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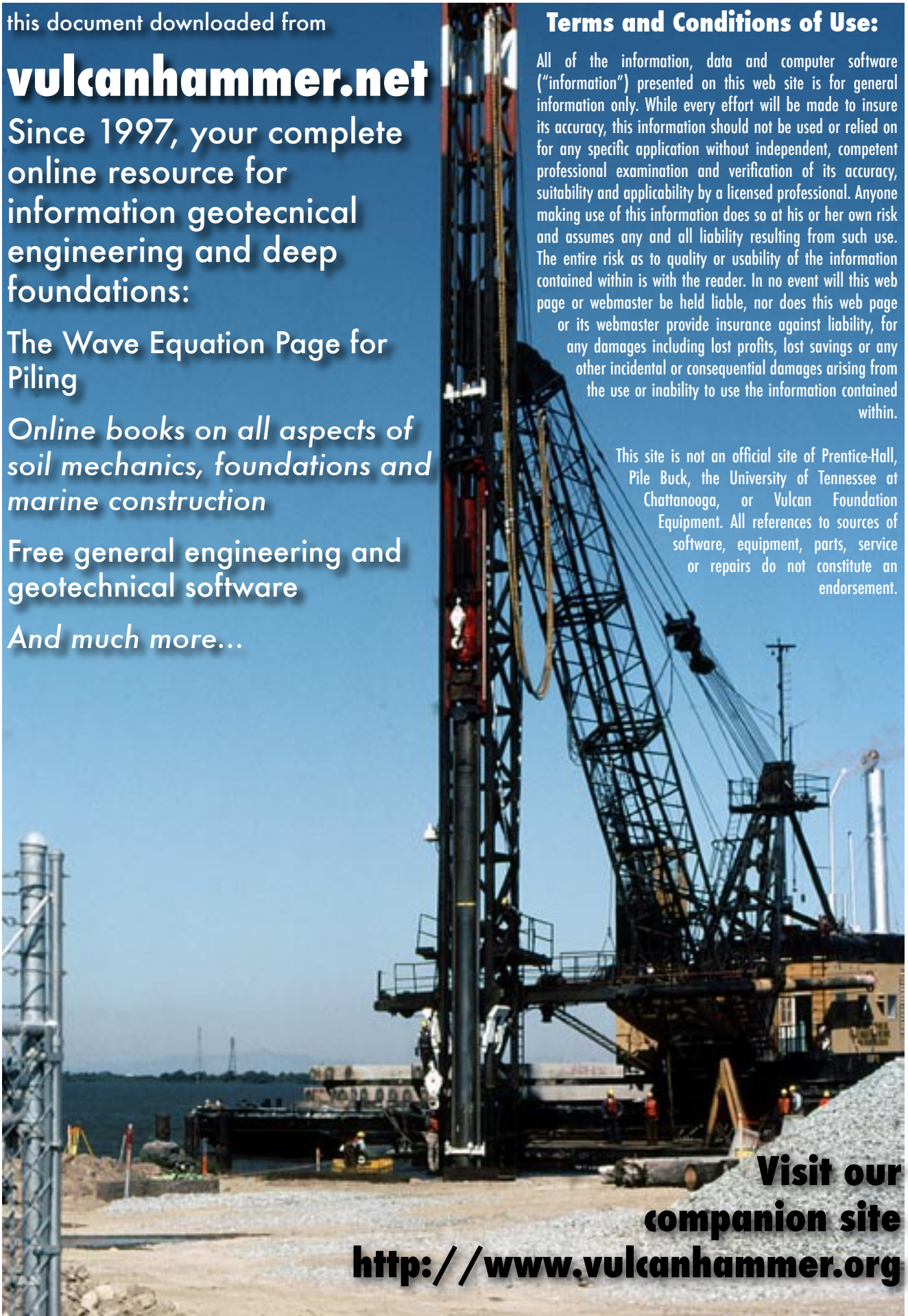
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Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume I

Developed following:

*AASHTO LRFD Bridge Design
Specifications, 4th Edition, 2007,
with 2008 and 2009 Interims.*

and

*AASHTO LRFD Bridge Construction
Specifications, 2nd Edition, 2004, with
2006, 2007, 2008, and 2009 Interims.*



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16. ABSTRACT This manual is the reference text used for the FHWA NHI courses No. 132042 and 132043 on Mechanically Stabilized Earth Walls and Reinforced Soil Slopes and reflects current practice for the design, construction and monitoring of these structures. This manual was prepared to enable the engineer to identify and evaluate potential applications of MSE walls and RSS as an alternative to other construction methods and as a means to solve construction problems. The scope is sufficiently broad to be of value for specifications specialists, construction and contracting personnel responsible for construction inspection, development of material specifications and contracting methods. With the aid of this text, the engineer should be able to properly select, design, specify, monitor and contract for the construction of MSE walls and RSS embankments. The MSE wall design within this manual is based upon Load and Resistance Factor Design (LRFD) procedures. This manual is a revision (to LRFD) and an update to the FHWA NHI-00-043 manual (which was based upon allowable stress design (ASD) procedures).			
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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m ³	cubic meters	35.71	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
TEMPERATURE				
°C	Celsius	1.8 C + 32	Fahrenheit	°F
WEIGHT DENSITY				
kN/m ³	kilonewton / cubic m	6.36	poundforce / cubic foot	pcf
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / sq in.	psi
kPa	kilopascals	20.9	poundforce / sq ft	psf

PREFACE

Engineers and specialty material suppliers have been designing reinforced soil structures for the past 35 years. Currently, many state DOTs are transitioning their design of substructures from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD) procedures.

This manual is based upon LRFD for MSE wall structures. It has been updated from the 2001 FHWA NHI-00-043 manual. In addition to revision of the wall design to LRFD procedures, expanded discussion on wall detailing and general updates throughout the manual are provided. The primary purpose of this manual is to support educational programs conducted by FHWA for transportation agencies.

A second purpose of equal importance is to serve as the FHWA standard reference for highway projects involving MSE wall and reinforced soil structures.

This Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS), Design and Construction Guidelines Manual which is an update of the current FHWA NHI-00-043, has evolved from the following AASHTO and FHWA references:

- AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2008 and 2009 Interim Revisions.
- *Earth Retaining Structures*, by B.F. Tanyu, P.J. Sabatini, and R.R. Berg, FHWA-NHI-07-071 (2008).
- AASHTO LRFD Bridge Construction Specifications, 2nd Edition, 2004, with 2006 Interim Revisions.
- *Geosynthetic Design and Construction Guidelines*, by R.D. Holtz, B.R. Christopher, and R.R. Berg, FHWA HI-07-092 (2008).
- *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations*, by R.R. Berg, FHWA-SA-93-025, January 1993.
- *Reinforced Soil Structures - Volume I, Design and Construction Guidelines - Volume II, Summary of Research and Systems Information*, by B.R. Christopher, S.A. Gill, J.P. Giroud, J.K. Mitchell, F. Schlosser, and J. Dunnicliff, FHWA RD 89-043 (1990).
- *Design and Construction Monitoring of Mechanically Stabilized Earth Structures*, by J.A. DiMaggio, FHWA, (1994).
- AASHTO Bridge T-15 Technical Committee unpublished working drafts for the update of Section 11.0 of the AASHTO LRFD Bridge Design Specifications.

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Lastly, the authors wish to acknowledge the extensive work of the late Victor Elias, P.E. for his vital contributions and significant effort as Lead Author in preparing the earlier two (1997, 2001) versions of this manual, and as the author of the earlier companion manuals on corrosion/degradation of soil reinforcements. Mr. Elias was instrumental in the introduction and implementation of reinforced soil technology in the U.S., as a Vice President for The Reinforced Earth Company from 1974 to 1985. He was instrumental in research, refinement of design methods, and standards of practice and codes for MSE walls, as a Consultant from 1985 until 2006.

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CHAPTER 1

INTRODUCTION

1.1 OBJECTIVES

New methods and technologies of retention and steepened-slope construction continue to be developed, often by specialty contractors and suppliers, to solve problems in locations of restricted Right-of-Way (ROW), at marginal sites with difficult subsurface conditions and other environmental constraints, and to expedite construction. Professionals charged with the responsibility for planning, designing, and implementing improvements and additions in such locations should understand the application, limitations and costs associated with a host of measures and technologies available.

This manual was prepared to assist design engineers, specification writers, estimators, construction inspectors and maintenance personnel with the selection, design, construction and maintenance of Mechanically Stabilized Earth Walls (MSEW) and Reinforced Soil Slopes (RSS).

The design, construction and monitoring techniques for these structures have evolved over the last three decades as a result of efforts by researchers, material suppliers and government agencies to improve some single aspect of the technology or the materials used. This manual is a comprehensive document that integrates all design, construction, materials, contracting, and monitoring aspects required for successful project implementation.

This manual has been developed in support of FHWA educational programs on the design, construction, and maintenance of MSE wall and RSS structures construction. Its principal function is to serve as a reference source to the materials presented. The manual serves as FHWA's primary technical guideline on the use of these technologies on transportation facilities.

1.1.1 Scope

The manual addresses in a comprehensive manner the following areas:

- Overview of MSE development and the cost, advantages, and disadvantages of using MSE structures.
- Available MSE systems and applications to transportation facilities.
- Basic soil-reinforcement interaction.
- Design of routine and complex MSE walls.

- Design of MSE walls for extreme events.
- Design detailing of MSE walls.
- Design of steepened RSS.
- Specifications and contracting approaches for both MSE walls and RSS construction.
- Construction monitoring and inspection.
- Design examples.
- A separate companion manual addresses long-term corrosion of metallic reinforcements and long-term degradation of polymeric reinforcements. Sections of the Corrosion/Degradation manual address the background of full-scale, long-term evaluation programs and the procedures required to develop, implement, and evaluate them. These procedures have been developed to provide practical information on this topic for MSE users for non-corrosion or polymer specialists, who are interested in developing long-term monitoring programs for these types of structures.

As an integral part of this Manual, several example calculations are appended that demonstrate individual design aspects.

1.1.2 Source Documents

This Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS), Design and Construction Guidelines Manual is an update of the current FHWA NHI-00-043 (Elias et al., 2001), has evolved from the following AASHTO and FHWA references:

- AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2008 and 2009 Interim Revisions
- *Earth Retaining Structures*, FHWA-NHI-07-071 (Tanyu et al., 2008)
- AASHTO LRFD Bridge Construction Specifications, 2nd Edition, 2004, with 2006, 2007, 2008, and 2009 Interim Revisions
- *Geosynthetic Design and Construction Guidelines*, FHWA NHI-07-092 (Holtz et al., 2008)
- *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations*, FHWA-SA-93-025 (Berg, 1993)
- *Reinforced Soil Structures - Volume I, Design and Construction Guidelines - Volume II, Summary of Research and Systems Information*, FHWA RD 89-043 (Christopher et al. 1990)
- *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*, FHWA, (Elias and Christopher, 1997)

Additional guidance, where not available from other sources, was specifically developed for this manual.

1.1.3 Terminology

Certain interchangeable terms will be used throughout this manual. For clarity, they are defined as follows:

Inclusion is a generic term that encompasses all man-made elements incorporated in the soil to improve its behavior. Examples of inclusions are steel strips, geotextile sheets, steel or polymeric grids, steel nails, and steel tendons between anchorage elements. The term *reinforcement* is used only for those inclusions where soil-inclusion stress transfer occurs continuously along the inclusion.

Mechanically Stabilized Earth Wall (MSE wall or MSEW) is a generic term that includes reinforced soil (a term used when multiple layers of inclusions act as reinforcement in soils placed as fill). Reinforced Earth[®] is a trademark for a specific reinforced soil system.

Reinforced Soil Slopes (RSS) are a form of reinforced soil that incorporate planar reinforcing elements in constructed earth-sloped structures with face inclinations of less than 70 degrees.

Geosynthetics is a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and geogrids.

Facing is a component of the reinforced soil system used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, dry cast modular blocks, gabions, welded wire mesh, shotcrete, timber lagging and panels, polymeric cellular confinement systems, and wrapped sheets of geosynthetics. The facing also plays a minor structural role in the stability of the structure. For RSS structures it usually consists of welded wire mesh, geosynthetic wrap-around, and/or some type of erosion control material.

Retained backfill is the fill material located behind the mechanically stabilized soil zone.

Reinforced fill is the fill material in which the reinforcements are placed.

Generic cross sections of MSE structures are shown in Figures 1-1 and 1-2.

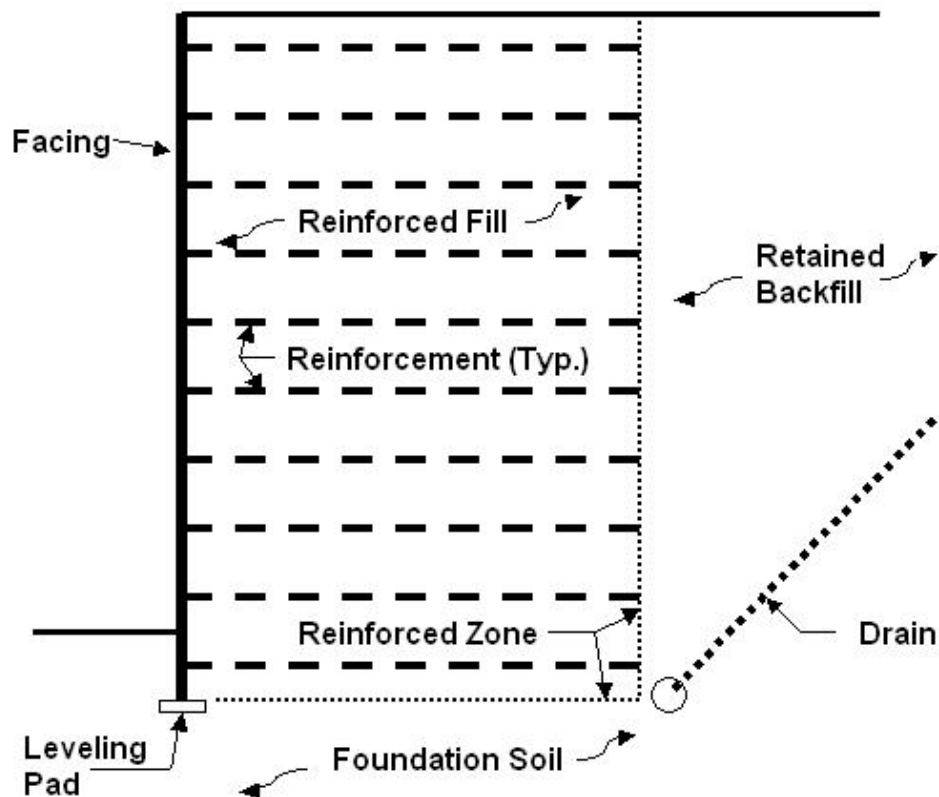
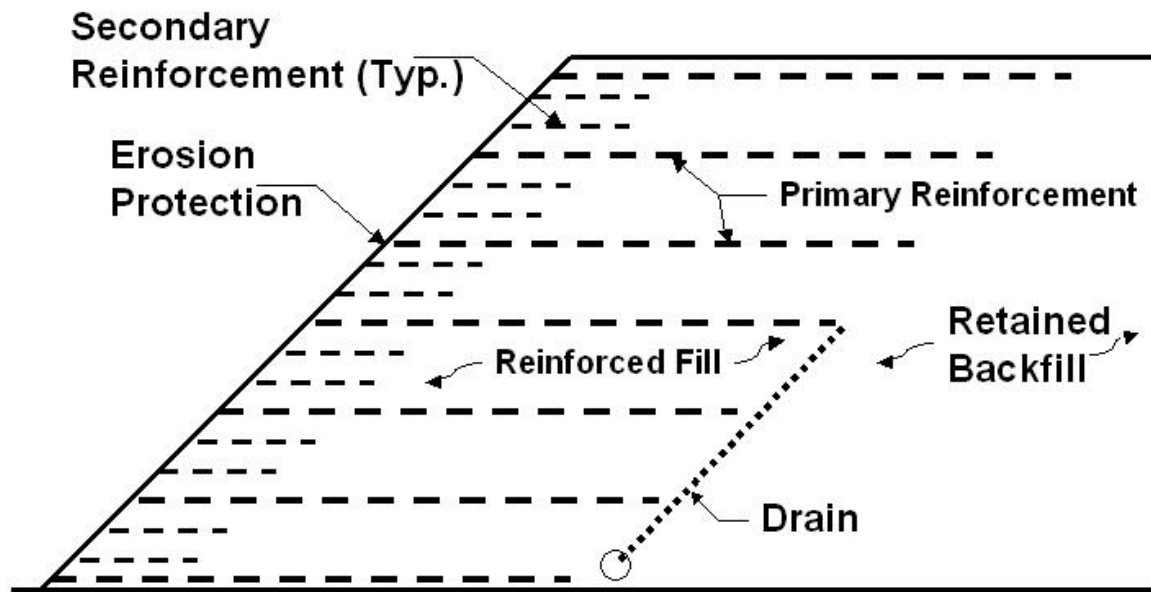


Figure 1-1. Generic cross section of an MSE structure.

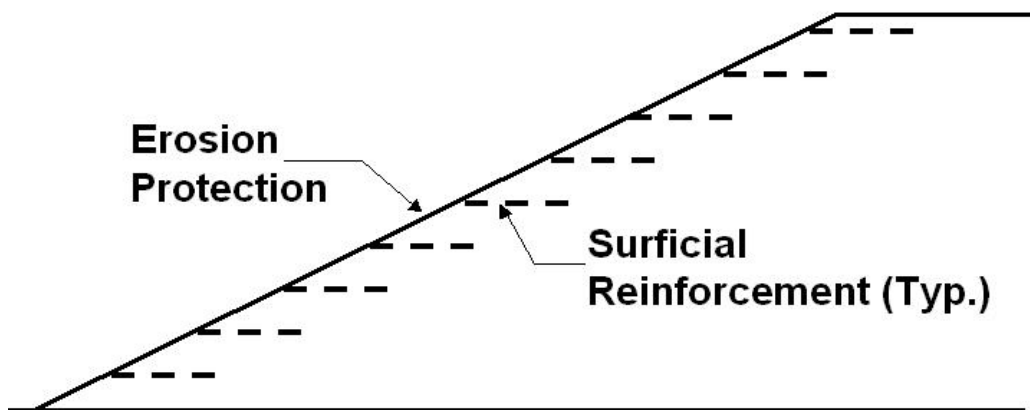
1.2 HISTORICAL DEVELOPMENT

Retaining structures are essential elements of every highway design. Retaining structures are used not only for bridge abutments and wing walls but also for slope stabilization and to minimize right-of-way for embankments. For many years, retaining structures were almost exclusively made of reinforced concrete and were designed as gravity or cantilever walls which are essentially rigid structures and cannot accommodate significant differential settlements unless founded on deep foundations. With increasing height of soil to be retained and poor subsoil conditions, the cost of reinforced concrete retaining walls increases rapidly.

Mechanically Stabilized Earth Walls (MSEWs) and Reinforced Soil Slopes (RSSs) are cost-effective soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements (inclusions) in the soil, the strength of the soil can be improved significantly. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely.



(a)



(b)

Figure 1-2. Generic cross sections of reinforced slope structures, reinforcements used to: (a) increase stability of a slope; and (b) provide improved compaction and surficial stability at edge of slopes (after Berg et al., 1990).

Inclusions have been used since prehistoric times to improve soil. The use of straw to improve the quality of adobe bricks dates back to earliest human history. Many primitive people used sticks and branches to reinforce mud dwellings. During the 17th and 18th centuries, French settlers along the Bay of Fundy in Canada used sticks to reinforce mud dikes. Some other early examples of man-made soil reinforcement include dikes of earth and tree branches, which have been used in China for at least 1,000 years (e.g., western portion of the Great Wall) and along the Mississippi River in the 1880s. Other examples include wooden pegs used for erosion and landslide control in England, and bamboo or wire mesh, used universally for revetment erosion control. Soil reinforcing can also be achieved by using live plant roots.

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth[®], a system in which steel strip reinforcement is used. The first wall to use this technology in the United States was built in 1972 on California State Highway 39, northeast of Los Angeles. Today, MSE walls are the wall of choice in most fill situations, and MSE walls are used extensively in the U.S. and worldwide. The highest permanent wall constructed in the United States is on the order of 150 ft (46 m) with an exposed height of approximately 135 ft (41 m).

Since the introduction of Reinforced Earth[®], several other proprietary and nonproprietary systems have been developed and used. Table 1-1 provides a partial summary of some of the current systems by proprietary name, reinforcement type, and facing system.

There are many available systems, as well as new systems that continue to be introduced into the market. Components, engineering details, system quality controls, etc. vary with each system. States, therefore, need a process to sort and evaluate MSE wall systems for potential pre-approval for use on their projects. The Highway Innovative Technology Evaluation Center (HITEC) provides review and evaluation of MSE walls. HITEC was established in 1994 within the American Society of Civil Engineers (ASCE) organization. HITEC's purpose is to accelerate the introduction of technological advances in products, systems, services, materials, and equipment to the highway and bridge markets. The evaluation of new and more cost-effective retaining wall systems is performed through HITEC's nationally-focused, earth retaining system (ERS) group evaluation program. The published reports provide reviews of design, construction, performance, and quality assurance information provided by the wall system suppliers with respect to conformance with the state-of-practice criteria as outlined in the HITEC Protocol. Wall system suppliers are encouraged to conduct an independent review of newly developed components and/or systems related to materials, design, construction, performance, and quality assurance. Some

public agencies, especially state DOTs, require HITEC evaluations or independent evaluations of wall components or wall systems, and obtaining such reviews has proven beneficial to wall system suppliers in securing acceptance of their system.

