCAVATION)

1. If soil parameters are assumed they should be verified by the Contractor's competent person or Engineer of Record ASAP.
2. Monitor water table and verify with assumed location for design.
3. Verify contractor is following the approved construction sequence.
4. Verify size and length of members prior to installation.
5. Verify grade of materials similar to or better than that used for design.
6. Review drainage around excavation to avoid wet condition.

7. Review adequacy of workmanship and verify that it is in accordance with the approved plans.
8. Verify safety rails and ladders are-in accordance with Cal-OSHA requirements.
9. If welds were designed using AWS allowable stresses, verify welders have current AWS certification.
10. Notify railroad and request flag person when applicable.
11. Continue settlement monitoring when excavation is adjacent to existing facilities.
12. If dewatering is required, verify that the dewatering system is in compliance with SWPPP (Storm Water Pollution Prevention Plan) requirements and discharge is less than 100,000 gals/day (378.5 x 10^3 L/day or 378.5 m^3/day).
13. Verify that dewatering is not causing undue settlement of surrounding facilities.

X. CONSTRUCTION CONSIDERATIONS (SHORING IN PLACE)

1. Daily review required by contractors competent person prior to workers entering an excavation to verify* shoring system is continuing to perform as designed and that all Cal-OSHA requirements are being followed.
2. Continue to monitor deflection of shoring and ground settlement adjacentto existing facilities (roadways, buildings, etc.) Settlement beyond that expected could be an indication of 'heave' or system failure.
3. Backfill behind beams and lagging if voids develop.
4. Verify that handrails and ladders are maintained in accordance with Cal-OSHA requirements[5].
5. Record time excavation is open if short term stress increases (SO days or less) were allowed.

XI. CONSTRUCTION CONSIDERATIONS (SHORING - AND BACKFILL)

1. Verify that the contractor is following the approved shoring removal sequence.
2. Request written notification from contractor to leave shoring in place.
3. Consider future affects of leaving temporary shoring in place.
4. For excavations within other's R/W, confirm with the property owner that the shoring material may be left in place.

---

[5] The Cal-OSHA Construction Safety Orders should be consulted. A partial list of applicable sections are; 1541, 1621, 1629, 1675, 1676, 1677 and 1678.

5. If timber lagging is allowed to be left in place, confirm that the lagging has been pressure treated.
6. Verify that all material is removed a minimum of 2' (610 mm) below finished grade.
7. Backfill all excavations in accordance with Section 19 of the Standard Specifications.
8. Show all temporary shoring material left in place on as-builts.
10. Record final settlement and deflection when excavation is adjacent to existing facilities and record in structure representative's daily diary.

XII. PROJECT SPECIFIC REQUIREMENTS

1.
2.
3.
4.
5.

APPENDIX H

SOUTHERN PACIFIC TRANSPORTATION COMPANY
GUIDELINES FOR DESIGN OF SHORING
IN CONNECTION WITH HIGHWAY
GRADE SEPARATION CONSTRUCTION

GENERAL

Temporary sheeting and shoring for support of adjacent track or tracks during construction will not be closer than 8'-6" from the center line of the track. Excavations will not be allowed closer than 8' -6" from the centerline of the track unless specifically approved by the Office of the Bridge Engineer of the Southern Pacific Transportation Company.

SHORING DESIGN AND DRAWINGS

All drawings and calculations for shoring shall be prepared and signed by a Registered Civil Engineer. The Engineer will be responsible for the accuracy of all controlling dimensions as well as the selection of soil design. values which will accurately reflect the actual field conditions. No excavation adjacent to Southern Pacific Lines will be allowed until the drawings and calculations are reviewed and approved by the Office of the Bridge Engineer for the Southern Pacific Transportation Company.

The drawings shall contain detailsof the shoring system showing sizes of all structural members, details of connections, and embedment depth. The drawings must be complete and accurately describe the nature of the work. The drawings shall include the following:

- plan view showing all of the proposed excavations and distances from the centerline of the track to the face of the excavation at each location.

- section view normal to the track showing the shoring location relative to the centerline of the track and the height of sheeting and track elevation in relation to the bottom of the excavation.

A minimum of 30 days should be allowed for the Railroad's review of such drawings provided that all material are in order. No excavation will be allowed until the drawings and calculations are reviewed and approved by the Office of the Bridge Engineer for the Southern Pacific Transportation Company.

SHORING SYSTEMS AND LIMITS OF EXCAVATION

Shoring located between 8' -6" and 10'-0" from the centerline of the track, when excavation is in natural ground or fill ground which has been placed with proof of adequate compaction control, also shoring between 8'-6" and 13'00" when excavation is on fill ground other than compaction controlled as stated above, shall be of a type where the shoring is installed in place prior to any excavation being performed, and where the excavation can be made with no possibility of disturbance or loss of soil material retained between the shoring and the track. Common shoring types fulfilling this requirement are interlocking edge steel sheet piling, tongue and groove edge precast concrete sheet piling, etc., which are driven into position prior to starting any excavation. Shoring types using lagging elements which are placed as excavation proceeds are not permitted within the limits specified in this paragraph.

Shoring outside the limits stated in the previous paragraph may be of other types such as soldier piles and lagging elements which are installed as the excavation proceeds.

Excavation pits, etc., within 13' -0" from the centerline of track shall have handrails installed. Minimum clearance from centerline-of track to the face of handrails will be 8'-6" on tangent track, and 9'-6" for track on a curve.

SOIL CLASSIFICATION

Soils to be retained as well as the soils depended upon for structural stability (passive resistance, shear strength, friction angle, etc.) shall be classified in accordance with he soil types listed in AREA Specifications Chapter 8, Part 5. This classification is to be part of the calculations. and shall be stated on the submitted plans and be verified by a Registered Civil or Geotechnical Engineer, This info&nation is in Appendix C of the Caltrans Trenching and Shoring Manual.