Currently, most process patents covering soil-reinforced system construction or components have expired, leading to a proliferation of available systems or components that can be separately purchased and assembled by the erecting contractor. The combination of components needs to be evaluated to assure compatibility with respect to longevity, constructability, and connection strength. The remaining patents in force generally cover only the method of connection between the reinforcement and the facing.

In the United States, a segmental precast facing unit 20 to 25 ft² (2 to 2.25 m²) generally square in shape is the facing unit of choice. More recently, larger precast units of up to 50 ft² (4.6 m²) have been used and are becoming more commonplace. Additionally, smaller dry-cast concrete masonry units are being used, generally in conjunction with geosynthetic reinforcements.

Table 1-1. Summary of Reinforcement and Face Details for MSE Wall Systems.

<u>System Name</u>	<u>Reinforcement Detail</u>	<u>Typical Face Detail¹</u>
Stabilized Earth Wall T&B Structural Systems LLC 6800 Manhattan Blvd, Suite 304 Ft. Worth Texas 76120	Galvanized welded steel wire mesh with W7 to W20 bars. Mesh width and spacing can vary. Epoxy-coated meshes also available.	Precast concrete panels 5 ft x 5 ft x 6 in. thick or 5 ft x 10 ft x 6 in. thick. Different size panels used at top and bottom to match project requirements.
Reinforced Earth® The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100 Vienna, VA 22182-2233	Ribbed galvanized steel strips, 0.157 in. thick, 2 in. wide. Or galvanized steel ladder strips, W10 wire, two longitudinal wires and cross bars spaced at 6 in.	Cruciform and square shaped precast concrete nominally 5 ft x 5 ft x 5.0 to 5.5 in. thick. Also rectangular shaped precast concrete nominally 5 ft x 10 ft x 5.5 in. thick. Variable height panels used at top and bottom of wall.
Retained Earth® The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100 Vienna, VA 22182-2233	Rectangular grid of W11, W15 or W20 galvanized steel wire, 24 x 6 in. grid. 2, 4, 5 or 6 longitudinal bars. Stainless steel mesh used in marine and corrosive environments.	Hexagonal and square precast concrete 5 ft x 5 ft x 5.5 in. thick. Also rectangular shaped precast concrete 5 ft x 10 ft x 5.5 in. thick. Variable height panels used at top and bottom of wall.
Mechanically Stabilized Embankment California Dept. of Transportation 1801 30 th Street P.O. Box 168041 Sacramento, CA 95816	Rectangular grid of W11, W15, and W20 galvanized welded wire mats, 6 longitudinal wires with variable transverse spacing.	Precast concrete; 5 ft square, 6 in. thick.
ARES Tensar International Corporation 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328	HDPE Geogrid	Precast concrete panel; rectangular 9 ft wide, 5 ft high, 5.5 in. thick.
Wire Faced Wall T&B Structural Systems LLC 6800 Manhattan Blvd, Ste 304 Ft. Worth Texas 76120	4 ft wide welded steel wire mesh. Mesh is 8 in. x 12, 18 or 24 in., of W4.5 to W20 bars. Size and configuration are variable.	Welded steel wire mesh facing. Several veneer facing options available.

<u>System Name</u>	<u>Reinforcement Detail</u>	<u>Typical Face Detail¹</u>
Welded Wire Wall Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	Welded steel wire mesh, 2' tall x 8' wide typical. Backing mat, Hardware Cloth or Filter Fabric depending on project. (With geotextile or shotcrete, if desired).
Reinforced Soil Embankment Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	Precast concrete unit 12.5 ft long, 24 in. high.
ArtWeld Gabions Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	ArtWeld Gabion baskets of various sizes and heights designed per project requirements.
Gabion Faced M.S.E. Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	ArtWeld Gabions of various sizes and heights connected to reinforcing mesh by spiral binders.
Eureka Reinforced Soil Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	Precast or cast-in-place concrete facing panels, shotcrete, sculpted shotcrete, or stacked stone.
Steepened Slope Hilfiker Retaining Walls 1902 Hilfiker Lane Eureka, CA 95503	Welded steel wire mesh, Galvanized or Non-Galvanized. Mesh reinforcements vary in spacing and gauges to meet project design specifications.	Welded steel wire mesh, 1 to 1 slope typical. Hardware Cloth or Filter Fabric depending on project. (With geotextile or shotcrete, if desired).
INTER-LOK Atlantic Concrete Industries P.O. Box 129 Tullytown, PA 19007	0.63 or 0.75 in. reinforcing steel bars fitted with 5 x 10 x 0.4 in. anchor plates and connected to a keyplate, and galvanized after fabrication.	Precast concrete panel; cross-shaped 6 ft wide and 3 ft high, 8 and 10 in. thick.
ISOGRID Neel Co. 6520 Deepford Street Springfield, VA 22150	Rectangular grid of W11 x W11 4 bars per grid.	Diamond shaped precast concrete units, 5 ft x 8 ft, 5.5 in. thick.
T-Block Wall System T&B Structural Systems LLC 6800 Manhattan Blvd Ste 304 Ft. Worth Texas 76120	Rectangular 4 ft wide welded steel wire mesh of W7 to W20 steel bars.	Dry cast concrete block 8 in high x 16 in long x 12 in deep.
MESA Tensar International Corporation 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328	HDPE Geogrid	MESA HP (high performance), DOT ³ OR Standard units (8 in. high by 18 in. long face, 10.8 in. nominal depth). (dry cast concrete)
Pyramid TM The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100 Vienna, VA 22182-2233	Galvanized welded wire ladders. Size varies with design requirements.	Dry cast concrete units, 8 in. high, 16 in. nominal length at face, 10 in. nominal depth.
Omega The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100 Vienna, VA 22182-2233	Geostraps composed of high tenacity polyester with polyethylene sheathing. Reinforcement used in marine and corrosive environments only.	Cruciform and square shaped precast concrete 5 ft x 5 ft x 5.5 in. thick. Also rectangular shaped precast concrete 5 ft x 10 ft x 5.5 in. thick. Variable height panels used at top and bottom of wall.
Geotrel TM The Reinforced Earth Company 8614 Westwood Center Dr, Ste 1100 Vienna, VA 22182	Geostraps composed of high tenacity polyester with polyethylene sheathing. Only used in temporary walls.	Welded steel wire mesh with geotextile backing.

<u>System Name</u>	<u>Reinforcement Detail</u>	<u>Typical Face Detail¹</u>
Terratrel™ The Reinforced Earth Company 8614 Westwood Center Drive Suite 1100 Vienna, VA 22182-2233	Ribbed galvanized steel strips, 0.157 in. thick, 2 in. wide. Or, galvanized steel ladder strips or mesh. Size varies with design requirements.	Welded steel wire mesh with geotextile backing or stone fill at wall face.
Maccaferri Terramesh System Maccaferri Gabions, Inc. 43A Governor Lane Blvd. Williamsport, MD 21795	Continuous sheets of galvanized double twisted woven wire mesh with PVC coating.	Rock filled gabion baskets laced to reinforcement.
Strengthened Earth Gifford-Hill & Co. 2515 McKinney Ave. Dallas, Texas 75201	Rectangular grid, W7, W9.5 and W14, transverse bars at 9 and 18 in.	Precast concrete units, rectangular or wing shaped, 6 ft x 7 ft x 5.5 in.
MSE Plus SSL 4740 Scotts Valley Drive Scotts Valley, CA 95066	Rectangular grid with W8 to W24 longitudinal bars and W8 to W20 transverse. Mesh may have 2 – 6 longitudinal bars spaced at 6 or 8 in.	Rectangular precast concrete panels 5 ft high, 5, 6, 10, and 12 ft wide, with a thickness of 6 or 7 in.
KeySystem – Inextensible Keystone Retaining Wall Systems 4444 W. 78 th Street Minneapolis, MN 55435	Galvanized welded wire ladder mat of W7.5 to W17 bars with crossbars at 6 – 24 in.	KeySystem concrete facing unit is 8 in high x 18 in. wide x 12 in. deep (dry cast concrete).
KeySystem – Extensible Keystone Retaining Wall Systems 4444 W. 78 th Street Minneapolis, MN 55435	Miragrid high-tenacity polyester geogrid soil reinforcement by TenCate Mirafi, polymer coated.	Keystone Compac concrete facing units are 8 in. high x 18 in. wide x 12 in. deep (dry cast concrete).
Tricon System Tricon Precast Ltd. 15055 Henry Road Houston, TX 77060	Galvanized welded-wire.	Rectangular precast concrete panels with a face area of 45 sq. ft.
Versa-Lok Retaining Wall Systems 6348 Highway 36 Blvd. Oakdale, MN 55128	PVC coated PET or HDPE geogrids.	Versa-Lok concrete unit 6 in. high x 16 in. long x 12 in. deep (dry cast concrete)
Anchor Wall Systems 5959 Baker Road Minnetonka, MN 55345	PVC coated PET geogrid.	Anchor Landmark concrete unit 15 in. high x 8 in. long x 12 (small unit) or 12.5 (large unit) in. deep (dry cast concrete).
EarthTrac™ HA EarthTec Inc. 413 Browning Ct. Purcellville, VA 20132	Ribbed galvanized steel strips, 0.188 in. thick by 2.36 in. wide.	Rectangular 5 ft x 10 ft precast concrete panels.
EarthTrac™ Wire EarthTec Inc. 413 Browning Ct. Purcellville, VA 20132	Ribbed steel strips, 0.188 in. thick by 2.36 in. wide; galvanized for permanent walls.	Welded wire basket 2.5 ft high by 10 ft wide.
EarthTrac™ Synthetic EarthTec Inc. 413 Browning Ct. Purcellville, VA 20132	PVC coated high tenacity polyester geostraps.	Precast concrete panels, rectangular or T-shaped.
¹ Additional facing types are possible with most systems.		

The use of geotextiles in MSE walls and RSS started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments constructed over weak subgrades. The first geotextile-reinforced wall was constructed in France in 1971, and the first structure of this type in the United States was constructed in 1974. Geogrids for soil

reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a growing portion of the market. Since the early 1980s, the use of geosynthetics in reinforced soil structures has increased significantly.

The first reported use of reinforced steepened slopes is believed to be the west embankment for the Great Wall of China. The introduction and economy of geosynthetic reinforcements has made the use of steepened slopes economically attractive. A survey of usage in the mid 1980s identified several hundred completed projects. At least an order of magnitude more RSS structures have been constructed since that study. The highest constructed RSS structure in the U.S. to date is 242 ft (74 m) (see Chapter 8).

A representative list of geosynthetic reinforcement manufacturers and suppliers is shown in Table 1-2.

Current Usage: It is believed that MSEWs have been constructed in every state in the United States. Major users include transportation agencies in Georgia, Florida, Texas, Pennsylvania, New York, and California, which rank among the largest road building states.

It is estimated that more than 9,000,000 ft² (850,000 m²) of MSE retaining walls with precast facing are constructed on average every year in the United States, which may represent more than half of all retaining wall usage for transportation applications.

The majority of the MSEWs for permanent applications either constructed to date or presently planned use a segmental precast concrete facing and galvanized steel reinforcements. The use of geotextile faced MSEWs in permanent construction has been limited to date. They are quite useful for temporary construction, where more extensive use has been made.

Recently, modular block dry cast facing units have gained acceptance due to their lower cost and nationwide availability. These small concrete units are generally mated with grid reinforcement, and the wall system is referred to as modular block wall (MBW). It is estimated that more than 3,000,000 ft² (280,000 m²) of MBW walls have been constructed yearly in the United States when considering all types of transportation related applications. The current yearly usage for transportation-related applications is estimated at about 100 projects per year.

The use of RSS structures has expanded dramatically in the last decade, and it is estimated that several hundred RSS structures have been constructed in the United States. Currently,

100 to 150 RSS projects are being constructed yearly in connection with transportation related projects in the United States, with an estimated projected vertical face area of 2,000,000 ft²/year (190,000 m²/yr).

Table 1-2 – Representative List of Geogrid and Geotextile Reinforcement Suppliers.

ACE Geosynthetics Enterprise Co., Ltd. No. 8 Kung 10 Rd. Yu-Shih Ind. Park, Tachia Taichung 43768 Taiwan www.geoace.com	Belton Industries Inc. 5600 Oakbrook Pkwy Ste 150 Norcross, GA 30093-1843 www.beltonindustries.com
Carthage Mills 4243 Hunt Rd Cincinnati, OH 45242-6645 www.carthagemills.com	Checkmate Geosynthetics Inc. Unit# 412 44500 South Sumas Rd. Chilliwack, BC V2R 5M3 Canada www.checkmategeogrid.com
Colbond Inc. PO Box 1057 1301 Sand Hill Rd Enka, NC 28728-1057 www.colbond.com	Fiberweb PLC 70 Old Hickory Blvd. Old Hickory, TN 37138 www.fiberweb.com
Dalco Nonwovens PO Box 1479 2050 Evergreen Dr Ne Conover, NC 28613-1479 www.dalcononwovens.com	Geo-Synthetics Inc. 2401 Pewaukee Rd Waukesha, WI 53188 www.geo-synthetics.com
GSE Lining Technology Inc. 19103 Gundle Rd Houston, TX 77073-3515 www.gseworld.com	Highland Industries Inc 629 Green Valley Rd., Suite 210 Greensboro, NC 27408 www.highlandindustries.com
Huesker Inc. PO Box 411529 Charlotte, NC 28241-1529 www.hueskerinc.com	Layfield Plastics Inc. 11603 180th St SW Edmonton, AB T5S 2H6 Canada www.layfieldgroup.com
Luckenhaus Technical Textiles Inc. 3130 Bee Tree Ln Signal Mountain, TN 37377-1441	Maccaferri Inc. 10303 Governor Lane Blvd Williamsport, MD 21795-3115 www.maccaferri-usa.com
NAUE America Inc. 3525 Piedmont Rd NE 7 Piedmont Center Ste 300 Atlanta, GA 30305-1578 www.naue.com	Propex Geosynthetics 6025 Lee Highway, Ste. 425 P.O. Box 22788 Chattanooga, TN 37422 www.propexinc.com
Saint-Gobain Technical Fabrics 1795 Baseline Rd Grand Island, NY 14072-2010 www.glasrid.com	SKAPS Industries 335 Athena Dr Athens, GA 30601 www.nevown.com

Strata Systems, Inc., Div. Glen Raven, Inc. PO Box 791 Statesville, NC 28687-0791 www.geogrid.com	SynTeen Technical Fabrics 1950 W Meeting St Lancaster, SC 29720-8811 www.synteen.com
Tenax Corp. 4800 E Monument St Baltimore, MD 21205-3031	TenCate Geosynthetics 365 S Holland Dr Pendergrass, GA 30567-4625 www.tencate.com
Tensar International Corporation 5883 Glenridge Dr NE Ste 200 Atlanta, GA 30328-5571 www.tensar-international.com	Thrace-LINQ Inc. 2550 W 5th North St Summerville, SC 29483-9665
TNS Advanced Technologies By Crown Resources 856 S Pleasantburg Dr Greenville, SC 29607-2455	Vantage Partners, LLC 1000 Bucks Industrial Park Dr Statesville, NC 28625-2575 www.vp-geos.com
* List is from the Geosynthetics Materials Association.	

1.3 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

The most significant revision/update of this reference manual is the change of design procedure for MSE walls from an allowable stress design (ASD) basis to load and resistance factor design (LRFD) basis. Transportation superstructures are designed using LRFD procedures, and logically the substructures supporting the superstructures should also be designed on a LRFD basis to provide design consistency on the overall project. Therefore, FHWA and the AASHTO Subcommittee on Bridges and Substructures established an October 1, 2010 deadline for implementation of LRFD in wall design.

Although the implementation of LRFD requires a change in design procedures for engineers accustomed to ASD, many advantages do exist. LRFD separately accounts for uncertainty in both resistance and load, and when appropriately calibrated, can provide more consistent levels of safety in the design of superstructure and substructure components in terms of reliability index. Section 11 of the AASHTO LRFD Specification (2007) provides information on LRFD for earth retaining structures including mechanically stabilized earth (MSE) walls. Section 10.4 of AASHTO (2007) provides detailed information on the evaluation of soil and rock properties to be used for design. Section 3 of AASHTO (2007) provides detailed information on vertical and lateral loads, and load factors for the design of retaining walls.

For many years, engineers have designed walls for highway and other applications using allowable stress design (ASD) methods. (Note that the AASHTO (2002) and FHWA (Elias et al., 2001) ASD references will not be updated by AASHTO or FHWA, respectively.) In

ASD, all uncertainty in applied loads and material resistance are combined in a factor of safety or allowable material stress. Furthermore, the factor of safety is independent of the method used to estimate the resistance. In LRFD, uncertainty in load and material resistance are accounted for separately. The uncertainty in load is represented by a load factor and the uncertainty in material resistance is represented by a resistance factor. More importantly, the resistance factor is a function of the method used to estimate the resistance and thus the model uncertainty is also included in the design process.

In the AASHTO-LRFD framework, there are four limit states, which represent distinct structural performance criteria: (1) strength limit states; (2) serviceability limit states; (3) extreme event limit states; and (4) fatigue limit states. For most earth retaining system designs, the strength or service limit states control the design. For walls subject to earthquake or vessel/vehicle impact, the extreme limit states may control.

This manual, and the accompanying training course curriculum materials, have been prepared assuming that the user is familiar with LRFD general procedures. Agencies can receive detailed training and reference materials on LRFD procedures for substructures from the FHWA NHI 130082 training course (see www.nhi.fhwa.dot.gov).

This manual also provides detailed procedures for the design, specification, and construction of reinforced soil slopes (RSS). The AASHTO LRFD Bridge Design Specifications (2007) do not address RSS structures. Therefore, the design for RSS remains based upon a limit equilibrium slope stability basis within this manual.

CHAPTER 2

SYSTEMS AND PROJECT EVALUATION

This chapter describes available MSE wall (MSEW) and RSS systems and components, their application, advantages, disadvantages and relative costs. Subsequently, it reviews typical construction sequence for MSEW and RSS construction, and outlines required site and project evaluations leading to the establishment of site-specific project criteria and details.

2.1 APPLICATIONS

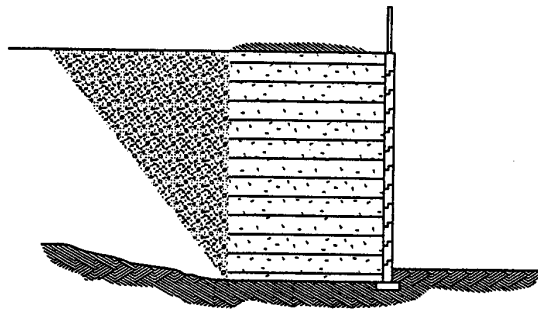
2.1.1 MSE Walls

MSEW structures are cost-effective alternatives for most applications where reinforced concrete or gravity type walls have traditionally been used to retain soil. These include bridge abutments and wing walls, as well as areas where the right-of-way is restricted, such that an embankment or excavation with stable side slopes cannot be constructed. They are particularly suited to economical construction in steep-sided terrain, in ground subject to slope instability, or in areas where foundation soils are poor.

MSE walls offer significant technical and cost advantages over conventional reinforced concrete retaining structures at sites with poor foundation conditions. In such cases, the elimination of costs for foundation improvements such as piles and pile caps, that may be required for support of conventional structures, have resulted in cost savings of greater than 50 percent on completed projects.

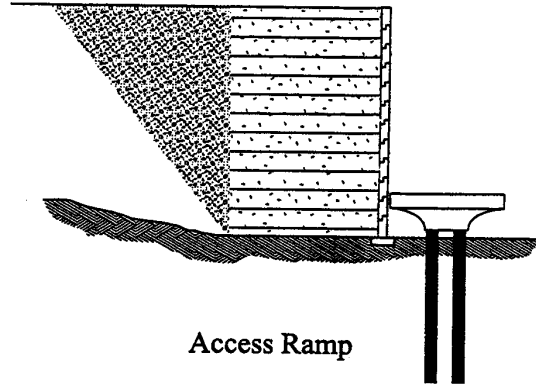
Representative uses of MSE walls for various applications are shown in Figure 2-1.

Temporary MSE wall structures have been especially cost-effective for temporary detours necessary for highway reconstruction projects. Temporary MSE walls are used to support temporary roadway embankments and temporary bridge abutments, as illustrated in Figure 2-2. MSE walls are also used as temporary support of permanent roadway embankments for phased construction, an example is shown in Figure 2-3.



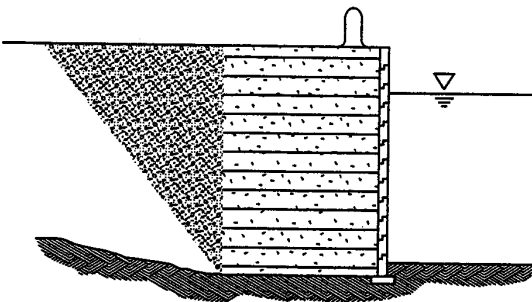
Retaining Wall

(a)



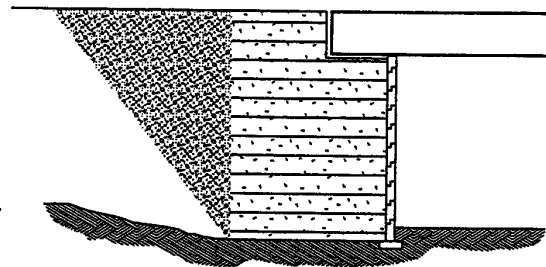
Access Ramp

(b)



Waterfront Structure

(c)



Bridge Abutment

(d)

Figure 2-1. Representative MSE wall applications (a) retaining wall; (b) access ramp; (c) waterfront structure; and (d) bridge abutment.



Figure 2-2. MSE walls to support temporary bridge abutment and roadway embankment.



Figure 2-3. MSE wall used to temporarily support a permanent roadway embankment for phased construction.

2.1.2 Reinforced Soil Slopes

Reinforced soil slopes (RSS) are a form of mechanically stabilized earth that incorporate planar reinforcing elements (typically geosynthetics) in constructed earth sloped structures with face inclinations of less than 70 degrees. As shown in Figure 2-4, multiple layers of reinforcement are placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. RSS structures are cost-effective alternatives for new construction and reconstruction where the cost of fill, right-of-way, and other considerations may make a steeper slope desirable.

There are two primary purposes for using reinforcement in engineered slopes.

- To increase the stability of the slope, particularly if a steeper than *safe* unreinforced slope is desirable or after a failure has occurred as shown in Figure 2-4.
- To provide improved compaction at the edges of a slope, thus decreasing the tendency for surface sloughing as shown in Figure 1-2b.

Reinforcement is used to construct an embankment at an angle steeper than could otherwise be safely constructed with the same soil. The increase in stability allows for construction of steepened slopes on firm foundations for new highways and as an alternative to flatter unreinforced slopes and to retaining walls. Roadways can also be widened over existing flatter slopes without encroaching beyond existing right-of-ways. In the case of repairing a slope failure, the new slope will be safer, and reusing the slide debris rather than importing higher quality backfill may result in substantial cost savings. These applications are illustrated in Figure 2-4.

The second purpose for using reinforcement is at the edges of a compacted fill slope to provide lateral resistance during compaction. The increased lateral resistance allows for an increase in compacted soil density over that normally achieved and provides increased lateral confinement for the soil at the face. Even modest amounts of reinforcement in compacted slopes have been found to prevent sloughing and reduce slope erosion. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope.

Further compaction improvements have been found in cohesive soils through the use of geosynthetics with in-plane drainage capabilities (e.g., nonwoven geotextiles) that allow for rapid pore pressure dissipation in the compacted soil.

Compaction aids placed as intermediate layers between reinforcement in steepened slopes may also be used to provide improved face stability and to reduce layers of more expensive primary reinforcement as shown in Figure 1-2.

Other applications of reinforced slopes have included:

- Decreased bridge spans.
- Temporary road widening for detours.
- Prevention of surface sloughing during periods of saturation.
- Embankment construction with wet, fine-grained soils.
- Permanent levees.
- Temporary flood control structures.

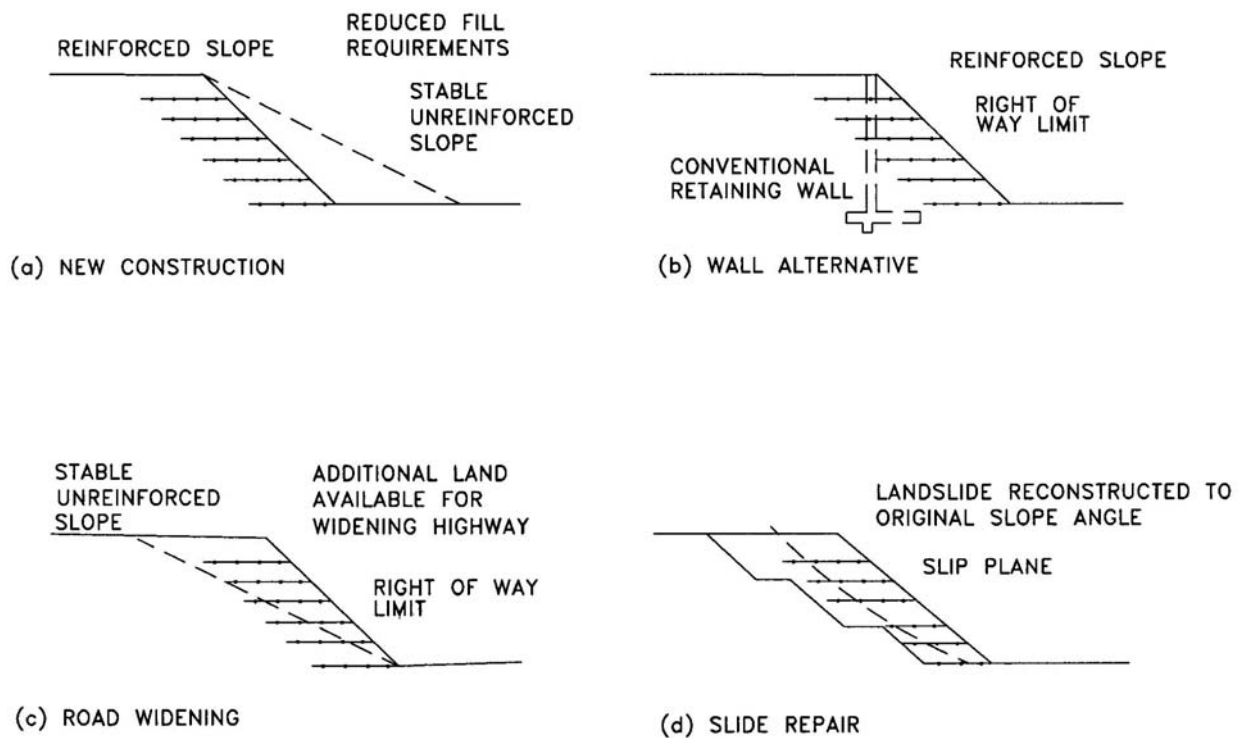


Figure 2-4. Application of reinforced soil slopes.

2.2 ADVANTAGES AND POTENTIAL DISADVANTAGES

2.2.1 Advantages of Mechanically Stabilized Earth (MSE) Walls

MSE walls have many advantages compared with conventional reinforced concrete and concrete gravity retaining walls. MSE walls:

- Use simple and rapid construction procedures and do not require as large of construction equipment.
- Do not require special skills for construction.
- Require less site preparation than other alternatives.
- Need less space in front of the structure for construction operations.
- Reduce right-of-way acquisition.
- Do not need rigid, unyielding foundation support because MSE structures are tolerant to deformations.
- Are cost effective.
- Are technically feasible to heights in excess of 100 ft (30 m).

Pre-manufactured materials, rapid construction, and, competition among different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. MSE walls are likely to be more economical than other wall systems for walls higher than about 10 ft (3 m) or where special foundations would be required for a conventional wall.

One of the greatest advantages of MSE walls is their flexibility and capability to tolerate deformations due to poor subsoil foundation conditions. Also, based on observations in seismically active zones, these structures have demonstrated a higher resistance to seismic loading than rigid concrete wall structures.

Precast concrete facing elements for MSE walls can be made with various shapes and textures (with little extra cost) for aesthetic considerations. Masonry units, timber, and gabions also can be used to blend in the environment.

2.2.2 Advantages of Reinforced Soil Slopes (RSS)

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the result of material and right-of-way savings. It also may be possible to decrease the quality of materials required for construction. For example, in repair of landslides it is possible to reuse the slide debris rather than to import higher quality backfill. Right-of-way savings can be a substantial benefit, especially for road widening projects in urban areas where acquiring new right-of-way is always expensive and, in some cases,

unobtainable. RSS also provide an economical alternative to retaining walls. In some cases, reinforced slopes can be constructed at about one-half the cost of MSEW structures.

The use of vegetated-faced reinforced soil slopes that can be landscaped to blend with natural environments may also provide an aesthetic advantage over retaining wall structures. However, there are some potential maintenance issues that must be addressed such as mowing grass-faced steep slopes; however, these can be satisfactorily handled in design.

In terms of performance, due to inherent conservatism in the design of RSS, they are actually safer than flatter, unreinforced slopes designed at the same factor of safety. As a result, there is a lower risk of long-term stability problems developing with a reinforced slope. Such problems often occur in compacted fill slopes that have been constructed to low factors of safety and/or with marginal materials (e.g. deleterious soils such as shale, fine grained low cohesive silts, plastic soils, etc.). The reinforcement may also facilitate strength gains in the soil over time from soil aging and through improved drainage, further improving long-term performance.

2.2.3 Potential Disadvantages

The following general potential disadvantages may be associated with all reinforced soil structures, and are dependent upon local and project conditions:

- Require a relatively large space (e.g., excavation if in a cut) behind the wall or slope face to install required reinforcement.
- MSE walls require the use of select granular fill. (At some sites, the cost of importing suitable fill material may render the system uneconomical.) Reinforced fill requirements for RSS are typically less restrictive.
- The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners.

2.3 RELATIVE COSTS

Site specific costs of a soil-reinforced structure are a function of many factors, including cut-fill requirements, wall/slope size and type, in-situ soil type, available backfill materials, facing finish, temporary or permanent application, etc. It has been found that MSE walls with precast concrete facings are usually less expensive than reinforced concrete retaining

walls for heights greater than about 10 ft (3 m) and average foundation conditions. Modular block wall (MBW) unit faced walls are competitive with concrete walls at all heights and also for small projects.

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition). A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can accommodate relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. Typical total costs for permanent transportation MSE walls range from \$30 to \$65 per ft² (\$320 to \$650 per m²) of face, and generally vary as function of height, size of project, aesthetic treatment, site accessibility, and cost of select wall fill. However, reinforced fill costs vary considerably across the U.S. and regional costs may be much higher than the indicated range (not just for MSE walls, but for other wall types as well). Some example costs are presented with the case histories in Section 2.10.

The actual cost of a specific MSEW structure will depend on the cost of each of its principal components. For segmental precast concrete faced structures, typical relative costs are:

- Erection of panels and contractors profit - 20 to 30 percent of total cost.
- Reinforcing materials - 15 to 30 percent of total cost.
- Facing system - 20 to 40 percent of total cost.
- Reinforced wall fill including placement - 30 to 60 percent of total cost, where the fill is a select granular fill from an off-site borrow source.

The additional cost for panel architectural finish treatment ranges from \$0.50 to \$1.50 per ft² (\$5 to \$15 per m²) depending on the complexity of the finish. Traffic barrier costs average \$170 per linear foot (\$550 per linear m). In addition, consideration must be given to the cost of excavation, which may be somewhat greater than for other systems due to the required width of the reinforcement zone. MBW faced walls at heights less than 15 ft (4.5 m) are typically less expensive than segmental panel faced walls by 10 percent or more.

The economy of using RSS must be assessed on a case-by-case basis, where use is not dictated by space constraints. For such cases, an appropriate benefit to cost ratio analysis should be conducted to determine whether a steeper slope with the reinforcement is justified economically over the alternative flatter slope with its increased right-of-way and materials costs, etc. It should be kept in mind that guardrails or traffic barriers are often necessary for steeper embankment slopes and additional costs such as erosion control systems for slope face protection must be considered.

With respect to economy, the factors to consider are as follows:

- Cut or fill earthwork quantities.
- Size of slope area.
- Average height of slope area.
- Angle of slope.
- Cost of nonselect versus select backfills.
- Temporary and permanent erosion protection requirements.
- Cost and availability of right-of-way needed.
- Complicated horizontal and vertical alignment changes.
- Need for temporary excavation support systems.
- Maintenance of traffic during construction.
- Aesthetics.
- Requirements for guardrails and traffic barriers.

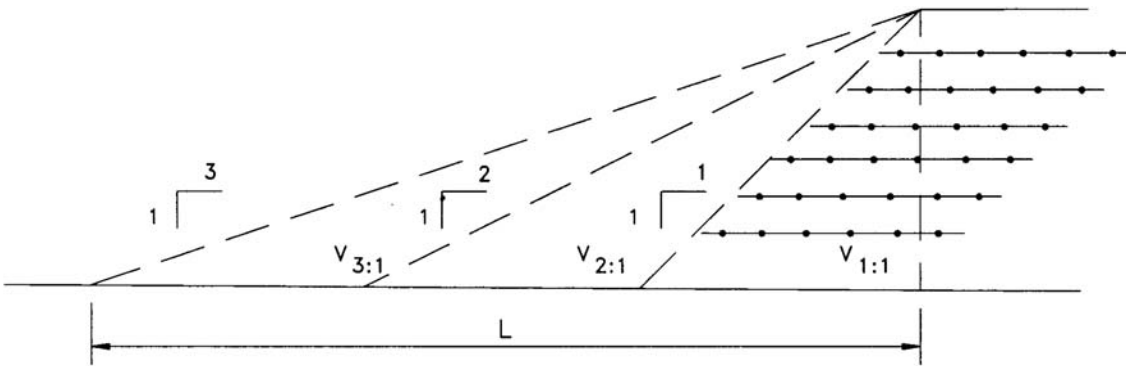
The actual bid cost of a specific RSS structure depends on the cost of each of its principal components. Based on limited data, typical relative costs are:

- Reinforcement - 45 to 65 percent of total cost
- Reinforced fill - 30 to 50 percent of total cost
- Face treatment - 5 to 10 percent of total cost

High RSS structures have relatively higher reinforcement and lower backfill costs. Recent bid prices suggest costs ranging from \$10/ft² to \$24/ft² (\$110/m² to \$260/m²) as a function of height.

For applications in the 30 to 50 ft (10 to 15 m) height range, bid costs of about \$16/ft² (\$170/m²) have been reported. These prices do not include safety features and drainage details.

A rapid, first-order assessment of cost items for comparing a flatter unreinforced slope with a steeper reinforced slope is presented in Figure 2-5.



$$\begin{array}{ll} V_{3:1} = V & V_{3:1} = L \\ V_{2:1} = \frac{2}{3}V & V_{2:1} = \frac{2}{3}L \\ V_{1:1} = \frac{1}{3}V & V_{1:1} = \frac{1}{3}L \end{array}$$

COST:

$$\begin{array}{l} 3H:1V = V_{\text{SOIL}} + L_{\text{LAND}} + \text{Guardrail* (?) + Hydroseeding (?)} \\ 2H:1V = \frac{2}{3}V_{\text{SOIL}} + \frac{2}{3}L_{\text{LAND}} + \text{Guardrail + Erosion Control + High Maintenance} \\ 1H:1V = \frac{1}{3}V_{\text{SOIL}} + \frac{1}{3}L_{\text{LAND}} + \text{Reinforcement + Guardrail + Erosion Control} \end{array}$$

* Include guardrail or traffic barrier cost if required.

Figure 2-5. Cost evaluation of reinforced soil slopes.

2.4 DESCRIPTION OF MSE and RSS SYSTEMS

2.4.1 Systems Differentiation

Since the expiration of the fundamental process and concrete facing panel patents obtained by the Reinforced Earth Company for MSE wall systems and structures, the engineering community has adopted a generic term *Mechanically Stabilized Earth* (MSE) to describe this type of retaining wall construction.

Trademarks, such as Reinforced Earth[®], Retained Earth[®], Genesis[®] etc., describe systems with some present or past proprietary features or unique components marketed by nationwide commercial suppliers. Other trademark names appear yearly to differentiate systems marketed by competing commercial entities that may include proprietary or novel components or for special applications.

A system for either MSEW or RSS structures is defined as a complete supplied package that includes design, specifications and all prefabricated materials of construction necessary for

the complete construction of a reinforced soil structure. Often technical assistance during the planning and construction phase is included. Components marketed by commercial entities for integration by the owner, or others, into a coherent package are not classified as systems. Generic systems created by combining components are also possible; however, the components must be tested and evaluated together in the form of the final system. Components cannot be substituted without complete evaluation of the impact on the system.

2.4.2 Types of Systems

MSE/RSS systems can be described by the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and the type of facing and connections.

Reinforcement Geometry Three types of reinforcement geometry can be considered:

- **Linear unidirectional.** Strips, including smooth or ribbed steel strips, or coated geosynthetic strips over a load-carrying fiber.
- **Composite unidirectional.** Grids or bar mats characterized by grid spacing greater than 6 in. (150 mm).
- **Planar bi-directional.** Continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh. The mesh is characterized by element spacing of less than 6 in. (150 mm).

Reinforcement Material Distinction can be made between the characteristics of metallic and nonmetallic reinforcements:

- **Metallic reinforcements.** Typically of mild steel. The steel is usually galvanized.
- **Nonmetallic reinforcements.** Generally polymeric materials consisting of polyester or polyethylene.

The performance and durability considerations for these two classes of reinforcement vary considerably and are detailed in the companion Corrosion/Degradation manual (FHWA NHI-09-087; Elias et al., 2009).

Reinforcement Extensibility There are two classes of extensibility relative to the soil's extensibility:

- **Inextensible.** The deformation of the reinforcement at failure is much less than the deformability of the soil. Steel strip and bar mat reinforcements are inextensible.
- **Extensible.** The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil. Geogrid, geotextile, and woven steel wire mesh reinforcements are extensible.

2.4.3 Facing Systems

The types of facing elements used in the different MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing, as shown in the FHWA Federal Lands Highway Division's Roadway Aesthetic Treatments Photo Album (RATPA) available at <http://gallery.companys39.com/FLH/gallery/>. In addition, the facing provides protection against backfill sloughing and erosion, and provides, in certain cases, drainage paths. The type of facing influences settlement tolerances. Major facing types are:

- **Segmental precast concrete panels.** The various shapes and dimensions of segmental precast panels are summarized in Table 1-1, and examples are illustrated in Figure 2-6 (and in Figure 5-33). The precast concrete panels have a minimum thickness of 5-½ inches (140 mm) and are of a square, rectangular, cruciform, diamond, or hexagonal geometry. Typical nominal panel dimensions are 5-foot (1.5 m) high and 5- or 10-foot (1.5 or 3 m) wide. Temperature and tensile reinforcement of the concrete are required and should be designed in accordance with Section 5 of AASHTO LRFD Specifications for Highway Bridges (2007).
- **Dry cast modular block wall (MBW) units.** These are relatively small, squat concrete units that have been specifically designed and manufactured for retaining wall applications. The weight of these units commonly ranges from 30 to 110 lbs (15 to 50 kg), with units of 75 to 110 lbs (35 to 50 kg) routinely used for highway projects. Unit heights typically range from 4 to 12 in. (100 to 300 mm) for the various manufacturers, with 8-in. (200 mm) typical. Exposed face length usually varies from 8 to 18 in. (200 to 450 mm). Nominal front to back width (dimension perpendicular to the wall face) of units typically ranges between 8 and 24 in. (200 and 600 mm). Units may be manufactured solid or with cores. Full height cores are filled with aggregate during erection. Units are normally dry-stacked (i.e. without mortar or bearing pads) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys. They are referred to by trademarked names such as Keystone[®], Landmark[®], Mesa[®], Versa-Lok[®], etc. Several example MBW units are illustrated in Figure 2-7.
- **Welded Wire Mesh (WWM).** Wire grid can be bent up at the front of the wall to form the wall face. This type of facing is used for example in the Hilfiker, Tensar, and Reinforced Earth wire faced retaining wall systems. This type of facing is commonly used for RSS with face angles of about 45 degrees and steeper.