Where the provisions of these guidelines are more restrictive than the requirements of the Public Utilities Commission Orders, Department of Industrial Safety, OSHA, or other governmental agencies, then the above guidelines shall apply for shoring adjacentto railroad tracks of the Southern Pacific Transportation Company.

LIVE LOAD SURCHARGE

All excavations within the limits detailed in the sheet titled Shoring Requirements drawing SP5, shall be designed for Railroad live load surcharge.

All shoring designed for Railroad surcharge shall be based on Cooper E-80 live load. AREA Chapter 8, Part 20 Section C, Paragraph 2(b), refers to the Boussinesq equation as a method to determine lateral pressure values for Railroad surcharge loading. The use of the AREA Boussinesq equation is not the only method available to obtain lateral pressures. Pressures significantly less than those determined by the Boussinesq equation do not adequately consider Railroad live load surcharge. i.e. The Boussinesq equation will be used to determine the minimum lateral pressure due to Cooper E-80 live load for shoring adjacent to tracks belonging to the Southern Pacific Transportation Company. The Boussinesq equation from the above referenced section is shown in the sheet titled Shoring requirements SP15.

A minimum equipment live load surcharge to be applied is an equivalent height of soil two feet high with unit weight of 110 pounds per cubic foot.

EARTH PRESSURES

For level backfill the minimum equivalent fluid pressure of 36 pounds per cubic foot shall be used in designing the shoring. This corresponds to Type 2 soil as defined in AREA Chapter 8 Part 5, Table 5.2.5 and 5.3.2. For any other type of soils as defined in AREA Chapter 8 Part 5, Table 5.2.5 and 5.3.2 the designer of the shoring system will calculate and use the corresponding equivalent fluid pressure.

Where the internal friction angle (ϕ), or the cohesion of the soils have been ascertained by boring and tests and the values for equivalent fluid pressure have been established by a Registered Professional Engineer specializing in geotechnical engineering, then these values may be used in lieu of the tabulated values providing the ϕ and C values determined by test have been reduced by 15% to allow for the dynamic effect of train loadings on the retained materials.

The minimum values for retained soils shall be those stated for Type 2 soil, namely, unit weight of soil = 110 pcf, angle of internal friction ϕ = 30°, and cohesion = 0.

When shoring sloping soils, the sloping soil surcharge shall be computed per AREA Chapter 8, Part 5, Appendix C for the corresponding soil. This information is in Appendix C of the Caltrans Trenching and Shoring Manual.

MISCELLANEOUS

Walkways and railings shall be constructed around open excavations adjacent to the operating tracks when shoring is within 13'-0" from the centerline of the tracks. Railings shall not be closer than 8'-6" horizontally from centerline of the track.

Approval of the excavation plan does not relieve the designer and/or contractor of the ultimate responsibility and liability for the excavation plan.

The design of shoring using cantilevered sheet pile walls, or cantilevered soldier pile systems, also sheet pile or soldier pile systems using tie backs or raker struts in which the tiebacks or struts are not preloaded are considered as flexible shoring systems. Excavations which are cross-strutted to the opposite side (trench type), and the tieback or raker strut is preloaded are considered as rigid shoring systems and should be treated differently from flexible systems.

All retaining structures shall be investigated and be safe against slip circle type failure.

DESIGN STRESSES

No increase in allowable stresses is allowed.

The maximum allowable design stresses listed are based on the use of undamaged high-quality materials. Stresses and loadings shall be reduced by the Engineer if lesser quality materials are to be used.

TIMBER

The species and grade of timber or lumber used shall be shown on the drawings.

Compression perpendicular to the grain	450 psi
Compression parallel to the grain but not to exceed 1,600 psi	$480{,}000/\ (L/d)^2$ psi

Flexural stress	1,500 psi
Horizontal shear	140 psi

In the foregoing formulas, L is the unsupported length; d is the least dimension of a square or rectangular column, or the width of a square of equivalent cross-sectional area for round columns.

STEEL

For identified grades of steel, design stresses, except stresses due to flexural compression, shall not exceed those specified in the Manual of Steel Construction as published by the American Institute of Steel Construction (AISC).

When the grade of steel cannot be positively identified, design stresses, except stresses due to flexural compression, shall not exceed either those specified in said AISC Manual for ASTM Designation: A-36 steel or the following:

Tension, axial and flexural	24,000 psi
Compression, axial (when L/r is-less than 120)	
for used steel	$16{,}000 - 0.38\,(L/r)^2$ psi
for new steel	$20{,}000 - 0.40(L/r)^2$ psi
Shear on gross section of web	14,500 psi
Web crippling for rolled shapes	27,000 psi

For all grades of steel, design stresses and deflections. shall not exceed the following:

Compression, flexural (14,400,000)/(Ld/bt) psi but not to exceed 24,000 psi for unidentified steel or steel conforming to ASTM Designation: A-36 nor $0.6F_y$ for other identified steel.

In the foregoing formulas, L is the unsupported length; d is the least dimension of rectangular columns, or the width of a square of equivalent cross-sectional area for round columns, or the depth of beams; b is the width and t is the thickness of the compression flange; r is the radius of gyration of the member. All dimensions are expressed in inches F_y is the specified minimumyield stress in psi, for the grade of steel used.

Sheet Pile sections =
2/3 tensile yield for steel
1/3 compressive strength for concrete in compression. No tension allowed.

ANCHOR RODS

Tension, axial and flexural	24,000 psi
Compression, axial (when L/r is less than 120)	
for used steel	$16{,}000 - 0.38(L/r)^2$ psi
for new steel	$20{,}000 - 0.40(L/r)^2$ psi

PRESTRESS STRAND OR ROD

Allowable working stress	
(Other than tieback)	0.6(Ultimate Strength)
(Used as tieback)	0.4(Ultimate Strength)

If strand or rod is used as a structural element and will be in service for a long period then the structural member must be protected from corrosion. Acceptable protection against corrosion is grease or PVC pipe.

STEEL WIRE CABLE

$$\text{Allowable working load in Lbs} = \frac{\text{Rated Breaking Strength}}{2.5}$$

If wire cable is used as a structural element and Will be in service for a long period then the structural member must be protected from corrosion.