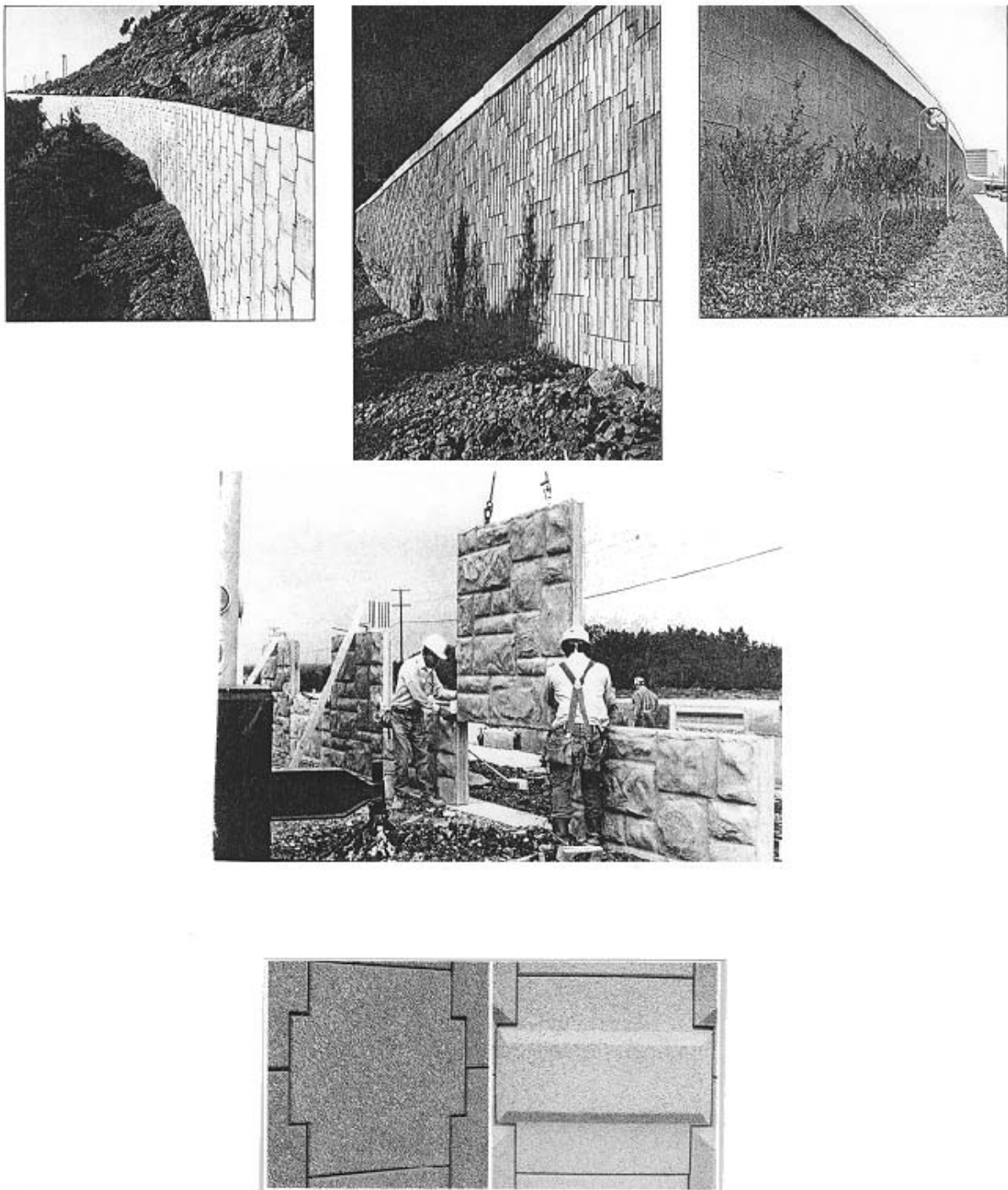
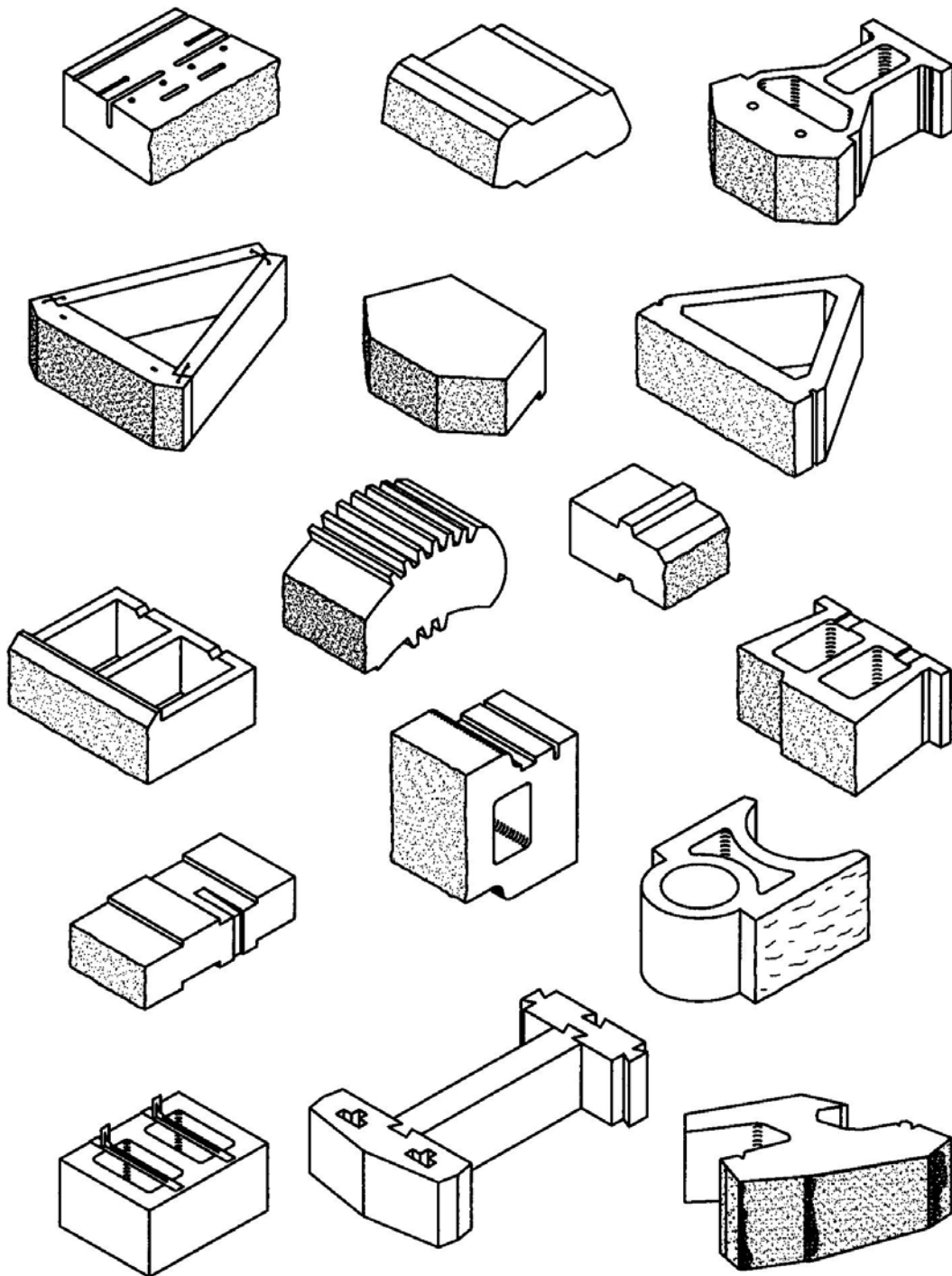


Figure 2-6. Example MSE wall facing treatments.
 (See Figure 5-33 and <http://gallery.company39.com/FLH/gallery/> for additional example facings.)



**NOTE: THE UNITS PRESENTED ARE PROPRIETARY AND/OR
PATENTED SYSTEMS**

Figure 2-7. Examples of commercially available MBW units (NCMA, 1997).

- **Gabion Facing.** Gabions (rock-filled wire baskets) can be used as MSE wall or RSS facing with reinforcing elements consisting of welded wire mesh, welded bar-mats, geogrids, geotextiles or the double-twisted woven mesh placed between or integrally manufactured with the gabion baskets. For example, this facing system is used by Maccaferri for their Terramesh® wall system.
- **Geosynthetic Facing.** Geosynthetic reinforcements are looped around at the facing to form the exposed face of the MSEW or RSS. These faces are susceptible to ultraviolet light degradation, vandalism, and damage due to fire. Geogrid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and fabric facing. Vegetation can grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance.
- **Post-construction Facing.** For wrapped faced walls, the facing – whether geotextile, geogrid, or wire mesh – can be attached after construction of the wall by shotcreting, guniting, cast-in-place concrete or by attaching prefabricated facing panels made of concrete, wood, or other materials. This multi-staging facing approach adds cost but is advantageous where significant settlement is anticipated.

Precast elements can be cast in several shapes and provided with facing textures to match environmental requirements and blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure.

Retaining structures with metal facings have the disadvantage of shorter life because of corrosion, unless provision is made to compensate for it. Facings using welded wire or gabions have the disadvantages of an uneven surface, exposed backfill materials, more tendency for erosion of the retained soil, possible shorter life from corrosion of the wires, and more susceptibility to vandalism. These disadvantages can, of course, be countered by providing shotcrete or by hanging facing panels on the exposed face and compensating for possible corrosion with galvanization and thicker wire. The greatest advantages of such facings are low cost, ease of installation, design flexibility, good drainage (depending on the type of wall fill) that provides increased stability, and possible treatment of the face for vegetative and other architectural effects. The facing can easily be adapted and well blended with natural country environment. These facings, as well as geosynthetic wrapped facings, are especially advantageous for construction of temporary or other structures with a short-term design life.

Dry cast segmental block MBW facings may raise some concerns as to durability in aggressive freeze-thaw environments where deicing salts are used. Recent research has shown that the MBW mix design must be specifically formulated to produce durable, freeze-thaw resistant units. Agencies should confirm locally manufactured units resistance with laboratory freeze-thaw testing. The current specifications in Chapter 10 have been developed to address this issue. Further, because the cement is not completely hydrated during the dry cast process, (as is often evidenced by efflorescence on the surface of units), a highly alkaline regime may establish itself at or near the face area, and may limit the use of some geosynthetic products as reinforcements.

The slope face of RSS structures is usually vegetated if approximately 1:1 or flatter. The vegetation requirements vary by geographic and climatic conditions and are therefore, project specific. Details are outlined in Section 10.5.

2.4.4 Reinforcement Types

Most, although not all, MSE wall systems with precast concrete panels use steel reinforcements that are typically galvanized. The two types of steel reinforcements currently in use with segmental panel faced MSE walls are:

1. Steel strips. The currently commercially available strips are ribbed top and bottom, 2 in. (50 mm) wide and 5/32-inch (4 mm) thick. Smooth strips 2- to 4³/₄-in. (60 to 120 mm) wide, 1/8 to 5/32-inch (3 to 4 mm) thick have been used.
2. Steel grids. Welded wire grid using two to six W7.5 to W24 longitudinal wire spaced at either 6 or 8 in. (150 or 200 mm). The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 9 to 24 in. (230 to 600 mm). Welded steel wire mesh spaced at 2 by 2-inch (50 by 50 mm) of thinner wire has been used in conjunction with a welded wire facing. Some MBW systems use steel grids with two longitudinal wires.

Most MBW systems use geosynthetic reinforcement, predominantly geogrids. The following soil reinforcement types are widely used and available:

3. High Density Polyethylene (HDPE) geogrid. These are of uniaxial manufacture and are available in up to 6 grades of strength. This type of reinforcement is also used with segmental panel facing.

4. PVC coated polyester (PET) geogrid. Available from a number of manufacturers. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. For longevity the PET is supplied as a high molecular weight fiber and is further characterized by a low carboxyl end group number.

Other types of soil reinforcements, and their applications, include:

5. Geotextiles. High strength geotextiles can be used principally in connection with reinforced soil slope (RSS) construction. Both polyester (PET) and polypropylene (PP) geotextiles have been used.
6. Double twisted steel mesh. The Terramesh[®] system by Maccaferri, Inc. uses a metallic, soft-temper, double twisted mesh soil reinforcement that is galvanized and then coated with poly vinyl chloride (PVC). This reinforcement is used for RSS and gabion faced MSE wall construction. Note that this reinforcement is classified as an *extensible* type of reinforcement due to its manufacturing geometry even though it is metallic.
7. Geosynthetic strap. Although not (currently) widely used, a geosynthetic strap type reinforcement has been used with segmental panel faced MSE walls. The strap consists of PET fibers encased in a polyethylene (PE) sheath.

2.4.5 Reinforced Fill Materials

MSEW Structures MSE walls require high quality wall fill for durability, good drainage, constructability, and good soil reinforcement interaction which can be obtained from well graded, granular materials. Many MSE systems depend on friction between the reinforcing elements and the soil. In such cases, a material with high friction characteristics is specified and required. Some systems rely on passive pressure on reinforcing elements, and, in those cases, the quality of reinforced wall fill is still critical. These performance requirements generally eliminate soils with high clay contents.

From a reinforcement capacity point of view, lower quality wall fills could be used for MSEW structures; however, a high quality granular wall fill has the advantages of better drainage, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement and compaction advantages in using granular soils. These include an increased rate of wall erection and improved maintenance of wall alignment tolerances. Appropriate use of lower quality reinforced fill and design considerations for its use is discussed in Chapter 3.

RSS Structures Reinforced Soil Slopes are normally not constructed with rigid facing elements. Slopes constructed with a flexible face can thus readily tolerate minor distortions that could result from settlement, freezing and thawing, or wetting-drying of the backfill. As a result, any soil meeting the requirements for embankment construction could be used in a reinforced slope system. However, a higher quality material offers fewer durability concerns for the reinforcement, and is easier to handle, place and compact, which speeds up construction.

2.4.6 Appurtenant Materials of Construction

Walls using precast concrete panels require bearing pads in their horizontal joints that provide some compressibility and movement between panels during elastic compression and settlement of the reinforced fill and preclude concrete-to-concrete contact. These materials are generally EPDM rubber or HDPE. The compressibility and thickness of the horizontal joint material should be a function of the wall height. Walls with heights greater than 50 ft (15 m) may require thicker or more compressible joints to accommodate the larger vertical loads due to the weight of panels in the lower third of the structure.

All joints of precast concrete panels are covered with a geotextile filter strip to prevent the migration of fines from the reinforced wall fill.

Bearing pads are not routinely used with MBW units. A zone of aggregate fill, usually 1-ft wide, is used behind the MBW units and within units with cores. This gravel readily compacted and conforms to the MBW unit. A filter is required between the gravel zone and wall fill, and can either be a soil filter or a geotextile filter (see Chapter 5).

2.5 CONSTRUCTION SEQUENCE

The following is an outline of the principal sequence of construction for MSEW and RSS. Specific systems, special appurtenances and specific project requirements may vary from the general sequence indicated.

2.5.1 Construction of MSEW systems with precast panel facings

The construction of MSEW systems with a precast panel facing is carried out as follows:

- **Preparation of subgrade.** This step involves removal of unsuitable materials from the area to be occupied by the retaining structure. All organic matter, vegetation, slide debris and other unstable materials should be stripped off and the subgrade compacted.

In unstable foundation areas, ground improvement methods, such as excavation and replacement, or dynamic compaction, stone columns, wick drains, etc. (see FHWA NHI-06-019 and NHI-06-020, Elias et al., 2006) would be constructed prior to wall erection.

- **Placement of a leveling pad for the erection of the facing elements.** This generally unreinforced concrete pad is often only 1 ft (300 mm) wide and 6 in. (150 mm) thick and is used for MSEW construction only, where concrete panels are subsequently erected. A wider concrete pad is recommended for MBW unit erection.

The purpose of this pad is to serve as a guide for facing panel erection and is not intended as a structural foundation support.

- **Erection of the first row of facing panels on the prepared leveling pad.** Facings may consist of either precast concrete panels or dry cast MBW units.

The first row of facing panels may be full, or half-height panels, depending upon the type of facing used. Only the first tier of panels must be braced to maintain stability and alignment. Subsequent rows of panels are simply wedged and clamped to adjacent panels. For construction with MBW units, full sized blocks are used throughout with no shoring.

The erection of facing panels and placement of the soil backfills should proceed simultaneously.

- **Placement and compaction of reinforced wall fill on the subgrade to the level of the first layer of reinforcement and its compaction.** The fill should be compacted to the specified density, usually 95 to 100 percent of AASHTO T-99 maximum density and within the specified range of optimum moisture content. Compaction moisture contents dry of optimum are recommended.

A key to good performance is *consistent* placement and compaction. Wall fill lift thickness must be controlled based on specification requirements and vertical distribution of reinforcement elements. The uniform loose lift thickness of the reinforced backfill should not exceed 12 in. (300 mm). Reinforced wall fill should be dumped into or parallel to the rear and middle of the reinforcement and bladed toward the front face.

Retained backfill placement and compaction behind the reinforced volume should proceed simultaneously.

- **Placement of the first layer of reinforcing elements on the wall fill.** The reinforcements are placed and connected to the facing panels, when the compacted fill has been brought up to the level of the connection. The reinforcements are generally placed perpendicular to back of the facing panels. More detailed construction control procedures associated with each construction step are outlined in Chapter 11.
- **Placement of the wall fill over the reinforcing elements to the level of the next reinforcement layer and compaction of the wall fill.** The previously outlined steps are repeated for each successive layer.
- **Construction of traffic barriers and copings.** This final construction sequence is undertaken after the final panels have been placed, and the wall fill has been completed to its final grade.

A complete sequence is illustrated in Figures 2-8 through 2-10.

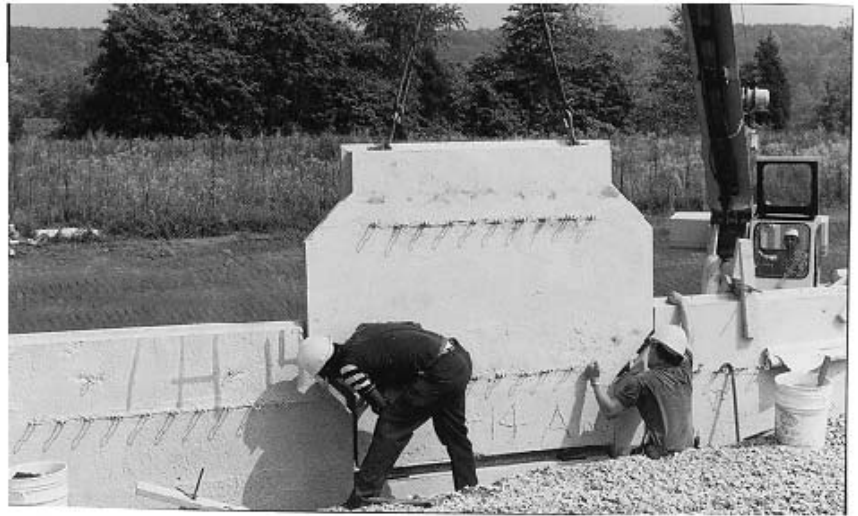


Figure 2-8. Erection of precast panels.



Figure 2-9. Fill spreading and reinforcement connection.



Figure 2-10. Compaction of the reinforced wall fill.

2.5.2 Construction of MSE systems with Flexible Facings

Construction of flexible-faced MSE walls, where the reinforcing material also serves as facing material, is similar to that for walls with precast facing elements. For flexible facing types such as welded wire mesh, geotextiles, geogrids or gabions, the erection of the first level of facing element requires only a level grade. A concrete footing or leveling pad is not usually required unless precast elements are to be attached to the system after construction.

Construction proceeds as outlined for segmental facings with the following exceptions:

- **Placement of first reinforcing layer.** Reinforcement with anisotropic strength properties (i.e., many geosynthetics) should be placed with the principal strength direction perpendicular to face of structure.

The reinforcement should be secured with retaining pins to prevent movement during reinforced fill placement.

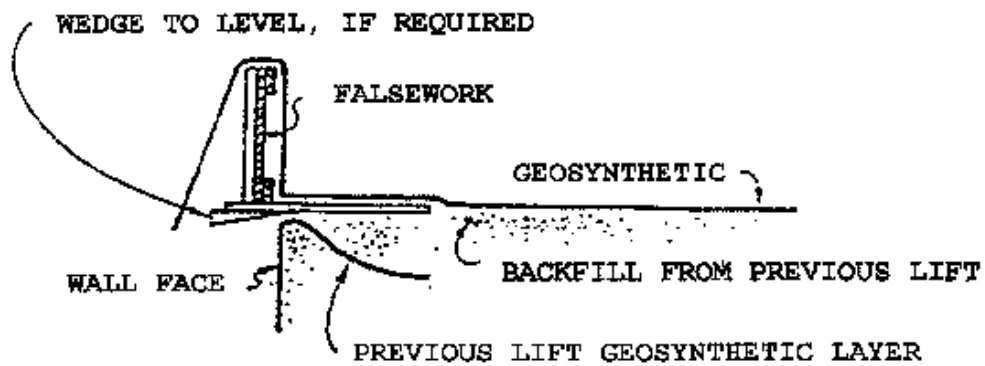
Adjacent sheets should be overlapped a minimum of 6 in. (150 mm) along the edges perpendicular to the face. Alternatively, with geogrid or wire mesh reinforcement, the edges may be butted and clipped or tied together.

- **Face Construction.** Place the geosynthetic layers using face forms as shown in Figure 2-11. For temporary support of forms at the face, form holders should be placed at the base of each layer at approximately 4 ft (1.20 m) horizontal intervals. Details of temporary formwork are shown in Figure 2-12. These supports are essential for achieving good compaction. When using geogrids or wire mesh, it may be necessary to use a geotextile or hardware cloth to retain the wall fill material at the face.

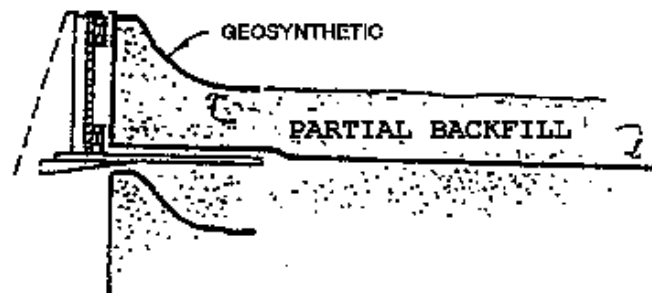
When compacting wall fill within 3 ft (~1 m) of the wall face, a hand-operated vibratory compactor is recommended.

The return-type method or successive layer tie method as shown in Figure 2-12 can be used for facing support. In the return method, the reinforcement is folded at the face over the wall fill material, with a minimum return length of 4 ft (1.25 m) to ensure adequate pullout resistance. Consistency in face construction and compaction is essential to produce a wrapped facing with satisfactory appearance.

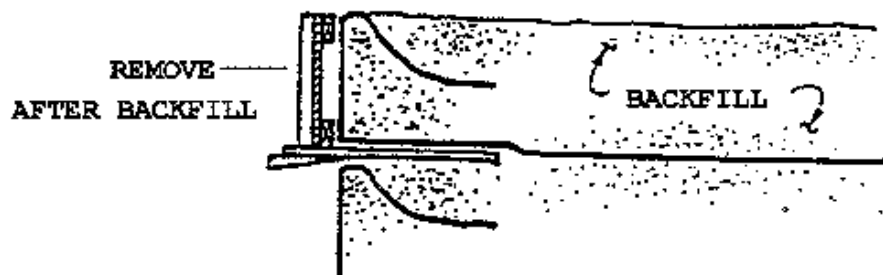
Apply facing treatment (shotcrete, precast facing panels, etc.). Some alternative facing systems for flexible faced walls and slopes are shown in Figure 2-13.



- (1) PLACE FALSEWORK AND GEOSYNTHETIC ON PREVIOUS LIFT

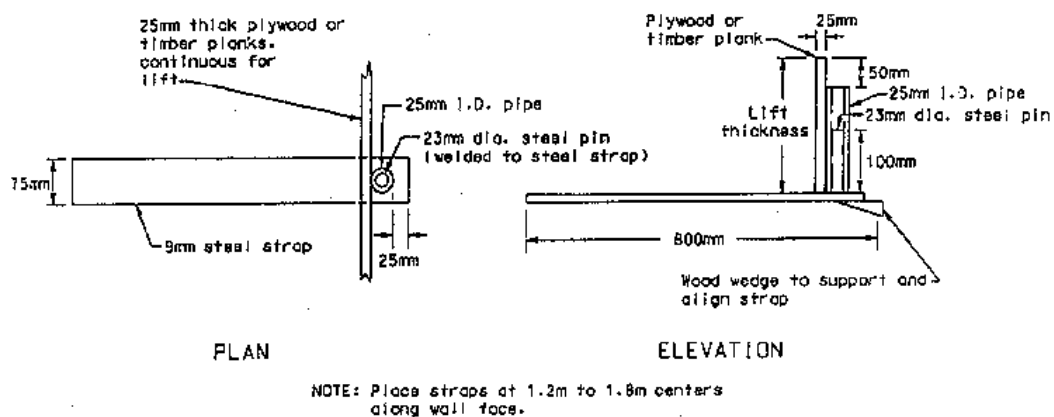


- (2) PLACE/COMPACT PARTIAL BACKFILL AND OVERLAP GEOSYNTH



- (3) PLACE/COMPACT REMAINDER OF BACKFILL LIFT

Figure 2-11. Lift construction sequence for geosynthetic faced MSE walls.