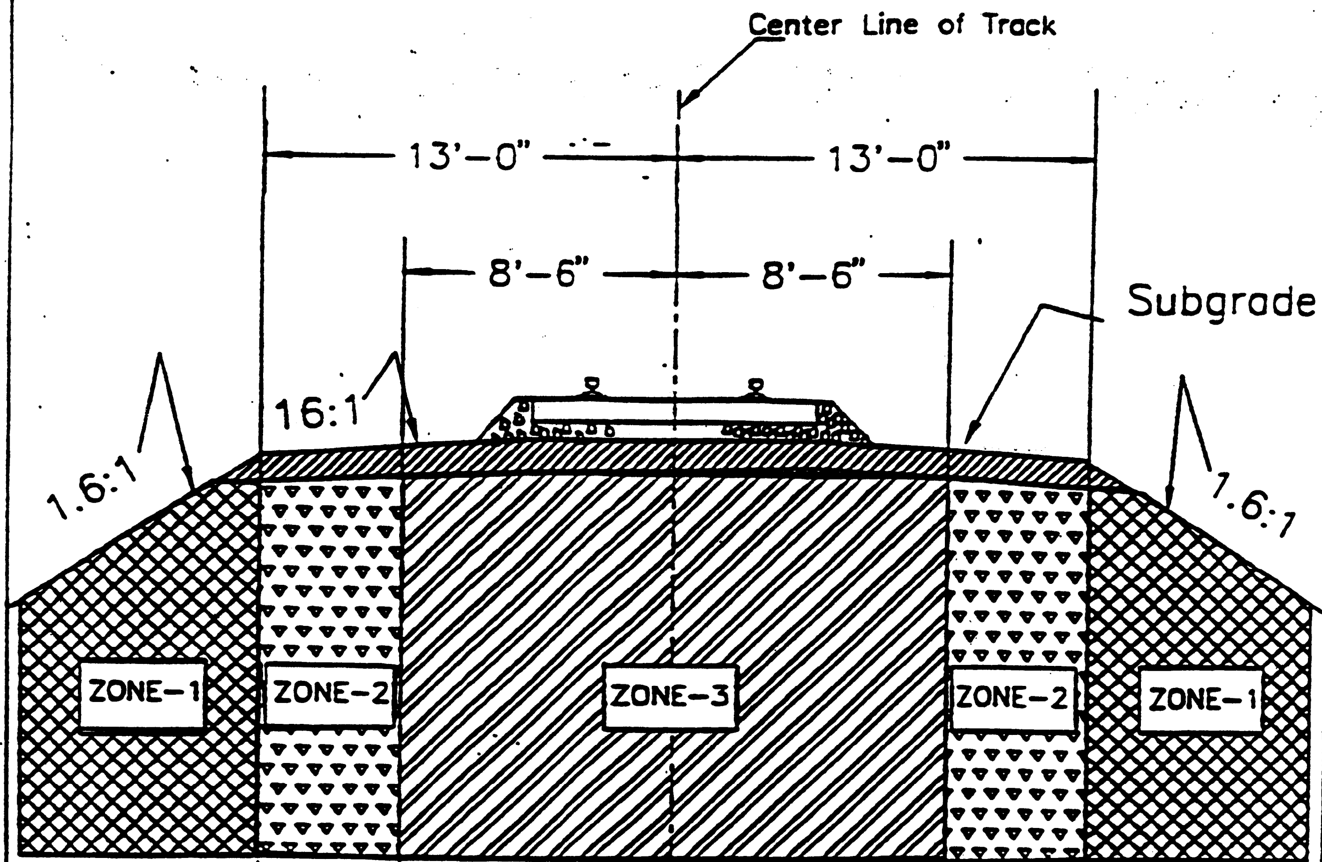
CONCRETE

Allowable concrete stresses shall comply with AREA Chapter 8, Part 2.

FACTORS OF SAFETY

For anchor blocks, deadman, etc.	2.0
In the use of passive pressure for stability	2.0
In the use of soil shear strength and friction based on vertical loads	1.5
Slip circle failure of structure as a whole, or any part except anchor blocks, deadman, etc	1.5
Slip circle failure of anchor bolts, deadman, etc.	1.5
Soil bearing pressures	U.B.C. Section 29.

NO INCREASE IN STRESS OR REDUCTION OF SAEETY FACTORS IS ALLOWED.



ROADBED PROFILE

SHORING REQUIREMENTS

ZONE-1 Excavation within ZONE-1 will require shoring for protection of the Railroad.

ZONE-2 Excavation within ZONE-2 will require shoring consisting of interlocking sheeting for protection of the Railroad.

ZONE-3 NO EXCAVATION WILL BE ALLOWED IN ZONE-3

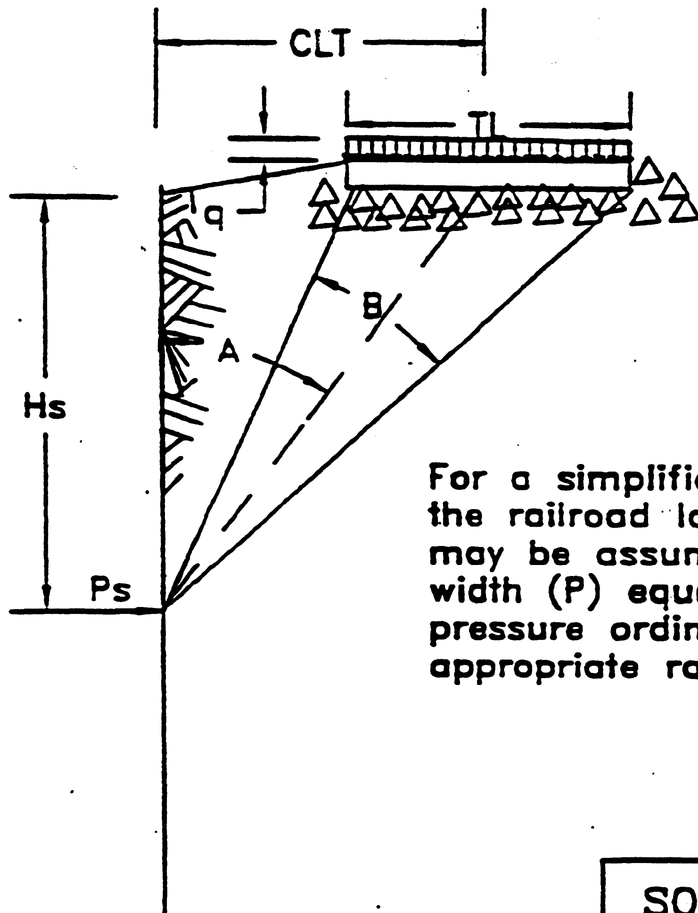
NOTE:

Excavations outside ZONE-1 may require shoring for safety. Lateral pressures due to train loadings do not affect shoring design outside of ZONE-1.

N.G.P.

SOUTHERN PACIFIC LINES
ROADBED PROFILE
SHORING REQUIREMENTS
OFFICE OF CHIEF ENGINEER
DATE 10-25-93
DRAWING: ACAD\SP5

$P_s = (2q/\pi)\,((B+\sin B)\sin^2 A + (B-\sin B)\cos^2 A)$
$B = \tan^{-1}((CLT + TL/2)/H_s) - \tan^{-1}((CLT - TL/2)/H_s)$ in radians
$A = B/2 + \tan^{-1}((CLT - TL/2)/H_s)$ in radians
q = uniform surcharge load from Cooper E-80 load = 80,000 Lbs/(5*TL)
CLT = Distance from face of shoring to centerline of track
TL = tie length = 9.0 ft standard
H_s = Height from bottom of tie to any point in the face of the shoring



For a simplified engineering analysis, the railroad loading surcharge pressure may be assumed rectangular with width (P) equal to 0.8 of the maximum pressure ordinate as given by the appropriate railroad curve.

ALL PRESSURES SHOWN IN CHARTS ARE BASED ON BOUSSINESQ EQUATION FOR STRIP LOADS SHOWN IN A.R.E.A SPECIFICATIONS CHAPTER 8, SECTION 20 C, PART 2b

N.G.P.

SOUTHERN PACIFIC LINES
BOUSSINESQ EQUATION
RAILROAD SURCHARGE
OFFICE OF CHIEF ENGINEER
DATE 10-25-93
DRAWING: ACAD\SP15

MISCELLANEOUS TOPICS COVERED AT THE 1996 WINTER TRAINING MEETINGS

This memorandum will summarize some of the pertinent questions that were raised during the instruction of the 1996 Trenching and Shoring Class. It seemed appropriate that this information be included in the Trenching and Shoring Manual to ensure uniformity and to enhance what was presented at the training sessions.

1. Is the use of the ‘flagpole' method acceptable?

Generally the flagpole method refers to an analysis procedure shown in the Uniform Building Code; Section 1806.7.2.1 ('94 UBC), Section 2907 (‘91 UBC).

Discussion with ICBO (International Conference of Building Officials) the publishers of the Uniform Building Code revealed that this method was incorporated into their code at the request of the outdoor advertising industry. The official implied that it would not prudent to use. this method as an analysis tool for excavation type work. It is important that if the UBC method is chosen, that it be used consistently with the tables published with that method.

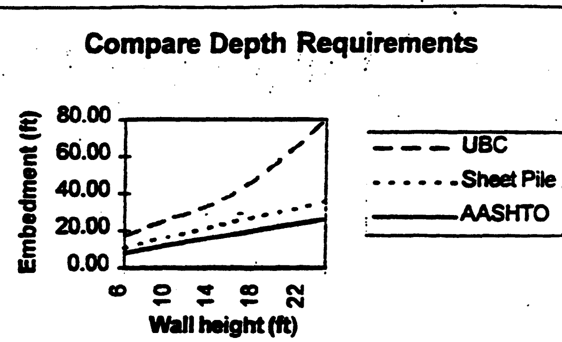
The following chart shows a comparison of unfactored embedment depth between three methods of analysis for a soldier pile wall for both a 72 psf and 100 psf surcharge load. The three methods represented here are the following:

- Uniform Building Code, Section 1806.7.2.1 (‘94 UBC), Section 2907 ('91 UBC).
- AASHTO method of analysis for temporary flexible cantilevered walls with discrete vertical wall elements.
- Sheet pile analysis for soldierpile walls.

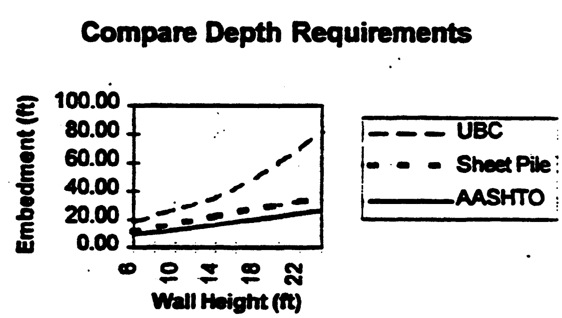
The soil properties for this example are as follows:

H = 8 '
γ = 120 pcf
ϕ = 30°
b = 1' round pile.

Surcharge = 72 psf			
H	AASHTO	UBC	Sheet Pile
6	7.95	16.70	10.52
8	10.08	21.30	13.43
10	12.20	25.80	16.32
12	14.12	29.80	18.95
14	16.06	33.80	21.61
16	18.03	39.80	24.30
18	20.02	48.00	27.02
20	22.02	58.00	29.75
22	24.04	68.60	32.51
24	26.07	80.00	35.27



Surcharge = 100 psf			
H	AASHTO	UBC	Sheet Pile
6	9.58	17.90	11.17
8	11.11	22.50	14.12
10	12.79	27.00	17.10
12	14.66	30.90	19.60
14	16.56	34.90	22.21
16	18.49	42.00	24.85
18	20.44	50.00	27.53
20	22.41	60.00	30.22
22	24.40	70.00	32.94
24	26.41	82.00	35.68



As can be seen-from the charts, the UBC method appears very conservative. If a designer chooses to use pressures other than those from the charts listed within the code, then the accuracy of using this method diminishes.

2. May an existing footing be used to increase the passive pressure?

Existing footings may be used to increase the passive resistance on the embedment depth of soldier or sheet piles. To determine the amount of aid it may offer, several methods can be used to determine the amount of lateral pressure the footing applies to piles. Two of these methods are:

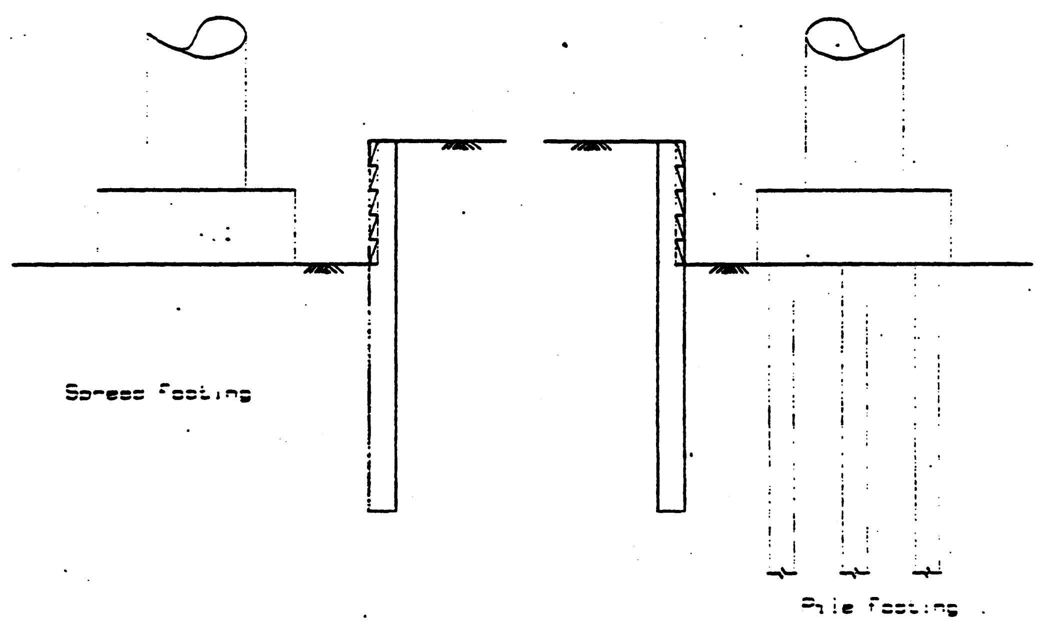
1) Boussinesq equation
2) $(\gamma + q)K_pD$

Where :

Y = Unit weight of the soil
q = Uniform surcharge pressure
D = Depth of embedment
K_p = Coefficient of passive pressure for the soil.

Because there is often a gap of four or more feet between the shoring system and the footing for most footing retrofits, the Boussinesq equation recommended for determining the amount of pressure acting against the embedded depth of the pile.

If the footing is tight against the shoring system, adding the surcharge to the unit weight of the soil and multiplying by the appropriate K_p value as shown above would be an acceptable alternative to using the Boussitiesq equation. Before doing so, ensure that the passive wedge acts within the width of the footing.



This methodology is applicable for both spread and pile footings. If the permanent piles are to be utilized as part of the retention system, then a rigorous analysis should be submitted verifying the-resisting capabilities of the piles.

When strutting against an existing footing, the permanent structure piles should not be subjected to a load greater than 13 kips/pile.

3. What is the effect of reducing K_p in lieu of increasing embedment depth (D) by 20 - 40%.

As discussed on page 8-3 of the Trenching and Shoring Manual, passive resistance should be initially reduced by dividing K_p by 1.5 to l.75; or alternatively increase the computed D (depth of embedment) by 20% to 40% The preferred method is to adjust K_p, but either approach is acceptable. Using an adjustment of Kp agrees with what Bowles says in his book Foundation Analysis and Design.

The following results reflect what occurs when both methods are applied to the problems in the handout from the '96 Trenching and Shoring class:

1. Sheet pile problem

   Original

   $K_p = 3.0$
   $D = 15.8'$ $D*1.3 = 20.5'$
   $S_{req} = 23.4$ in^3

   Reduce K_p

   $K_p = 3.0/1.5 = 2.0$
   $D = 21.3'$
   $S_{req} = 31.8$ in^3

2. Soldier pile problem

   Original (Sheet pile analysis)

   $K_p = 3.0$
   $D = 14.9'$ $D*1.2 = 17.8'$
   $S_{req} = 135$ in^3

Reduce K_p (Sheet pile analysis)

K_p = 3.0/1.5 = 2.0
D = 19.8'
S_{req} = 186 in^3

3. Soldier pile problem

Original (AASHTO analysis)

K_p = 3.0
D = 13.1' D*1.2 = 16.0'
S_{req} = 122 in^3

Reduce K_p (Sheet pile analysis)
K_p = 3.0/1.5 = 2.0
D = 16.0
S_{req} = 141 in^3

4. Are photocopies of the PE's signature and stamp acceptable on copies of plans or must the PE stamp and sign each photocopy submitted?

According to the Board of Registration, photocopies of original sealed/signed plans would be satisfactory in meeting the intent of the business and professional code. The engineer of record is responsible for changes that have been made to the plans provided that the engineer of record is aware of the changes made.