GEOTEXTILE TEMPORARY FORM SYSTEM DETAIL

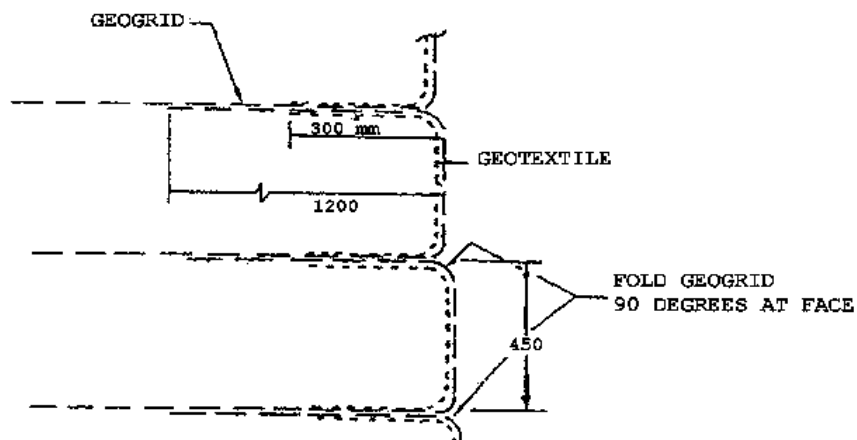
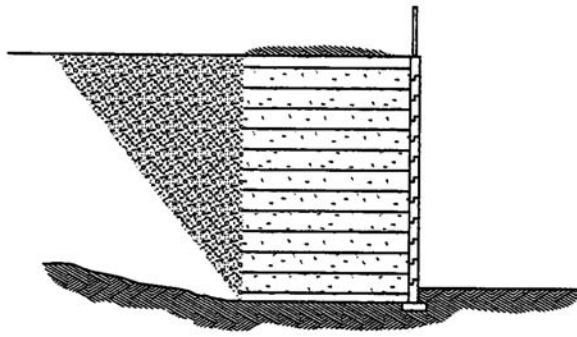
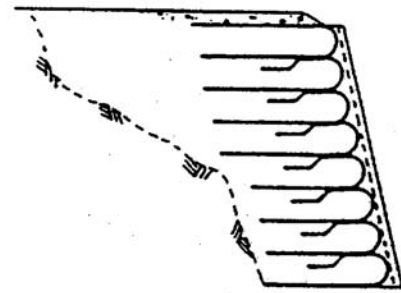


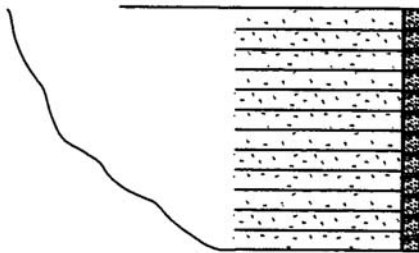
Figure 2-12. Typical geosynthetic face construction detail.
(25 mm = 1 in.)



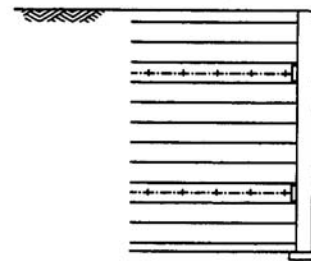
**Segmental
Precast Concrete**



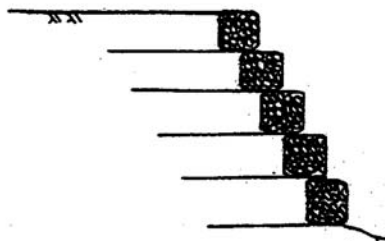
**Geotextile
Wrapped-Facing
with Shotcrete Cover**



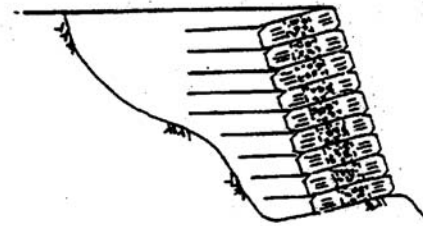
Modular Block Wall Units



Full-Height Concrete Panel



Gabion-Facing



Tire-Facing Units

Figure 2-13. Types of geosynthetic reinforced soil wall facing (after Wu, 1994).

2.5.3 RSS Construction

The construction of RSS embankments is considerably simpler and consists of many of the elements outlined for MSEW construction. They are summarized as follows:

- Site preparation.
- Construct subsurface drainage features.
- Place reinforcement layer.
- Place and compact backfill on reinforcement.
- Construct face. Details of the available methods are outlined in Chapter 8, construction.
- Place additional reinforcement and reinforced fill.
- Construct surface drainage features.

Key stages of construction are illustrated in Figure 2-14, and the complete sequence is fully outlined in Chapter 8.

2.6 SITE EVALUATION

2.6.1 Site Exploration

The feasibility of using an MSEW, RSS or any other type of earth retention system depends on the existing topography, subsurface conditions, and soil/rock properties. It is necessary to perform a comprehensive subsurface exploration program to evaluate site stability, settlement potential, need for drainage, etc., before repairing a slope or designing a new retaining wall or bridge abutment. Where the select backfill is to be obtained from on-site sources, the extent and quality must be fully explored to minimize contractor claims for changed conditions.

Subsurface investigations are required not only in the area of the construction but also behind and in front of the structure to assess overall performance behavior. The subsurface exploration program should be oriented not only towards obtaining all the information that could influence the design and stability of the final structure, but also to the conditions which prevail throughout the construction of the structure, such as the stability of temporary construction slopes that may be required.

The engineer's concerns include the bearing resistance of the foundation materials, the allowable deformations, and the stability of the structure. Necessary parameters for these analyses must be obtained.



Figure 2-14. Reinforced slope construction: (a) geogrid and fill placement; (b) soil filled erosion control mat placement; and (c) finished, vegetated 1:1 slope.

The cost of a reinforced soil structure is greatly dependent on the availability of the required type of reinforced fill and retained backfill materials. Therefore, investigations must be conducted to locate and test locally available materials that may be used for reinforced fill and retained backfill with the selected system.

2.6.2 Field Reconnaissance

Preliminary subsurface investigation or reconnaissance should consist of collecting any existing data relating to subsurface conditions and making a field visit to obtain data on:

- Limits and intervals for topographic cross sections.
- Access conditions for work forces and equipment.
- Surface drainage patterns, seepage, and vegetation characteristics.
- Surface geologic features, including rock outcrops and landforms, and existing cuts or excavations that may provide information on subsurface conditions.
- The extent, nature, and locations of existing or proposed below-grade utilities and substructures that may have an impact on the exploration or subsequent construction.
- Available right-of-way.
- Areas of potential instability such as deep deposits of weak cohesive and organic soils, slide debris, high groundwater table, bedrock outcrops, etc.

Reconnaissance should be performed by a geotechnical engineer or by an engineering geologist. Before the start of field exploration, any data available from previous subsurface investigations and those that can be inferred from geologic maps of the area should be studied. Topographic maps and aerial photographs, if available, should be studied. Much useful information of this type is available from the U.S. Geological Survey, the Natural Resources Conservation Service, the U.S. Department of Agriculture, and local planning boards or county offices.

2.6.3 Subsurface Exploration

The subsurface exploration program generally consists of soil soundings, borings, and test pits. The type and extent of the exploration should be decided after review of the preliminary data obtained from the field reconnaissance, and in consultation with a geotechnical engineer or an engineering geologist. The exploration must be sufficient to evaluate the geologic and subsurface profile in the area of construction. For guidance on the extent and type of required investigation, the FHWA NHI-01-031 Subsurface Investigations – Geotechnical Site Characterization reference manual (Mayne et al., 2002), should be reviewed.

The following guidelines are recommended (Christopher et al., 1990) for the subsurface exploration for potential MSE applications:

- Borings: The type (soil boring and/or cone penetration), number, location, and depth of investigation points generally are dictated by the project stage (i.e., feasibility study, preliminary, or final design), availability of existing geotechnical data, variability of subsurface conditions, length of the structure, what the structure supports, and other project details. Soil borings should be performed along the front and the back of the proposed reinforced soil structure. The width of the MSE wall or slope structure may be assumed as 0.8 times the anticipated height. Borings at the following intervals should be considered:
 - 100 ft (30 m) along the alignment of the reinforced soil structure; and
 - 150 ft (45 m) along the back of the reinforced soil structure
- The boring depth should be controlled by the general subsurface conditions. Where bedrock is encountered within a reasonable depth, rock cores should be obtained for a length of about 10 ft (3 m). This coring will be useful to distinguish between solid rock and boulders. Deeper coring may be necessary to better characterize rock slopes behind new retaining structures. In areas of soil profile, the borings should extend at least to a depth equal to twice the height of the wall/slope. If subsoil conditions within this depth are found to be weak and unsuitable for the anticipated pressures from the structure height, then the borings must be extended until reasonably strong soils are encountered.
- In each boring, soil samples should be obtained at 5-foot (1.5 m) depth intervals and at changes in strata for visual identification, classification, and laboratory testing. Methods of sampling may follow AASHTO T 206 or AASHTO T 207 (*Standard Penetration Test* and *Thin-Walled Shelby Tube Sampling*, respectively), depending on the type of soil. In granular soils, the Standard Penetration Test can be used to obtain disturbed samples. In cohesive soils, undisturbed samples should be obtained by thin-walled sampling procedures. In each boring, careful observation should be made for the prevailing water table, which should be observed not only at the time of sampling but also at later times to obtain a good record of prevailing water table conditions. If necessary, piezometers should be installed in a few borings to observe long-term water levels.
- Both the Standard Penetration Test and the Cone Penetration Test, ASTM D3441, provide data on the strengths and density of soils. In some situations, it may be desirable to perform in-situ tests using a dilatometer, pressuremeter, or similar means to determine soil modulus values.

- Adequate bulk samples of available soils should be obtained and evaluated as indicated in the following testing section to determine the suitability of the soil for use as backfill in the MSE structures. Such materials should be obtained from all areas from which preliminary reconnaissance indicates that borrow materials will be used.
- Test-pit explorations should be performed in areas showing instability or to explore further availability of the borrow materials for backfill. The locations and number of test pits should be decided for each specific site, based on the preliminary reconnaissance.

2.6.4 Laboratory Testing

Soil samples should be visually examined and appropriate tests performed for classification according to the Unified Soil Classification System (ASTM D2488). These tests permit the engineer to decide what further field or laboratory tests will best describe the engineering behavior of the soil at a given project site. Index testing includes determination of moisture content, Atterberg limits, and gradation. The dry unit weight of representative undisturbed samples should also be determined.

Shear strength determination by unconfined compression tests, direct shear tests, or triaxial tests will be needed for external stability analyses of MSE walls and slopes. At sites where compressible cohesive soils are encountered below the foundations of the MSE structure, it is necessary to perform consolidation tests to obtain parameters for performing service state settlement analyses. Both undrained and drained (effective stress) parameters should be obtained for cohesive soils, to permit evaluation of both long-term and short-term conditions.

Of particular significance in the evaluation of any material for possible use as backfill are the grain size distribution and plasticity. The effective particle size (D_{10}) can be used to estimate the permeability of cohesionless materials. Laboratory permeability tests may also be performed on representative samples compacted to the specified density. Additional testing should include direct shear tests on a few similarly prepared samples to determine shear strength parameters under long and short-term conditions. The compaction behavior of potential backfill materials should be investigated by performing laboratory moisture-density relationship tests according to AASHTO T 99, or T 180.

Properties to indicate the potential aggressiveness of the backfill material and the in-situ soils behind the reinforced soil zone must be measured. Tests include:

- pH (AASHTO T 289; ASTM D4972)
- Electrical resistivity.

- Salt content including water soluble sulfate (AASHTO T 290), sulfides (ASTM D4327), and chlorides (ASTM D4327).

The test results will provide necessary information for planning degradation protection measures and will help in the selection of reinforcement elements with adequate durability. designers.

2.6.5 Foundation Soils

The development and implementation of an adequate subsurface investigation program for the existing foundation conditions is a key element for ensuring successful project implementation. Causes for distress experienced in projects are often traced to inadequate subsurface exploration programs that did not disclose local or significant areas of soft soils, causing significant local differential settlement and distress to the facing panels. In a few documented extreme cases, such foundation weakness caused complete foundation failures leading to catastrophic collapses.

Determination of engineering properties for foundation soils should be focused on establishment of bearing resistance, global stability, settlement potential, and position of groundwater levels. For bearing capacity determinations, frictional and cohesive parameters (ϕ , c) as well as unit weights (γ_T) and groundwater position are normally required in order to calculate bearing resistance in accordance with Article 10.6.3.1 for soil and 10.6.3.2 for rock in AASHTO (2007). The effects of load inclination and footing shape may be omitted for Strength Limit State.

For foundation settlement determinations, the results of conventional settlement analyses with Service Limit State load factors, and using laboratory time-settlement data, coefficients of consolidation C_c , in conjunction with approximate value for compression index C_v , obtained from correlations to soil index tests (moisture content, Atterberg limits) should be used. The results of settlement analyses, especially with respect to differential settlement should be used to determine the ability of the facing and connection system to tolerate such movements or the necessity for special details or procedures to accommodate the differential movement anticipated.

Major foundation weakness and compressibility may require the consideration of ground improvement techniques to achieve adequate bearing capacity, or limiting total or differential settlement. Techniques successfully used, include surcharging with or without prefabricated vertical drains, stone columns, dynamic compaction, compaction grouting and the use of lightweight fill to reduce settlement. Additional information on ground improvement

techniques can be found in the FHWA Ground Improvement Manuals, FHWA NHI-06-019 and FHWA NHI-06-020 (Elias et al., 2006). As an alternate for MSE walls, faces constructed of geosynthetic wraps, welded wire mesh or gabion baskets, which will tolerate significant differential settlement, could be constructed and permanent facings such as concrete panels attached after the settlement has occurred, see Section 3.6.6. Of particular concern, are situations where the MSEW structure may terminate adjacent to a rigidly supported structure such as a pile supported abutment at the end of a retained approach fill.

Evaluation of these foundation related issues are typically beyond the scope of services provided by wall/slope system suppliers. Evaluations of this type are the responsibility of agency engineers or consultant geotechnical and are required before selection of the appropriate MSE wall or RSS system.

2.7 PROJECT EVALUATION

2.7.1 Structure Selection Factors

The major factors that influence the selection of an MSE/RSS alternative for any project include:

- Geologic and topographic conditions
- Environmental conditions
- Size and nature of the structure
- Aesthetics
- Durability considerations
- Performance criteria
- Availability of materials
- Experience with a particular system or application
- Cost

Many MSEW systems have proprietary features. Some companies provide services including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction assistance.

The various wall systems have different performance histories, and this sometimes creates difficulty in adequate technical evaluation. Some systems are more suitable for permanent walls, others are more suitable for low walls, and some are applicable for remote areas while others are more suited for urban areas. The selection of the most appropriate system will thus depend on the specific project requirements.

RSS embankments have been constructed with a variety of geosynthetic reinforcements and treatments of the slope face. These factors again may create an initial difficulty in adequate technical evaluation, but with the use of this manual easily addressed by department personnel to prepare generic designs. A number of geosynthetic reinforcement suppliers provide design services as well as technical assistance during construction.

Specific technical issues focused on selection factors are summarized in the following sections.

2.7.2 Geologic and Topographic Conditions

MSE structures are particularly well suited where a "fill-type" wall must be constructed or where side-hill fills are indicated. Under these latter conditions, the volume of excavation may be small, and the general economy of this type of construction is not jeopardized. Economic advantages diminish with large cut volumes to accommodate the reinforced soil structure, but in many instances remain viable.

The adequacy of the foundation to support the fill weight must be determined as a first-order feasibility evaluation.

Where soft compressible soils are encountered, preliminary stability analyses must be made to determine if sufficient shear strength is available to support the weight of the reinforced fill. As a rough first approximation for vertically faced MSE structures, the available shear strength must be equal to at least 2.0 to 2.5 times the weight of the fill structure. For RSS embankments the required foundation strength is somewhat less and dependent on the actual slope considered.

Where these conditions are not satisfied, ground improvement techniques (see FHWA NHI-06-019 and NHI-06-020, Elias et al., 2006) must be considered to increase the bearing capacity at the foundation level. These techniques include but are not limited to:

- Excavation and removal of soft soils and replacement with a compacted structural fill.
- Use of lightweight fill materials.
- In-situ densification by dynamic compaction or improvement by use of surcharging with or without prefabricated vertical drains.
- Construction of aggregate columns.

Where marginal to adequate foundation strength is available, preliminary settlement analyses should be made to determine the potential for differential settlement, both longitudinally along a proposed structure as well as transverse to the face. This second-order feasibility

evaluation is useful in determining the appropriate type of facing systems for MSE walls and in planning appropriate construction phasing to accommodate the settlement.

In general, concrete-faced MSE structures using discrete articulating panels can accommodate maximum longitudinal differential settlements of about 1/100, without the introduction of special sliding joints between panels. Full-height concrete panels are considerably less tolerant and generally should not be considered where differential settlements are anticipated. MBW unit faced walls can accommodate maximum longitudinal differential settlements of about 1/200, with the introduction of special slip joints.

The performance of reinforced soil slopes generally is not affected by differential longitudinal settlements.

2.7.3 Environmental Conditions

The primary environmental condition affecting reinforcement type selection and potential performance of MSE structures is the aggressiveness of the in-situ ground regime that can cause deterioration to the reinforcement. Post construction changes must be considered where deicing salts or fertilizers are subsequently used.

For steel reinforcements, in-situ regimes containing chloride and sulfate salts generally in excess of 200 PPM accelerate the corrosive process as do acidic regimes characterized by a pH of less than 5 (Elias, 1989). Alkaline regimes characterized by pH > 10 will cause accelerated loss of galvanization.

Certain in-situ regimes have been identified as being potentially aggressive for geosynthetic reinforcements. Polyester (PET) degrades in highly alkaline or acidic regimes. Polyolefins appear to degrade only under certain highly acidic conditions.

For additional specific discussions on the potential degradability of reinforcements, refer to the companion Corrosion/Degradation reference manual and are summarized in Section 3.5.

A secondary environmental issue is site accessibility, which may dictate the nature and size of the facing for MSE wall construction. Sites with poor accessibility or remote locations may lend themselves to lightweight facings such as geotextile or geogrid wrapped facings and vegetative covers; metal skins; welded wire mesh, gabions, modular blocks (MBW) which could be erected without heavy lifting equipment.

RSS construction with an organic vegetative cover must be carefully chosen to be consistent with native perennial cover that would establish itself quickly and would thrive with available site rainfall.

2.7.4 Size and Nature of Structure

Theoretically there is no upper limit to the height of MSE wall that can be constructed. Structures up to 135 ft (41 m) have been successfully constructed in the U.S. with steel reinforcements, although such heights for transportation-related structures are rare. RSS embankments have been constructed to up to a height of 242 ft (74 m) in the U.S. with geogrid reinforcements, but again such heights for transportation-related structures are rare.

Practical limits are often dictated by economy, available ROW, and the tensile strength of commercially available soil reinforcing materials. For bridge abutments there is no theoretical limit to the span length that can be supported, although the longer the span, the greater is the area of footing necessary to support the beams. Since the nominal bearing resistance of the reinforced fill for the service limit state is usually limited to 4000 psf (200 kPa), a large abutment footing further increases the span length, adding cost to the superstructure. This additional cost must be balanced by the potential savings of the MSE alternate to a conventional abutment wall, which would have a shorter span length. As an option in such cases, it might be economical to consider support of the bridge beams on deep foundations, placed within (or in front of) the reinforced fill zone.

The lower limit to height is usually dictated by economy. When used with traffic barriers, low walls on good foundations of less than 10 to 14 ft (3 to 4 m) are often uneconomical, as the cost of the overturning moment leg of the traffic barrier approaches one-third of the total cost of the MSE structure in place. For cantilever walls, the barrier is simply an extension of the stem with a smaller impact on overall cost.

The total size of structure (square feet of face) has little impact on economy compared with other retaining wall types. However, the unit cost for small projects of less than 3,000 ft² (300 m²) is likely to be 10 to 15 percent higher.

RSS may be cost effective in rural environments, where ROW restrictions exist or on widening projects where long sliver fills are necessary. In urban environments, they should be considered where ROW is available, as they are generally more economical than vertically faced MSE wall structures.

2.7.5 Aesthetics

Precast concrete facing panels may be cast with an unlimited variety of texture and color for an additional premium that seldom exceeds 15 percent of the facing cost, which on average would mean a 4 to 6 percent increase on total in-place cost.

Modular block wall facings are often comparable in cost to precast concrete panels except on small projects (less than 4,000 ft² {400 m²}) where the small size introduces savings in erection equipment cost and the need to cast special, made-to-order concrete panels to fit what is often irregular geometry. MBW facings may be manufactured in color and with a wide variety of surface finishes.

The outward face treatment of RSS, generally is by vegetation, which is initially more economical than the concrete facing used for MSE structures. However, maintenance costs may be considerably higher, and the long-term performance of many outward face treatments has not been established.

2.7.6 Questionable Applications

The current AASHTO LRFD Specifications (2007) states that MSE walls should not be used under the following conditions:

- When utilities other than highway drainage must be constructed within the reinforced zone where future access for repair would require the reinforcement layers to be cut. A similar limitation should be considered for RSS structures.
- With galvanized metallic reinforcements exposed to surface or ground water contaminated by acid mine drainage or other industrial pollutants as indicted by low pH and high chlorides and sulfates.
- When floodplain erosion may undermine the reinforced fill zone, or where the depth to scour cannot be reliably determined.

2.8 ESTABLISHMENT OF PROJECT CRITERIA

The engineer should consider each topic area presented in this section at a preliminary design stage and determine appropriate elements and performance criteria.

The process consists of the following successive steps:

- Consider all possible alternatives.
- Choose a system (MSEW or RSS).
- Consider facing options.
- Develop performance criteria (loads, design heights, embedment, settlement tolerances, foundation capacity, effect on adjoining structures, etc.).
- Consider effect of site on corrosion/degradation of reinforcements.

2.8.1 Alternates

Cantilever, gravity, semi gravity, or counterforted concrete walls or soil embankments are the usual alternatives to MSE walls and abutments and RSS.

In cut situations, in-situ walls such as tieback anchored walls, soil nailed walls or nongravity cantilevered walls are often more economical, although where limited ROW is available, a combination of a temporary in-situ wall at the back end of the reinforcement and a permanent MSE wall is often competitive.

For waterfront or marine wall applications, sheetpile walls with or without anchorages or prefabricated concrete bin walls that can be constructed in the wet are often, if not always, both more economical and more practical to construct.

2.8.2 Facing Considerations

The development of project-specific aesthetic criteria is principally focused on the type, size, and texture of the facing, which is the only visible feature of any MSE structure.

For permanent applications, considerations should be given to MSE walls with precast concrete panels. They are constructed with a (near) vertical face. Currently, the size of panels commercially produced varies from 20 to 50 ft² (1.8 to 4.5 m²). Generally, full height panels may be considered for walls up to about 14 to 16 ft (4 to 5 m) in height on foundations that are not expected to settle. Experienced contractors have successfully constructed taller full height panels (e.g., 25 ft {7.5 m}) on competent foundations. The precast concrete panels can be manufactured with a variety of surface textures and geometries, as shown in Figure 2-6.

For permanent applications, considerations should be given to MBW facings, which are available in a variety of shapes and textures as shown in Figure 2-7. They range in facial area from 0.5 to 1 ft² (0.05 to 0.1 m²). An integral feature of this type of facing is a front

batter ranging from nominal up to 15 degrees. Project geometric constraints, i.e., the bottom of wall and top of wall horizontal limits, may limit the amount of permissible batter and, thus, the types of MBW units that may be used. Note that the toe of these walls step back as the foundation elevation steps up, due to the stacking arrangement and automatic batter.

Other facing options are gabion, timber faced, or vegetated.

For temporary walls, significant economy can be achieved with geosynthetic wrapped facings. They may be made permanent by applying gunite or cast-in-place concrete in a post-construction application.

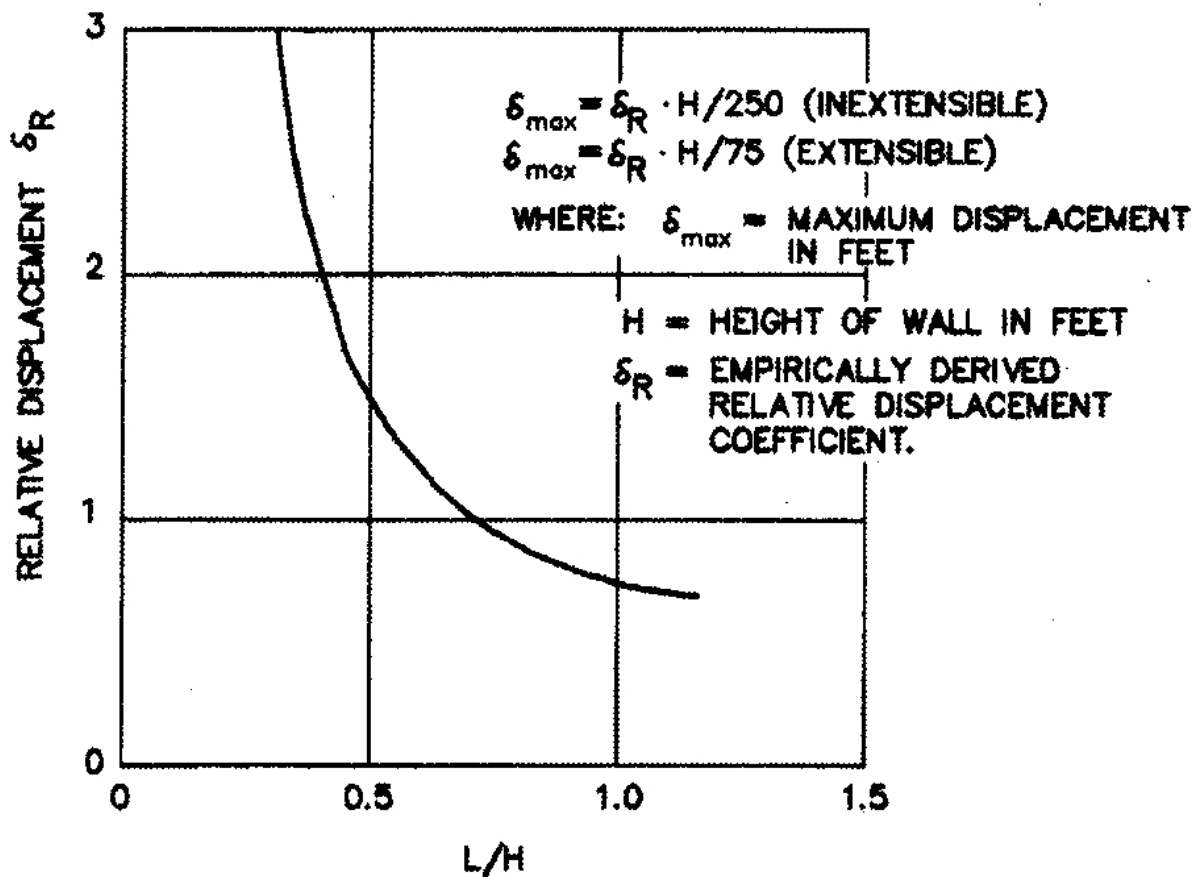
For RSS structures, the choice of slope facing may be controlled by climatic and regional factors. For structures of less than 33 ft (10 m) height with slopes of approximately 1 Horizontal:1 Vertical (1H:1V) or flatter, a vegetative "green slope" can be usually constructed using an erosion control mat or mesh and local grasses. Where vegetation cannot be successfully established and/or significant run-off may occur, armored slopes using natural or manufactured materials may be the only choice to reduce future maintenance. For additional guidance see Section 8.5.

2.8.3 Performance Criteria

Performance criteria for MSE structures with respect to design requirements are governed by design practice or codes such as contained in Article 11.10 of 2007 AASHTO LRFD Specifications for Highway Bridges. These requirements consider load and resistance factors with respect to various failure modes and materials, and for various limit states. No specific AASHTO guidance is presently available for RSS structures.

With respect to lateral wall displacements, no method is presently available to definitively predict lateral displacements, most of which occur during construction. The horizontal movements depend on compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-panel connection details, and details of the facing system. A rough estimate of probable lateral displacements of simple structures that may occur during construction can be made based on the reinforcement length to wall-height ratio and reinforcement extensibility as shown in Figure 2-15, for the serviceability limit check.

This figure indicates that increasing the length-to-height ratio of reinforcements from its theoretical lower limit of 0.5H to 0.7H, decreases the deformation by 50 percent.



For $L = 0.7 H$

Metallic (inextensible) reinforcement $\approx 3/4$ -in. per 10 ft of wall height

Geogrid (moderately extensible) reinforcement ≈ 1 in. per 10 ft of wall height

Geotextile (extensible) reinforcement ≈ 1.5 in. per 10 ft of wall height

Based on 20 ft high walls, relative displacement increases approximately 25% for every 400 psf surcharge. Experience indicates that for higher walls, the surcharge effect may be greater.

NOTE: This figure is only a guide. Actual displacement will depend, in addition to the parameters addressed in the figure, on soil characteristics, compaction effort, and contractor

Figure 2-15. Empirical curve for estimating lateral displacement during construction for MSE walls (after FHWA RD 89-043 {Christopher et al., 1990}).

Performance criteria are both site and structure-dependent. Structure-dependent criteria consist of safety factors or a consistent set of load and resistance factors as well as tolerable movement criteria of the specific MSE structure selected.

Recommended MSE Wall load and resistance factors with respect to the various potential failure modes and limit states are presented in Chapter 4.

A number of site-specific project criteria need to be established at the inception of design:

- **Design limits and wall height.** The length and height required to meet project geometric requirements must be established to determine the type of structure and external loading configurations.
- **Alignment limits.** The horizontal (perpendicular to wall face) limits of bottom and top of wall alignment must be established as alignments vary with batter of wall system. The alignment constraints may limit the type and maximum batter of the wall facing, particularly with MBW units.
- **Length of reinforcement.** A minimum reinforcement length of $0.7H$ is recommended for MSE walls. Longer lengths are required for structures subject to surcharge loads, or where foundation conditions affect lateral sliding and/or global/compound slope stability, as listed in Table 2-1. Shorter lengths can be used in special situations (see Chapter 6).
- **External loads.** The external loads may be soil surcharges required by the geometry, adjoining footing loads, loads as from traffic, and/or traffic impact loads. The magnitude of the minimum traffic loads outlined in Article 3.11.6.4 (AASHTO, 2007) is a uniform load equivalent to 2 ft (0.6 m) of soil over the traffic lanes. The traffic load is greater for some cases (see Tables 4-5 and 4-6).
- **Wall embedment.** The minimum embedment depth for walls from adjoining finished grade to the top of the leveling pad should be based on bearing, settlement, and slope stability considerations. Current practice based on local bearing considerations, recommends the minimum embedment depths listed in Table 2-1.

Table 2-1. Typical Minimum Length of Reinforcement.

Case	Typical Minimum L/H Ratio
Static loading with or with traffic surcharge	0.7
Sloping backfill surcharge	0.8
Seismic loading	0.8 to 1.1

Table 2-2. Minimum MSEW Embedment Depths.

Slope in Front of Wall	Minimum Embedment Depth to Top of Leveling Pad*
All Geometries	2 ft minimum
horizontal (walls)	H/20
horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
1.5H:1V	H/5
* Minimum depth is the greater of applicable values listed, frost depth, or scour depth.	

Larger values may be required, depending on shrinkage and swelling of foundation soils, seismic activity, and/or scour. A greater embedment depth may also be required based upon bearing, settlement, and/or global stability calculations. As noted, the minimum in any case is 2 ft (0.6 m), except for structures founded on rock at the surface, where no embedment may be used. Alternately, frost-susceptible soils could be overexcavated and replaced with non-frost susceptible fill, hence reducing the embedment depth (and overall wall height).

A minimum horizontal bench 4-ft (1.2 m) wide as measured from the face shall be provided in front of walls founded on slopes. The bench may be formed or the slope continued above that level (11.10.2.2, AASHTO {2007}), as illustrated in Figure 2-16. The horizontal bench is intended to provide resistance against general bearing failure and to provide access for maintenance inspections (C11.10.2.2, AASHTO {2007}).

For walls constructed along rivers and streams where the depth of scour has been reliably determined, a minimum embedment of 2 ft (0.6 m) below scour depth is recommended.

Embedment is not required for RSS unless dictated by stability requirements.

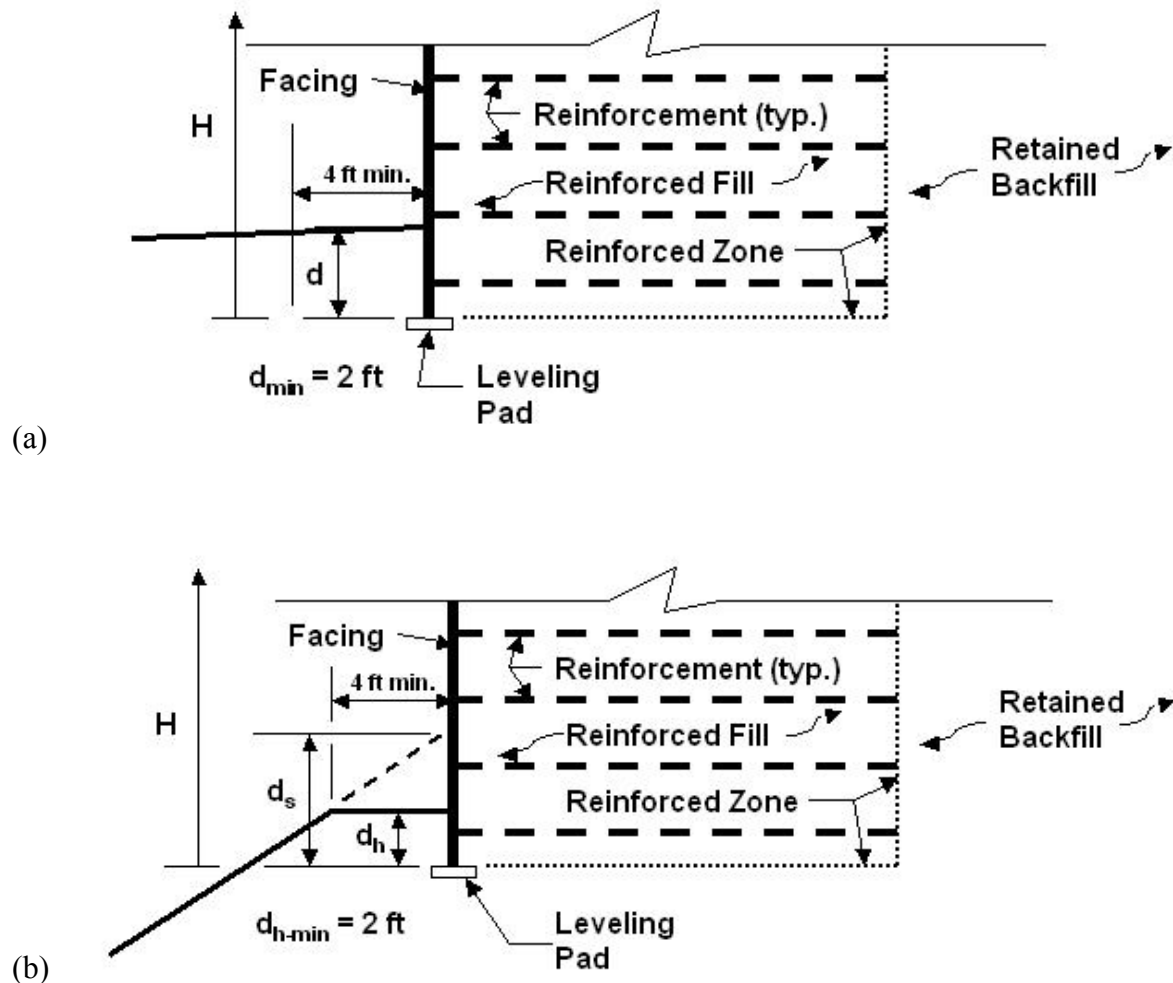


Figure 2-16. MSE wall embedment depth requirements, (a) level toe condition and (b) benched slope toe condition (d_h = minimum depth for horizontal slope and d_s = minimum depth for sloping toe, from Table 2-2).

- **Seismic Activity.** Due to their flexibility, MSE wall and slope structures are quite resistant to dynamic forces developed during a seismic event, as confirmed by the excellent performance in several recent earthquakes.

Seismic loading analysis of MSE walls is an Extreme Event limit state. Psuedo-static analysis procedures for seismic stability are presented in Chapter 7. Note that for sites

where the anticipated ground acceleration is greater than 0.29 g, significant total lateral structure movements may occur, and a deformation analysis for the structure is recommended (C11.10.7.1, AASHTO {2007}).

MSE walls should be designed/checked for seismic stability on all sites where the A_s coefficient is greater than 0.05. For RSS structures, seismic analyses should be included regardless of acceleration magnitude.

- **Tolerance of precast facing panels to settlement.** MSE structures have significant deformation tolerance both longitudinally along a wall and perpendicular to the front face. Therefore, poor foundation conditions seldom preclude their use. However, where significant differential settlements are anticipated (greater than 1/100) sufficient joint width and/or slip joints must be provided to preclude panel cracking. This factor may influence the type and design of the facing panel selected.

Square panels generally adapt to larger longitudinal differential settlements better than long rectangular panels of the same surface area. A joint width of $\frac{3}{4}$ -inch (20 mm) is generally recommended. Guidance on differential settlements that can be tolerated is presented in Table 2-3, for panels with a surface of 30 ft² (2.8 m²) or less and for panels with surface area greater than 30 ft² (2.8 m²) and less than or equal to 75 ft² (7 m²).

Bearing pads used between segmental precast concrete panels should be designed to accommodate downdrag forces on it due to elastic settlement of the wall fill. Bearing pad design and specification are addressed in Section 3.6.1.a and Section 10.5, respectively.

MSE walls constructed with full height panels should be limited to differential settlements of 1/500. Walls with drycast facing (MBW) should be limited to settlements of 1/200. For walls with welded wire facings, the limiting differential settlement should be 1/50.

Where significant differential settlement perpendicular to the wall face is anticipated, the reinforcement connection may be overstressed. Where the back of the reinforced soil zone will settle more than the face, the reinforcement could be placed on a sloping fill surface which is higher at the back end of the reinforcement to compensate for the greater vertical settlement. This may be the case where a steep surcharge slope is constructed. This latter construction technique, however, requires that surface drainage be carefully controlled after each day's construction. Alternatively, where significant differential settlements are anticipated, ground improvement techniques may be warranted to limit the settlements.

Table 2-3. Relationship Between Joint Width and Limiting Differential Settlements for MSE Precast Panels (after C11.10.4.1 AASHTO {2007}).

Joint Width	Limiting Differential Settlement	
	Area $\leq 30 \text{ ft}^2$	$30 \text{ ft}^2 < \text{Area} \leq 75 \text{ ft}^2$
$\frac{3}{4}$ -in. (20 mm)	1/100	1/200

2.8.4 Design Life

MSE walls should be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the wall. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less.

A greater level of safety and/or longer service life (i.e., 100 years) may be appropriate for walls that support true bridge abutments, buildings, critical utilities, or other facilities for which the consequences of poor performance or failure would be severe.

The quality of in-service performance is an important consideration in the design of permanent retaining walls. Permanent walls shall be designed to retain an aesthetically pleasing appearance, and not require significant maintenance throughout their design service life.

For RSS structures, similar minimum design life ranges should be adopted.

2.9 PROPRIETARY ASPECTS

The distinguishing characteristics of MSE trademarked systems from generic systems are patented features or materials of construction.

At present the following significant components are known to be covered by unexpired patents:

- Connection details between grid reinforcement and precast panel covered by a number of patents issued to various suppliers. In general, these patents cover a specific design for the concrete-embedded portion of connecting member only.
- Most MBW facing units are covered by recent design patents.

A number of patents may be in force for specific MSE construction methods under water, specific types of traffic barriers constructed over MSE walls, and facing attachments to temporary facings.

2.10 CASE HISTORIES – MSE WALLS

2.10.1 Mn/DOT Crosstown Project MSE Walls

MSE walls were used extensively on the Crosstown Project, located in Minneapolis, Minnesota. The walls are used to widen existing roadways, construct new ramps, and construct new bridge approaches. The project enlarges and streamlines the I35W and Mn Hwy 62 interchange. This is a heavily traveled roadway in a congested urban area. Several bridges were widened and several new bridges were constructed as part of this project. The detailed wall designs and the wall components were supplied by a Minnesota Department of Transportation (Mn/DOT) pre-approved MSE wall vendor. Design followed the ASD method (AASHTO 17th Edition, 2002). Most of the walls were constructed during the 2008 and 2009 construction seasons.

Approximately 300,000 ft² (28,000 m²) of MSE walls were constructed in 24 separate walls. Typical wall heights are approximately 25 to 30 ft (7.5 to 9 m), and the maximum wall height is 45 ft (14 m). The MSE walls are faced with architectural segmental precast panels and reinforced with steel bar mats. Facing panels are painted after wall construction. The architectural relief included false columns, on the long and tall walls, as shown in Figure 2-17. The reinforced wall fill is an angular, well-graded sand. The walls are designed for a 100-year life. Many walls have traffic barriers on top of the reinforced zoned. The barriers were designed by the Mn/DOT project design consultant. A geomembrane was specified and installed across the top of the reinforced zones to prevent, or minimize, infiltration of de-icing salt runoff into the reinforced fill.

Temporary welded wire mesh (WWM) faced walls were also used on this project for temporary bridge abutments (see Figure 2-2), bridge approach embankments, and

construction staging. These walls were also designed and supplied by the same Mn/DOT pre-approved vendor.

The use of MSE walls on this project provided a relatively rapid means of wall construction, and produced structures designed for a 100-year life. Cast-in-place, concrete cantilever walls were also extensively used on this project. Approximately 500,000 ft² (49,000 m²) of C.I.P. walls were constructed.

The cost of these MSE walls was \$30.50 per ft² (\$330 per m²) of face area, plus \$17.00 per yd³ (\$20 per m³) for the reinforced wall fill and \$375.00 per yd³ (\$450 per m³) of concrete for the traffic barrier moment slab. MSE wall cost with the select granular wall fill and moment slab was approximately \$54.50 per ft² (\$585 per m²) of face area. The MSE wall costs do not include the traffic barrier and noise wall. The cost of the cast-in-place walls on the project was \$67.20 per ft² (\$723 per m²) of face area plus cost of backfill, for an approximate total cost of \$76 per ft² (\$820 per m²) of face area.