5. What does Cal-OSHA require for minimum surcharge loads and to what depth do we carry the surcharge load?

Cal-OSHA does not have a minimum surcharge load that needs to be carried for engineered systems. The Trenching and Shoring Manual states that you should use a minimum surcharge load of 72 psf (3510 N/m^2) This was derived from the Tables in Appendix C to Section 1541.1 of the Construction Safety Orders which includes a 2 ft (610 mm) height of soil. This 2 ft (610 mm) height of soil equates to a 72 psf (3510 N/m^2 load against the shoring when the following soil parameters are used:

ϕ = 30°, γ = 110 pcf (17 280 N/m^3)

The minimum surcharge (72 psf, 3510 N/m^2) reflects miscellaneous loads that may be adjacent to a shoring system that may not have been taken into account by the designer of the shoring system. Miscellaneous loads include such items as portable generators, small pickup trucks, workers etc. Loads of any substantial magnitude should be reviewed individually and not be assumed to be included in the minimum surcharge load.

It will be Division of Structures policy that the minimum surcharge load will be used with all shoring systems except when a surcharge from another source causes a lateral pressure of greater magnitude to be used. The lateral pressure from the minimum surcharge load will be carried to the bottom of the excavation or 10 ft (3.05 m), whichever is less.

6. How should surcharge loads be applied to shoring systems?

For the minimum surcharge load (72 psf, 3510 N/m^2), or alternate traffic surcharge (100 psf, 4790 N/m^2) carry the load to the bottom of the excavation or 10 ft (3.05 m) whichever is less. For building or other surcharges, carry the pressures developed from these surcharge loads to a depth where the pressure exerted by the surcharge is 100 psf (4790 N/m^2) or less. At this point the surcharge may be discontinued provided it is below the bottom of the excavation.

Surcharges due to Railroad loadings (Cooper E-80) will be carried to the bottom of the shoring system. In the case of sheet and soldier pile systems, the pressures developed from the railroad surcharge is applied from the top of the shoring system to the tip of the pile.

7. Will railroads allow tieback anchors under their tracks to remain in place after excavation is complete or will the anchor need to be removed?

The Southern Pacific Transportation Company response to this question was that it would be on a case by case basis, Some factors affecting their decision would be the future use of their facility and depth of the anchor. The contractor should contact the railroad company to determine if the anchor may be left in place. The date/time and the person spoken to should all be included in the submitted plan.

8. Railroad approval

Railroad approval may take the form of no exceptions taken, rejected, or exceptions taken.

In the case of "rejection", the contractor will need to correct and resubmit the plans to the Structure Representative for approval. A satisfactory resubmittal should be forwarded to the falsework engineer for Railroad approval following the same procedures for all shoring and falsework plans involving railroads.

In the case of exceptions taken, the contractor will still need to correct the deficiencies and resubmit the shoring plan to the Structure Representative for review. Provided the contractor made the necessary changes to the plan as requested by the railroad, then resubmitting the plan to the railroad will not be necessary.

9 . Grades on adjacent railroad tracks

When shoring is adjacent to railroad tracks it is important to monitor track settlement during all stages of shoring construction Grades should be established on the tie plates since they have a tendency to move or settle with the tie. Grades on the rail itself may be erroneous due to the rails ability to bridge across some of the low ties in an unloaded condition. Choose tie plates that do not move when trains cross over them.

SHORING ADJACENT TO UNION PACIFIC RAILROAD COMPANY TRACKS

Railroad Guidelines

To expedite the review process of shoring plans by the Union Pacific Railroad Company (UPRR), the drawings submitted by the contractor must adhere to the requirements of the UPRR. Until the UPRR issues new guidelines, the design of shoring systems for all UPRR and former Southern Pacific Transportation Company (SPTC) lines shall be in accordance with the SPTC shoring guidelines titled *GUIDELINES FOR DESIGN OF SHORING IN CONNECTION WITH HIGHWAY GRADE SEPARATION CONSTRUCTION*, with the exceptions noted herein. Refer to Trenching and Shoring Memo 3 (04/96) for a copy of these guidelines.

As always, the UPRR should be contacted to obtain the latest copy of their guidelines. The address and telephone number of the Manager of Public Projects will be listed in Section 13 of the contract Special Provisions.

The general shoring requirements of the UPRR have been revised from the SPTC guidelines shown on Trenching and Shoring Memo page H-3-8 (SPTC Drawing ACAD\SP5, "Southern Pacific Lines Roadbed Profile Shoring Requirements", dated October 25, 1993). The minimum construction clearance and loading requirements for shoring adjacent to the UPRR tracks are shown on UPRR sheet C.E. 106613,"General Shoring Requirements", dated March 31, 1998. Refer to Attachment No. 1 for a copy of this drawing.

The contract special provisions will list the clearance requirements measured from the centerline of the railroad tracks. If clearances are not included in the contract documents, refer to UPRR Std. Dwg. 0035, "Barriers and Clearances to be Provided at Highway, Street, and Pedestrian Overpasses", dated March 31, 1998 for minimum construction clearance requirements. Refer to Attachment No. 2 for a copy of this drawing. This drawing shows the latest UPRR clearance requirements and will be incorporated into future contracts.

Railroad Requirements

The UPRR requires the use of the Boussinesq equation to determine the minimum lateral pressure due to the Cooper E80 live load for shoring adjacent to their tracks for all UPRR and former SPTC lines. **The Cooper E80 railroad surcharge earth pressure curve allowed by the SPTC (shown as Chart 3.6 in Appendix C and other locations in the Trenching and**

Shoring manual) is <u>not</u> allowed by the UPRR. All references to the Chart 3.6 method of analysis should be discarded and the Boussinesq equation used. The application of the railroad surcharge shall be applied to the full depth of the shoring system.

Some common requirements are often overlooked and have resulted in submittals being returned by the railroad. The shoring plans should note how the contractor will gain access to the site, particularly if they must cross the railroad tracks. Track protection details are shown in the UPRR's *GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD*.

The shoring plans should note if there are any existing drainage ditches or access roads being affected by the Contractor's operations related to the shoring system. If there are no existing drainage facilities or access roads, the shoring drawings should note this fact. Keep in mind that personnel from the railroad who are unfamiliar with the site often review the shoring plans.