Figure 2-17. MSE wall construction on Mn/DOT Crosstown Project, 2008.

2.10.2 Veterans Memorial Overpass True Abutment MSE Walls

The Pima County Department of Transportation's Veterans Memorial Overpass (VMO) project is located in Tucson, Arizona. The project included a 5-span, 348-ft long bridge (shown in Figure 2-18) that takes Palo Verde Road over Aviation Parkway, Union Pacific railroad assembly area tracks, and 36th Street. The North and South abutments consist of spread footings on top of 25-ft and 35-ft high MSE walls, respectively. MSE walls were proposed as part of value engineering proposal to replace cast-in-place walls. The walls were designed using the ASD method (AASHTO 17th Edition, 2002). The walls were constructed in 2004 and 2005.

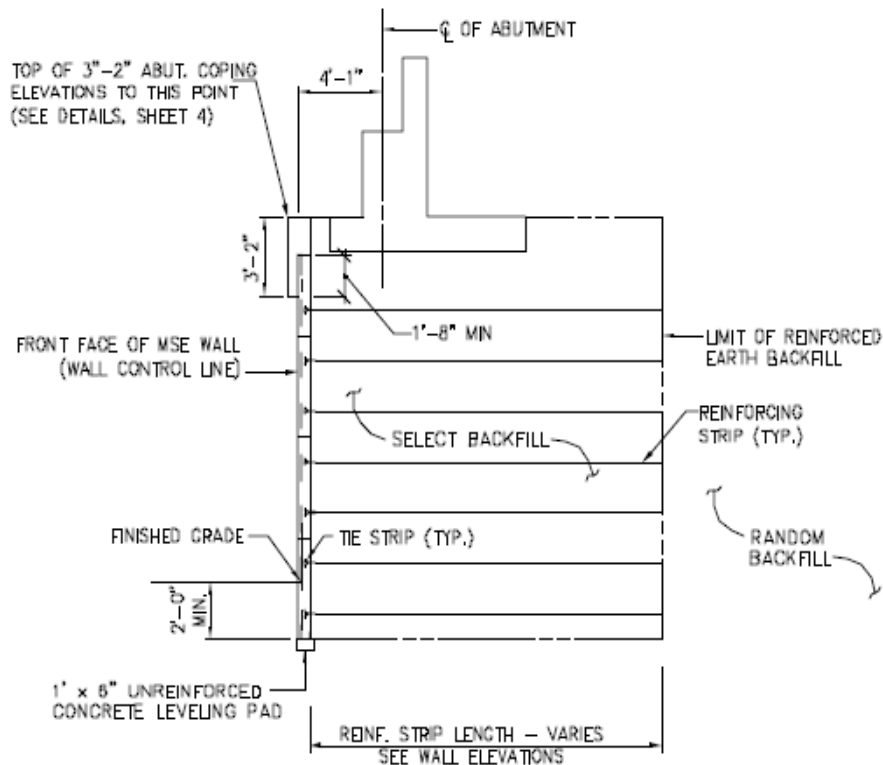
An abutment is shown in Figure 2-19, and abutment cross section is illustrated in Figure 2-20. Both abutments are 150 ft long. The bridge consists of simply supported AASHTO Type III girders on elastomeric bearing pads resting on an abutment footing. The bridge footing at each abutment is 10.75 ft wide and 10.2 ft high. Clearance between the back of the coping and the toe of the footing is 6 in. The length of reinforcements was equal to the height of the abutment. Reinforced fill was a select granular fill, in accordance with AASHTO/FHWA requirements. Ribbed steel reinforcing strips were used for soil reinforcements, with 5-ft tall x 10-ft wide precast concrete segmental panels.

The upper 9 ft of foundation soils were over-excavated and replaced with engineered fill because they were loose and potentially collapsible. Underlying soils were dense to very dense clayey sands with refusal N-values. Groundwater depth is greater than 150 ft.

This project was monitored with over 500 survey points. Settlements of less than 1 in., primarily occurring during construction, were measured. No noticeable post-construction settlement has been observed.



Figure 2-18. Veterans Memorial Overpass. Figure 2-19. MSE true bridge abutment.



TYPICAL SECTION AT ABUTMENT

SCALE: N.T.S.

Figure 2-20. Typical cross section, VMO project.

2.10.3 SeaTac Airport Runway Extension MSE Wall

The tallest MSE wall in the U.S. to date has an exposed height of 138 ft (42 m) at its tallest section and was constructed to limit encroachment on adjacent creeks and wetlands and increase the land use area for the Third Runway project at SeaTac Airport (Figure 2-21). The West MSE wall was one of several walls constructed for the runway extension and is approximately 1430 ft (436 m) long, has four tiers formed by 8 ft (2.4 m) setbacks, and had a constructed height of 150 ft (148 m). The wall supports 20 ft (6.1 m) high, unreinforced 2H:1V slope. The MSE wall used steel reinforcing strips with concrete facing panels. In the lower tier, up to 25, 2 in. (50 mm) wide by 0.24 in. (6 mm) thick strips with a length of 116 ft (35.4 m) were connected to the 5 by 5-ft (1.5 by 1.5-m) panels. A full discussion of project background, design aspects, and instrumentation of these MSE walls are provided by Sankey et al. (2007) and Stuedlein et al. (2007).



Figure 2-21. SeaTac Airport runway extension MSE wall.

2.10.4 Guanella Pass Roadway Reconstruction

The Guanella Pass project is located in the Front Range of the Colorado Rocky Mountains within the Pike and Arapaho National Forests approximately 50 miles west of Denver. The overall goal of the Guanella Pass Road improvement project is to balance transportation requirements and roadway maintenance within a sensitive human and natural environment. The 24-mile long route connects two principal east-west corridors, US 285 and Interstate Highway 70. The original roadway supported two-way traffic with various widths and many sharp switchbacks in very steep mountainous terrain.

Twenty-one wired-faced MSE walls, extending a total length of almost 12,000 feet, were constructed along the project to gain adequate roadway width (typical wall is shown in Figure 2-22). Alternately, very long and steep down-slope embankments could have been constructed, but would have significantly impacted the forest. The MSE walls were able to

limit the width of disturbance to a little more than the roadway width. Some of the walls, which were visible from the road, were faced with an architectural concrete cast-in-place facing (as shown in Figure 2-23). The facing form liner was specifically designed by the Forest Service to simulate a more natural rockery type of wall, which was used extensively on the cut side of the road.

Due to the site geometry, these walls were constructed on very steep slopes ranging from 1.3H:1V to 1.5H:1V. MSE retaining walls for this project were evaluated for global stability using limit equilibrium methods using a minimum factor of safety for global stability of 1.3. In order to achieve this factor of safety under these geometric and loading conditions, the designers needed to work with each site individually. The reinforcement lengths were longer than typically used on MSE walls; they ranged from 70% to 120% of the wall height. In addition, the designers were able to vary the wall embedment below finished grade. These two parameters were used to provide a stable structure to support the new roadway.

In addition, the project was able to utilize the on-site soils for the reinforced backfill. The reinforced backfill met most of the AASHTO and FHWA requirements with the exception of the No. 200 sieve. In order to be able to use most of the soils excavated during construction, the project specifications allowed the use of up to 20% passing the No. 200 sieve instead of 15%. This was a significant savings to the project since it would have been difficult to waste the excavated material within the construction limits and it would have been very costly to import material for the walls, since the project was so remote.

MSE walls were selected for this project primarily due to their ease of construction and flexibility in difficult terrain and remote sites.



Figure 2-22. Guanella Pass wire-faced wall.



Figure 2-23. Guanella Pass architectural concrete faced wall.

CHAPTER 3

SOIL REINFORCEMENT PRINCIPLES AND SYSTEM DESIGN PROPERTIES

This chapter outlines the fundamental soil reinforcement principles that governs structure behavior, and develops system design parameters which are used for specific MSE wall and RSS design, detailed in Chapters 4, 6, 7, and 9.

The objectives of this chapter are to develop:

- An understanding of soil-reinforcement interaction.
- Introduce normalized pullout capacity concepts.
- Develop design soil parameters for select reinforced fill, retained backfill and foundation bearing capacity.
- Establish structural design properties.

3.1 OVERVIEW

As discussed in Chapter 2, mechanically stabilized earth systems (MSEW and RSS) have three major components: reinforcing elements, facing system, and reinforced fill. Reinforcing elements may be classified by stress/strain behavior and geometry. In terms of stress/strain behavior, reinforcing elements may be considered inextensible (metallic) or extensible (polymeric). This division is not strictly correct because some newer glass-fiber reinforced composites and ultra-high-modulus polymers have moduli that approach that of mild steel. Likewise, certain metallic woven wire mesh reinforcements, such as hexagon gabion material, have a structure that will deform more than the soil at failure and are thus considered extensible. Based on their geometric shapes, reinforcements can be categorized as strips, grids or sheets. Facing elements, when employed, can be precast concrete panels or modular blocks, gabions, welded wire mesh, cast-in-place concrete, timber, shotcrete, vegetation, or geosynthetic material. Reinforced fill refers to the soil material placed within the zone of reinforcement. The retained soil refers to the material, placed or in-situ, directly adjacent to the reinforced fill zone. The retained soil is the source of earth pressures that the reinforced zone must resist. A drainage system below and behind the reinforced fill is also an important component, especially when using poorly draining backfill.

3.2 ESTABLISHMENT OF REINFORCED AND RETAINED FILL ENGINEERING PROPERTIES

3.2.1 Reinforced Fill Soil

The selection criteria of reinforced fill should consider long-term performance of the completed structure, construction phase stability and the degradation environment created for the reinforcements. Much of engineering communities' knowledge and experience with MSE wall structures to date has been with select, cohesionless backfill. Hence, knowledge about internal stress distribution, pullout resistance, and failure surface shape is constrained and influenced by the unique engineering properties of these soil types. Granular soils are ideally suited to MSE wall and RSS structures. Many agencies have adopted conservative reinforced fill requirements for both walls and slopes. These conservative properties are suitable for inclusion in standard specifications or special provisions when project specific testing is not feasible and when the quality of construction control and inspection may be in question. **It should be recognized, however, that using conservative reinforced fill property criteria cannot completely replace a reasonable degree of construction control and inspection.**

In general, these select reinforced fill materials will be more expensive than lower quality materials. The specification criteria for each application (walls and slopes) differ somewhat primarily based on performance requirements of the completed structure (allowable deformations) and the design approach. Material suppliers of proprietary MSE systems each have their own criteria for reinforced fills. Detailed project reinforced fill specifications, which uniformly apply to all MSE wall and RSS systems, should be provided by the contracting agency. The following requirements are consistent with current practice:

Select Granular Fill Material for the Reinforced Zone of Walls. All fill material used in the structure volume for MSE wall structures should be reasonably free from organic or other deleterious materials and should conform to the gradation limits, PI and soundness criteria listed in Table 3-1. Note that Table 3-1 presents a broad gradation range that is applicable across the United States. Individual DOTs may adjust this range based upon locally available and economical select granular fill. The reinforced fill should be well-graded in accordance with the Unified Soil Classification System (USCS) in ASTM D2487. Unstable broadly graded soils (i.e., $C_u > 20$ with concave upward grain size distributions) and gap-graded soils should be avoided (see Kenney and Lau, 1985, 1986 for a method to identify unstable soils). These soils tend to pipe and erode internally, creating problems with both loss of materials and clogging of drainage systems.

Table 3-1. MSE Wall Select Granular Reinforced Fill Requirements.

<u>Gradation:</u> (AASHTO T-27)	<u>U.S. Sieve Size</u>	<u>Percent Passing</u> ^(a)
	4 in. (102 mm) ^(a,b)	100
	No. 40 (0.425 mm)	0-60
	No. 200 (0.075 mm)	0-15
<u>Plasticity Index, PI</u> (AASHTO T-90)	PI ≤ 6	
<u>Soundness:</u> (AASHTO T-104)	The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss of less than 30 percent after four cycles (or a sodium sulfate value less than 15 percent after five cycles).	
Notes:		
(a) To apply default F* values, C _u , should be greater than or equal to 4.		
(b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to ¾-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.		

The fill material must be free of organic matter and other deleterious substances, as these materials generally result in poor performance of the structure and enhance degradation for reinforcements. Other materials *such as soils containing mica, gypsum, smectite, montomorrilonite or other soft durability* particles should be carefully evaluated as large strains are typically required to reach peak strength and pullout capacity, resulting in larger lateral and vertical deformation than with higher quality granular fills. Use of salvaged materials such as asphaltic concrete millings or Portland Cement Concrete rubble is not recommended. Recycled asphalt is prone to creep resulting in both wall deformation and reinforcement pullout. Recycled concrete has a potential to produce tufa precipitate from unhydrated cement, which can clog drains and exude a white pasty substance onto the wall face creating aesthetic problems. The recycled concrete typically does not meet electrochemical properties and its corrosion potential has also not been fully evaluated, especially if residual wire and rebar are present that could create problems with dissimilar metals.

The compaction specifications should include a specified lift thickness and allowable range of moisture content with reference to optimum. Compaction moisture control should be $\pm 2\%$ of optimum moisture, w_{opt} .

The compaction requirements of reinforced fill are different in close proximity to the wall facing (within 3 ft {1 m}). Lighter compaction equipment (e.g., walk-behind vibratory plate or roller) and thinner lifts are used near the wall face to prevent buildup of high lateral pressures from the compaction and to prevent facing panel movement. **Because of the use of this lighter equipment, a reinforced fill material of good quality in terms of both friction and drainage, such as crushed stone may be used close to the face of the wall to provide adequate strength and minimize settlement in this zone.** If an open graded fill is used adjacent to the face, filtration requirements with the reinforced wall fill must be addressed, see Section 5.3.3. It should be noted that granular fill containing even a few percent fines (which can even develop or increase due to breakdown during compaction) may not be free draining and drainage requirements should always be carefully evaluated.

Marginal Reinforced Fill for MSE walls. MSE wall reinforced fill materials outside of these gradation and plasticity index requirements (Table 3-1) have been used successfully; however, problems including significant distortion and structural failure have been observed with finer grained and/or more plastic soils. A recent NCHRP research study (NCHRP 24-22) on Selecting Reinforced Fill Materials for MSE Retaining Walls has confirmed that that reinforced fill with up to 35% passing a No. 200 (0.75 mm) sieve could be safely allowed in the reinforced fill, provided the properties of the materials are well defined and controls are established to address the design issues. Design issues include drainage, corrosion, deformations, reinforcement pullout, constructability, and performance expectations. While there may be a significant savings in using lower quality reinforced fill, the affect on performance must be carefully evaluated.

For MSE walls constructed with reinforced fill containing more than 15% passing a No. 200 (0.075 mm) sieve and/or a PI exceeding 6, both total and effective shear strength parameters should be evaluated in order to obtain an accurate assessment of horizontal stresses, sliding, compound failure (behind and through the reinforced zone) and the influence of drainage on the analysis. Both long-term and short-term reinforcement pullout tests as well as soil/reinforcement interface friction tests should be performed. Settlement characteristics must be carefully evaluated, especially in relation to downdrag stresses imposed on connections at the face and settlement of supported structures. Drainage requirements at the back, face, and beneath the reinforced zone must be carefully evaluated (e.g., use flow nets to evaluate influence of seepage forces and hydrostatic pressure). If marginal fill is used the surface of the wall should be positively sloped such that water drains away from the wall (which is a good practice for all MSE walls as discussed in Chapter 5, but most important if marginal fills are used). In addition, a geomembrane is recommended above the wall to preclude infiltration of seepage water into the fill (see Chapter 5, Section 5.3 for drainage design details). Again, these drainage features are good practice for all MSE walls. The

length of the upper 2 layers of reinforcement should be extended at least 3 to 5 ft beyond the lower reinforcement layers to reduce the potential for tension cracks to develop directly behind the reinforced zone. If the soil reinforcement is steel, the extended layers must be contained within select granular fill to avoid differential corrosion conditions.

Electrochemical tests should be performed on the reinforced fill to obtain data for evaluating degradation of reinforcements and facing connections (see Section 3.2.3). Moisture and density control during construction must be carefully controlled in order to obtain strength and interaction values. Deformation during construction also must be carefully monitored and maintained within defined design limits. Performance monitoring is also recommended for reinforced fill soils that fall outside of the requirements listed above, as detailed in Chapter 11.

Reinforced Rock Fill for Wall or RSS Structures. Material that is composed primarily of rock fragments (material having less than 25 percent passing a $\frac{3}{4}$ in (20 mm) sieve) should be considered to be a rock backfill. The maximum particle size should not exceed the limits listed in Table 3-1. Such material should meet all the other non-gradation requirements such as soundness and electrochemical properties in Tables 3-1 to 3-4. When such material is used, a very high survivability geotextile filter (e.g., Type 1 geotextile in accordance with AASHTO M 288), designed for filtration performance following the guidelines in FHWA NHI-07-092 (Holtz et al., 2008), should encapsulate the rock backfill to within 3 ft (1 m) below the wall coping. Adjoining sections of separation fabric should be overlapped by a minimum of 12 in. (0.30 m). Additionally, the upper 3 ft (1 m) of fill should contain no stones greater than 3 in. (75 mm) in their greatest dimension, and should be composed of material not considered to be rock backfill, as defined herein. Where density testing is not possible, trial fill sections should be constructed with agency supervisory personnel and geotechnical specialist present to determine appropriate watering, in situ modification requirements (e.g., grading), lift thickness, and number of passes to achieve adequate compaction. Compaction can be determined by measuring the settlement of the trial section at a number of points after each pass (e.g., a minimum of 5 points measured at the center of a 1 ft square plate is typically required). Several lifts should be constructed to determine the appropriate number of passes, which will maximize compaction without excessively crushing the rock at the surface. The number of passes to achieve at least 80 percent of the maximum settlement should be required.

Select Reinforced Fill for RSS Structures. Less select reinforced fill can be used for RSS since facings are typically flexible and can tolerate some distortion during construction. Even so, a high quality embankment fill meeting the following gradation requirements to facilitate compaction and minimize reinforcement requirements is recommended. The

guidelines listed in Table 3-2 are provided as recommended reinforced fill requirements for RSS construction.

RSS reinforced fill compaction should be based on 95% of AASHTO T-99, and $\pm 2\%$ of optimum moisture, w_{opt} .

RSS fill materials outside of these gradation and plasticity index requirements have been used successfully as well as unsuccessfully. For fill materials outside of these limits, default values for strength and pullout are no longer applicable and laboratory tests must be performed. Issues with drainage problems, excessive distortion and settlement (as discussed above for marginal fill in MSE walls) must be carefully evaluated with finer grained and/or more plastic soils. Performance monitoring is also recommended for reinforced fill soils that fall outside of the requirements listed above, as detailed in Chapter 11.

Table 3-2. RSS Granular Reinforced Fill Requirements.

<u>Gradation:</u> (AASHTO T-27)	<u>U.S. Sieve Size</u>	<u>Percent Passing</u>
	4 in. (102 mm) ^(a,b) ¾-inch (20 mm) ^(a)	100
	No. 4 (4.76 mm)	100 – 20
	No. 40 (0.425 mm)	0-60
	No. 200 (0.075 mm)	0 – 50
<u>Plasticity Index, PI</u> (AASHTO T-90)	PI ≤ 20	
<u>Soundness:</u> (AASHTO T-104)	Magnesium sulfate soundness loss less than 30% after 4 cycles, based on AASHTO T-104 or equivalent sodium sulfate soundness of less than 15 percent after 5 cycles.	
Note:		
(a) To apply default F* values, C _u , should be greater than or equal to 4.		
(b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to ¾-in. (19 mm) for geosynthetics, and epoxy and PVC coated steel reinforcements unless construction damage assessment tests are or have been performed on the reinforcement combination with the specific or similarly graded large size granular fill. Prequalification tests on reinforcements using standard agency fill materials should be considered.		

Design Strength of Select Granular Reinforced Fill. The MSE wall and RSS reinforced fill criteria outlined previously represent materials that have been successfully used throughout the United States and resulted in excellent performance of MSEW and RSS structures. Peak shear strength parameters are used in the wall and slope analyses. For MSE walls using well fill meeting the gradation requirements in Table 3-1, a maximum effective friction angle ϕ' of 34 degrees is usually assumed (in accordance with Article 11.10.6.2, AASHTO, 2007), unless project-specific fill is tested by triaxial (per AASHTO T-296) or direct shear (per AASHTO T-236), per Article 11.10.6.2 (AASHTO, 2007). However, some nearly uniform fine sands meeting the specifications limits may exhibit friction angles of 30 to 32 degrees. When contractor furnished sources are used, the specification may also require testing of the source material to verify that its friction angle meets specification requirements (e.g., 34 degrees). Higher values may be used if substantiated by laboratory direct shear or triaxial test results for the site specific material used or proposed. If the measured friction angle is greater than 40 degrees, the angle of friction used for design should not exceed 40 degrees (Article 11.10.6.2, AASHTO {2007}). In all cases, the cohesion of the reinforced fill is assumed to be zero.

For RSS structures, where a considerably greater percentage of fines (minus No. 200 sieve) is permitted, lower bound values of frictional strength equal to 28 to 30 degrees would be typical for the reinforced fill requirements listed. A significant economy could again be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value. Likewise, soils outside the gradation range listed should be carefully evaluated and monitored.

Limits of Reinforced Fill. For MSE walls, except back-to-back walls, and RSS, many agencies extend the reinforced fill beyond the free end of the reinforcement. Some agencies extend the reinforced fill 1 ft (0.3 m) beyond the reinforcement length, and some others extend the fill in a wedge behind the reinforced zone, as illustrated in Figure 3-1. For back-to-back walls wherein the free ends of the reinforcement of the two walls are spaced apart less than or equal to one-half the design height of the taller wall, reinforced backfill should be used for the space between the free ends of the reinforcements as well.