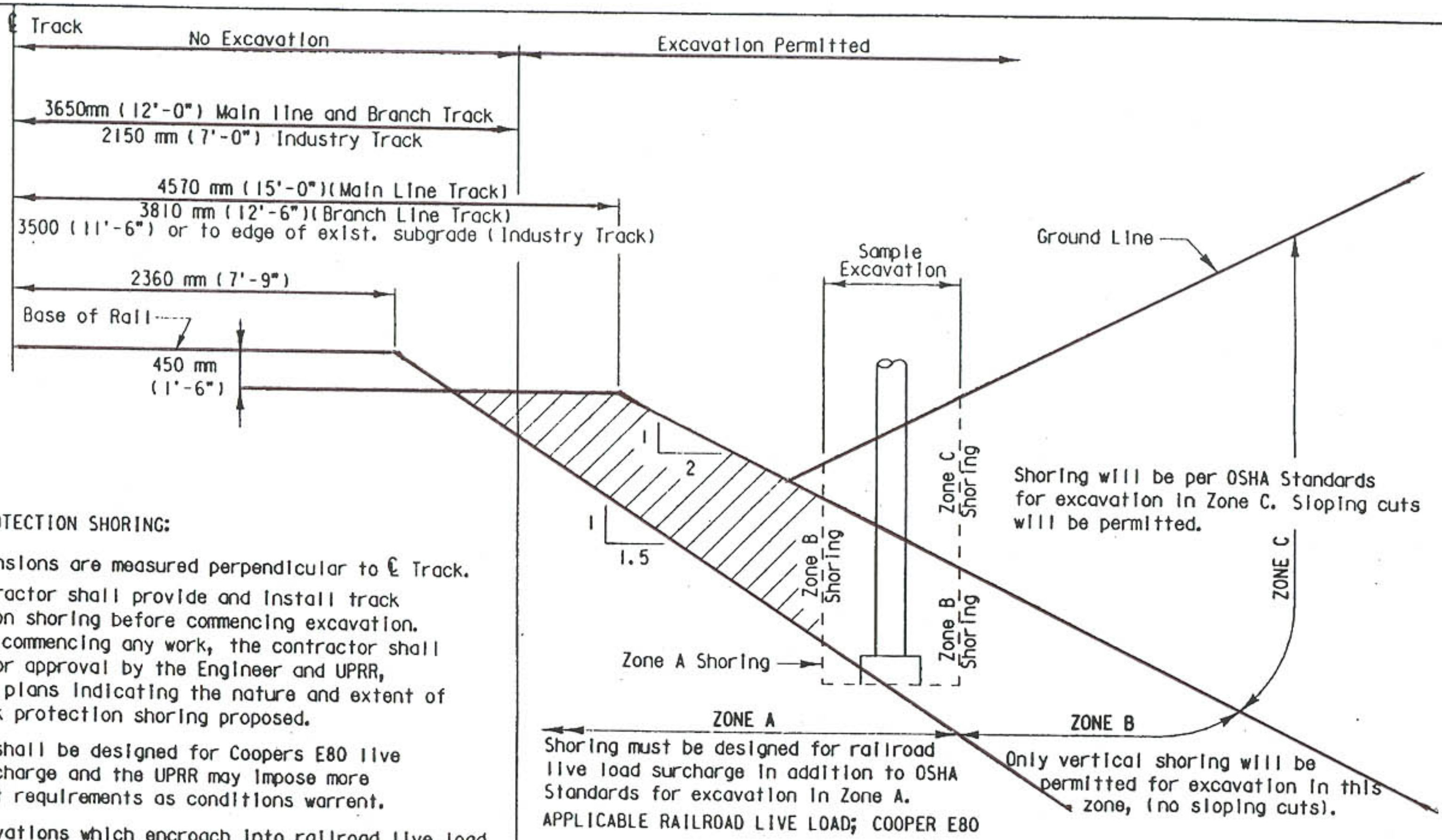
Railroad Review and Approval

The UPRR requires the submittal of a minimum of **five** sets of shoring plans and **three** sets of calculations and manufacturers' literature.

The UPRR has requested that drawings accompanying shoring plans be submitted on 11"x17" (279.4 mm x 431.8 mm) sized paper. Future special provisions will be revised to state this requirement. Until this request becomes a specification requirement, you should encourage that the contractor submit the five sets of shoring plans for railroad review on 11"x17" (279.4 mm x 431.8 mm) sized paper.

The above railroad requirements should be discussed at the pre-construction meeting with the Contractor. Approval of shoring plans adjacent to UPRR tracks will be contingent upon UPRR approving the plans. Note that the Contractor <u>must not begin</u> construction of any component of the shoring system within the railroad right-of-way until such time that railroad approval has been received.

Attachments



TRACK PROTECTION SHORING:

All dimensions are measured perpendicular to ℄ Track.

The contractor shall provide and install track protection shoring before commencing excavation. Prior to commencing any work, the contractor shall submit for approval by the Engineer and UPRR, detailed plans indicating the nature and extent of the track protection shoring proposed.

Shoring shall be designed for Coopers E80 live load surcharge and the UPRR may impose more stringent requirements as conditions warrent.

For excavations which encroach into railroad live load surcharge zone, shoring plans will be accompanied by a copy of the design calculations, and both must be stamped by a registered professional engineer.

Design of shoring shall comply with UPRR guidlines for design and construction of shoring adjacent to active railroad tracks.

TRACK PROTECTION SHORING REQUIREMENTS

UNION PACIFIC RAILROAD

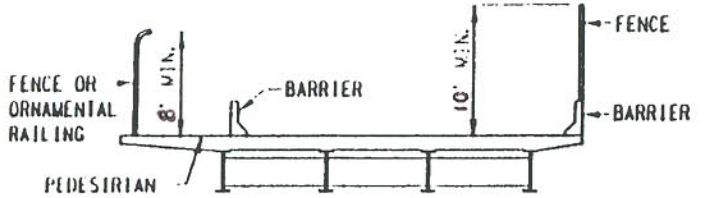
GENERAL SHORING REQUIREMENTS

OFFICE OF CHIEF ENGINEER DESIGN

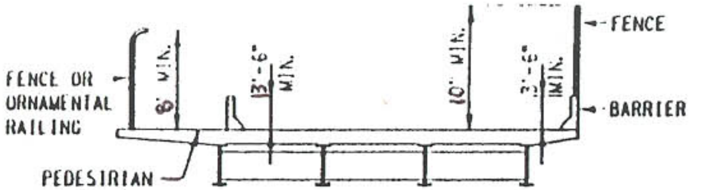
DATE: 3-31-98 REDRAWN

C.E. 106613

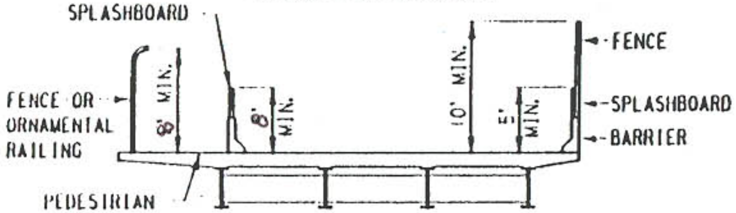
TRENCHING AND SHORING MEMO 5
ATTACHMENT No. 1 (01/00)
Sheet 1 of 1



VIADUCT CROSS SECTION
NO SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH 3'-6" SOLID BARRIER AND FENCE
FOR SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH SPLASH BOARD AND FENCE
FOR SNOW REMOVAL AREAS

GENERAL

Fence shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures.

Splashboards or solid 3'-6" high barrier rail shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures where snow removal is being performed.

Lights are to be installed on the underside of the viaduct where shadows cast by the structure would interfere with Railroad operations.

Slope paving shall be provided where end slopes equal to or exceed 2 horizontal to 1 vertical.

Falsework for costruction of overhead structures shall comply to UPRR guidelines.

Demolition of existing overhead structures shall comply to UPRR guidelines.

Temporary shoring shall be designed in accordance with UPRR's Shoring Requirements (Drawing No. 106613) and UPRR guidelines.

Applicant shall be responsible for identification, location, and protection of existing utilities.

Contact UPRR's "Call Before You Dig" at least 48 hours prior to commencing work at 1-800-336-9193 to determine location of fiber optics.

Exceptions to these standards must be approved by UPRR's Chief Engineer Design.

CLEARANCES

Minimum vertical clearance shall be 23 feet above the plane of top-of-rails. Additional clearance may be required for construction purposes or if sag of vertical curve must be adjusted or if future track raise for flood considerations or maintenance is probable.

Minimum horizontal clearances, measured at right angle from centerline of track, shall be as shown in elevation view.

Minimum construction clearances shall be 21 feet vertical above the plane of top-of-rails and 12 feet horizontal at right angle from centerline of track.

FUTURE TRACKS

Space is to be provided for one or more future tracks as required for long range planning or other operating requirements. Where provision is made for more than two tracks, space is to be provided for access road on both sides of tracks.

PIERS

Pier protection walls shall be provided in accordance with AREA Chapter 8, Part 2.[illegible].5 for piers within 25 feet of the centerline of track.

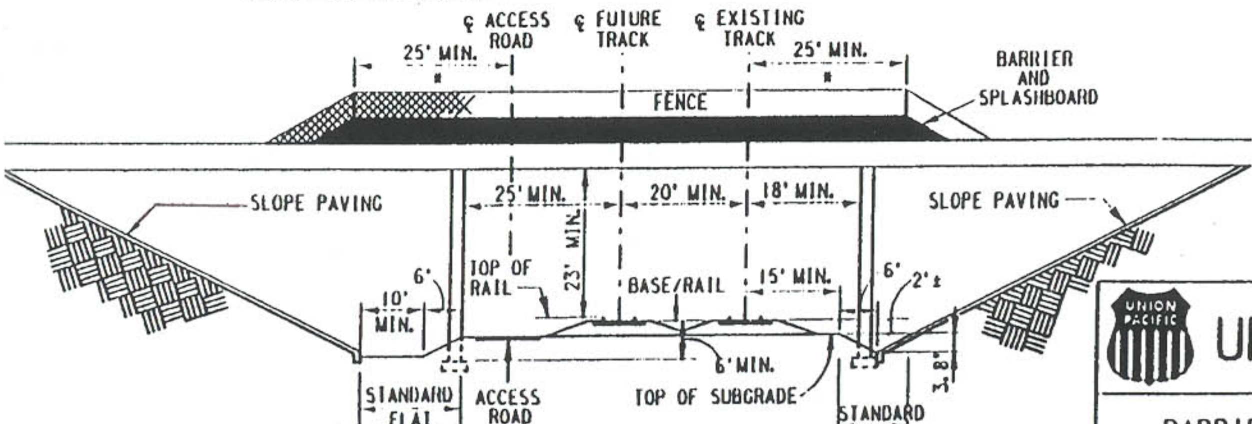
Top of footings within 25' from centerline of track shall be a minimum of 6 feet below base of rail and a minimum of 1 foot below flow line of ditch.

DRAINAGE

Drainage from the overpass shall be diverted away from UPRR's tracks and not discharged onto the tracks or roadbed.

A standard "V"-shaped or flat-bottom ditch shall be provided on each side of the tracks as necessary.

Culverts may be installed on opposite side of column from track in lieu of standard Railroad ditches when approved by Chief Engineer Design. Maintenance of culverts is to be at applicant's expense.



ELEVATION
PERPENDICULAR TO TRACKS

* Fences, splashboards, or solid barriers if required shall extend 25ft. beyond centerline of outer most track or access roadway.

UNION PACIFIC RAILROAD

BARRIERS AND CLEARANCES TO BE PROVIDED AT HIGHWAY, STREET, AND PEDESTRIAN OVERPASSES

OFFICE OF CHIEF ENGINEER DESIGN

REVISED: MAR. 31, 1998 STD. DWG. 0035

TRENCHING AND SHORING MEMO 5
ATTACHMENT No. 2 (01/00)
Sheet 1 of 1