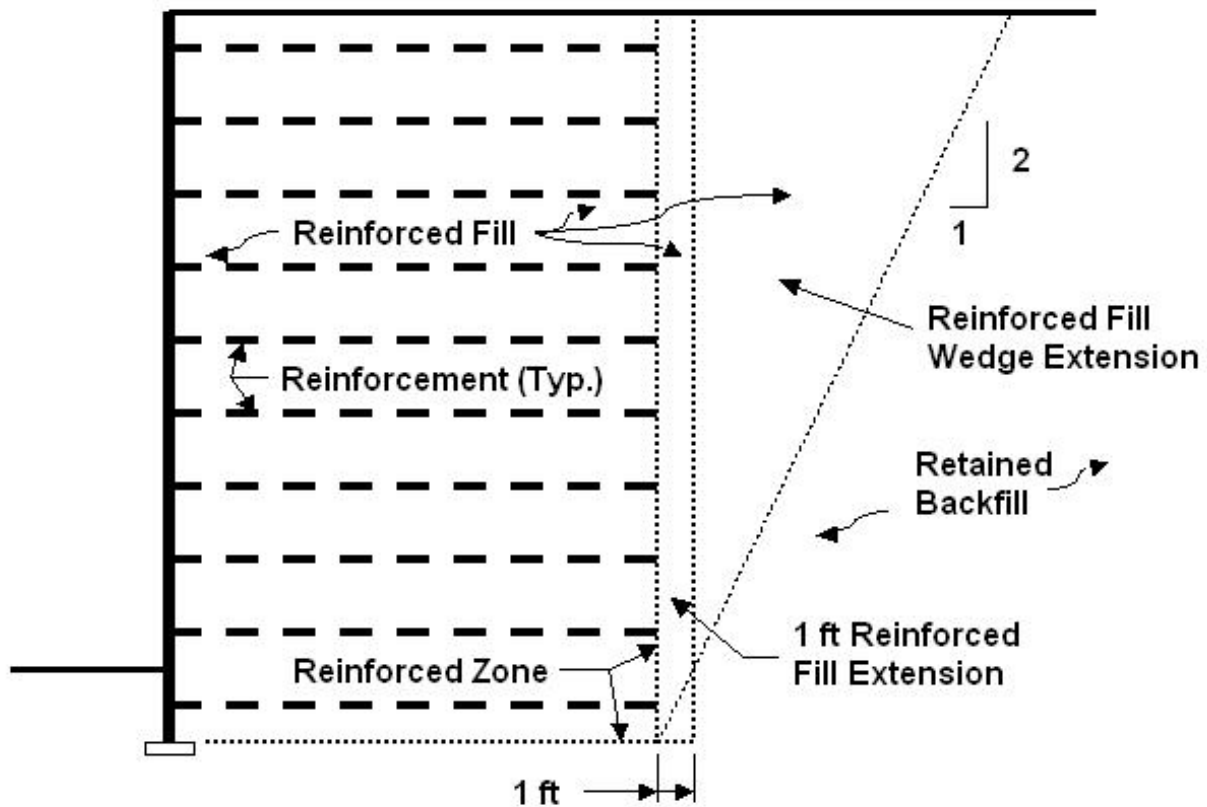


Figure 3-1. Examples of reinforced fill zone extension beyond the reinforced zone.

3.2.2 Retained Backfill and Natural Retained Soil

The key engineering properties required for the retained backfill are the strength and unit weight based on evaluation and testing of subsurface or borrow pit data. Friction angles (ϕ) may be determined from either by consolidated drained triaxial tests with pore pressure measurements or drained direct shear tests. As with reinforced fill, a cohesion value of zero is conservatively recommended for the long-term, effective strength of the retained fill. For backcut construction, if undisturbed samples cannot be obtained, friction angles may be obtained from in-situ tests or by correlations with index properties. The strength properties are required for the determination of the coefficients of earth pressure used in design as well as for overall stability analysis. In addition, the position of groundwater levels above the proposed base of construction must be determined in order to evaluate hydrostatic stresses in the retained fill and plan an appropriate drainage scheme to control ground water conditions. For most retained backfills lower bound frictional strength values of 28 to 30 degrees are reasonable for granular and low plasticity cohesive soils. For highly plastic retained fills and natural soils ($PI > 20$), even lower values would be indicated and should be evaluated for both drained and undrained conditions.

Backfill and natural soil behind the limits of the reinforced fill should be considered to be in the retained zone for a distance equal to 50 percent of the design height of the MSE wall. For the reasons discussed previously for reinforced fill, use of soils containing shale, mica, gypsum, smectite, montmorillonite or other soft particles of poor durability is discouraged and soundness limits should meet the criteria in Table 3-1.

The following are good practice to preclude potential problems with retained backfill soils. The percent fines, i.e., the fraction passing No. 200 sieve (0.075 mm), should be less than 50 and the Liquid Limit and Plasticity Index (PI) should be less than 40 and 20 percent, respectively, as determined in accordance with AASHTO T-90. The potential differential settlement/performance between the reinforced fill and retained backfill should be assessed. The agency should consider transition detailing between the reinforced zone and retained backfill by lengthening the upper two layers of soil reinforcement or extending the reinforced zone beyond the reinforcement length, as previously discussed. The maximum particle size in the retained backfill should be limited to the maximum particle size in the reinforced wall fill, at least within this transition zone. Material that is composed primarily of rock fragments (material having less than 25 percent passing a 3/4-inch sieve), should be considered to be a rock backfill (see Section 3.2.1).

3.2.3 Electrochemical Properties

The design of buried steel elements of MSE structures is predicated on reinforced fills exhibiting minimum or maximum electrochemical index properties and then designing the structure for maximum corrosion rates associated with these properties. These recommended index properties and their corresponding limits are shown in Table 3-3. **Reinforced fill soils must meet the indicated criteria to be qualified for use in MSE construction using steel reinforcements.**

Table 3-3. Recommended Limits of Electrochemical Properties for Reinforced Fills with Steel Reinforcement.

<u>Property</u>	<u>Criteria</u>	<u>Test Method</u>
Resistivity	> 3000 ohm-cm	AASHTO T-288
pH	> 5 and < 10	AASHTO T-289
Chlorides	< 100 PPM	ASMT D4327
Sulfates	< 200 PPM	ASTM D4327
Organic Content	1% max.	AASHTO T-267

Table 3-4. Recommended Limits of Electrochemical Properties for Reinforced Fills with Geosynthetic Reinforcements (FHWA NHI-09-087, Elias et al., 2009).

<u>Base Polymer</u>	<u>Property</u>	<u>Criteria</u>	<u>Test Method</u>
Polyester (PET)	pH	$3 < \text{pH} < 9$	AASHTO T-289
Polyolefin (PP & HDPE)	pH	$\text{pH} > 3$	AASHTO T-289

Where geosynthetic reinforcements are planned, the limits for electrochemical criteria will vary depending on the polymer. Limits, based on current research, are shown in Table 3-4.

3.3 REINFORCED SOIL CONCEPTS

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:

- Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.
- Reinforcements are distributed throughout the soil zone with a degree of regularity. localized.

3.3.1 Stress Transfer Mechanisms

Stresses are transferred between soil and reinforcement by friction (Figure 3-2a) and/or passive resistance (Figure 3-2b) depending on the reinforcement geometry.

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and the reinforcement surface. Reinforcing elements dependent on friction should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile, geosynthetic straps, and some geogrid layers.

Passive resistance occurs through the development of bearing type stresses on "transverse" reinforcement surfaces normal to the direction of soil reinforcement relative movement. Passive resistance is generally considered to be the primary interaction for bar mat, wire mesh reinforcements, and geogrids with relatively stiff cross machine direction ribs. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance.

The contribution of each transfer mechanism for a particular reinforcement will depend on the roughness of the surface (skin friction), normal effective stress, grid opening dimensions, thickness of the transverse members, and elongation characteristics of the reinforcement. Equally important for interaction development are the soil characteristics, including grain size, grain size distribution, particle shape, density, water content, cohesion, and stiffness.

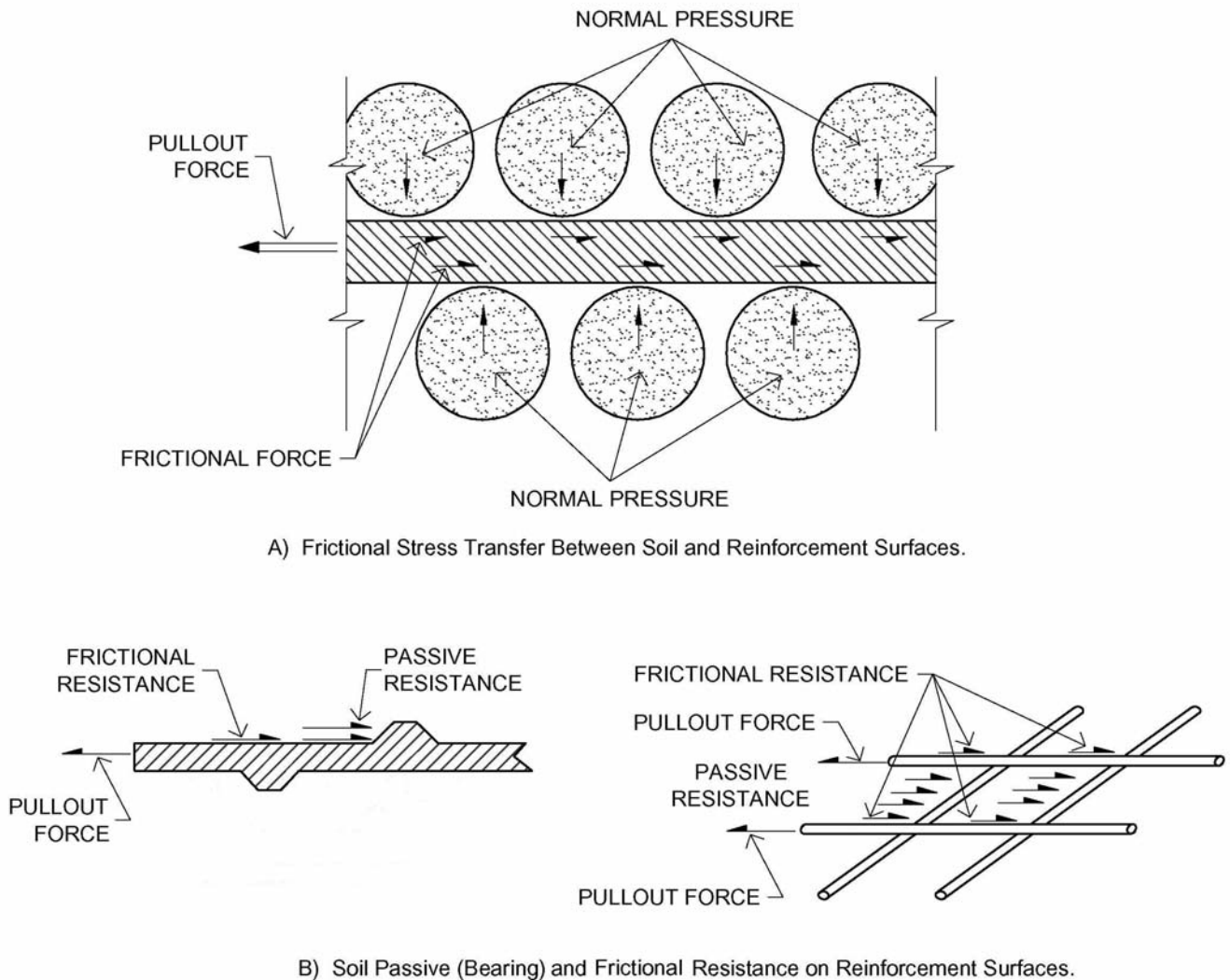


Figure 3-2. Stress transfer mechanisms for soil reinforcement.

3.3.2 Mode of Reinforcement Action

The primary function of reinforcements is to restrain soil deformations. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are resisted by the reinforcement tension and/or shear and bending.

- **Tension** is the most common mode of action of tensile reinforcements. All "longitudinal" reinforcing elements (i.e., reinforcing elements aligned in the direction of soil extension) are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.
- **Shear and Bending.** "Transverse" reinforcing elements that have some rigidity, can withstand shear stress and bending moments.

3.3.3 Geometric Characteristics

Two types can be considered:

- **Strips, bars, and steel grids.** A layer of steel strips, bars, or grids is characterized by the cross-sectional area, the thickness and perimeter of the reinforcement element, and the center-to-center horizontal distance between elements (for steel grids, an element is considered to be a longitudinal member of the grid that extends into the wall).
- **Geotextiles and geogrids.** A layer of geosynthetic strips is characterized by the width of the strips and the center-to-center horizontal distance between them. The cross-sectional area is not needed, since the strength of a geosynthetic strip is expressed by a tensile force per unit width, rather than by stress. Difficulties in measuring the thickness of these thin and relatively compressible materials preclude reliable estimates of stress.

The coverage ratio R_c is used to relate the force per unit width of discrete reinforcement to the force per unit width required across the entire structure. See Figure 3-3 (and later Figure 3-5) for an illustration of these terms.

$$R_c = \frac{b}{S_h} \quad (3-1)$$

where:

b = the gross width of the strip, sheet, or grid. For grids, b is measured from the center to center of the outside longitudinal bars as shown in Figure 3-3.

S_h = center-to-center horizontal spacing between strips, sheets, or grids

Note, $R_c = 1$ in the case of continuous reinforcement, i.e., each reinforcement layer covers the entire horizontal surface of the reinforced soil zone. Alternatively, for discrete reinforcements and segmental precast concrete facing, force per width may be more conveniently calculated per panel width, as defined later in Equation 4-25c, for layout and detailing.

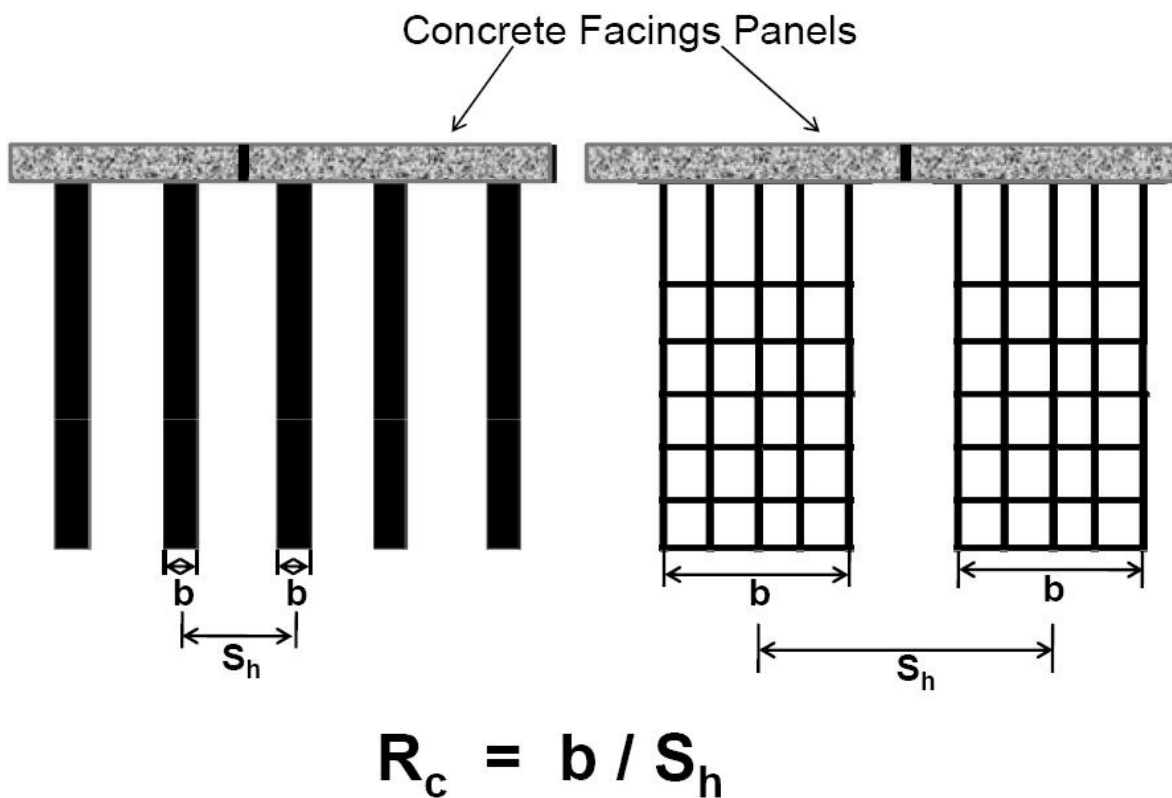


Figure 3-3. Coverage ratio.

3.4 SOIL REINFORCEMENT INTERACTION USING NORMALIZED CONCEPTS

Soil-interaction (pullout capacity) coefficients have been developed by laboratory and field studies, using a number of different approaches, methods, and evaluation criteria. A unified normalized approach developed in a FHWA research project is detailed below.

3.4.1 Evaluation of Pullout Performance

The design of the soil reinforcement system requires an evaluation of the long-term pullout performance with respect:

- Pullout capacity, i.e., the pullout resistance of each reinforcement should be adequate to resist the factored tensile force in the reinforcement with a specified resistance factor (or factor of safety in the case of RSS).
- Allowable displacement, i.e., the relative soil-to-reinforcement displacement required to mobilize the design tensile force should be smaller than the allowable displacement.
- Long-term displacement, i.e., the pullout load should be smaller than the critical creep load.

The pullout resistance of the reinforcement is mobilized through one or a combination of the two basic soil-reinforcement interaction mechanisms, interface friction and passive soil resistance against transverse elements of reinforcements such as bar mats, wire meshes, or geogrids. The load transfer mechanisms mobilized by a specific reinforcement depends primarily upon its structural geometry (i.e., composite reinforcement such as grids, versus linear or planar elements, thickness of transverse elements, and aperture dimension). The soil-to-reinforcement relative movement required to mobilize the design tensile force depends mainly upon the load transfer mechanism, the extensibility of the reinforcement material, the soil type, and the confining pressure.

The long-term pullout performance (i.e., displacement under constant design load) is predominantly controlled by the creep characteristics of the soil and the reinforcement material. Soil reinforcement systems will generally not be used with cohesive soils susceptible to creep. Therefore, creep is primarily controlled by the type of reinforcement. Pullout performance in terms of the main load transfer mechanism, relative soil-to-reinforcement displacement required to fully mobilize the pullout resistance, and creep potential of the reinforcement in granular (and low plasticity cohesive) soils for generic reinforcement types is provided in Table 3-5.

**Table 3-5. Reinforcement Pullout Performance in Granular and Cohesive
Soils of Low Plasticity.**

Generic Reinforcement Type	Major Load Transfer Mechanism	Range of Displacement at Specimen Front	Long Term Deformation
Inextensible strips			
Smooth	Frictional	0.05 in. (1.2 mm)	Noncreeping
Ribbed	Frictional + passive	0.5 in. (12 mm)	Noncreeping
Extensible composite plastic strips	Frictional	Dependent on reinforcement extensibility	Dependent on reinforcement structure and polymer creep
Extensible sheets			
Geotextiles	Frictional	Dependent on reinforcement extensibility (1 to 4 in.) {25 to 100 mm}	Dependent on reinforcement structure and polymer creep characteristics
Inextensible grids			
bar mats	Passive + frictional	0.5 to 2 in. (12 to 50 mm)	Noncreeping
welded wire meshes	Frictional + passive	0.5 to 2 in. (12 to 50 mm)	Noncreeping
Extensible grids			
Geogrids	Frictional + passive	Dependent on extensibility (1 to 2 in.) {25 to 50 mm}	Dependent on reinforcement structure and polymer creep characteristics
woven wire meshes	Frictional + passive	1 to 2 in. (25 to 50 mm)	Noncreeping

3.4.2 Estimate of the Reinforcement Pullout Capacity in MSE Structures

The pullout resistance of the reinforcement is defined by the ultimate tensile load required to generate outward sliding of the reinforcement through the reinforced soil zone. Several approaches and design equations have been developed and are currently used to estimate the pullout resistance by considering frictional resistance, passive resistance, or a combination of both. The design equations use different interaction parameters, so it is difficult to compare the pullout performance of different reinforcements for a specific application.

For design and comparison purposes, a normalized definition of pullout resistance will be used throughout the manual. The pullout resistance, P_r , at each of the reinforcement levels per unit width of reinforcement is given by:

$$P_r = F^* \alpha \sigma'_v L_e C \quad (3-2)$$

where:

- $L_e C$ = the total surface area per unit width of the reinforcement in the resistive zone behind the failure surface
- L_e = the embedment or adherence length in the resisting zone behind the failure surface
- C = the reinforcement effective unit perimeter; e.g., $C = 2$ for and sheets, and because the edges are neglected $C = 2$ for strips and grids
- F^* = the pullout resistance (or friction-bearing-interaction) factor
- α = a scale effect correction factor to account for a non linear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (generally 1.0 for metallic reinforcements and 0.6 to 1.0 for geosynthetic reinforcements, see Table 3-6).
- σ'_v = the effective vertical stress at the soil-reinforcement interfaces.

The correction factor α depends, therefore, primarily upon the strain softening of the compacted granular backfill material, and the extensibility and the length of the reinforcement. For inextensible reinforcement, α is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The α factor (a scale correction factor) can be obtained from pullout tests on reinforcements with different lengths as presented in Appendix B, or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, $\alpha = 0.8$ for geogrids and $\alpha = 0.6$ for geotextiles (extensible sheets) is recommended (see Table 3-6).

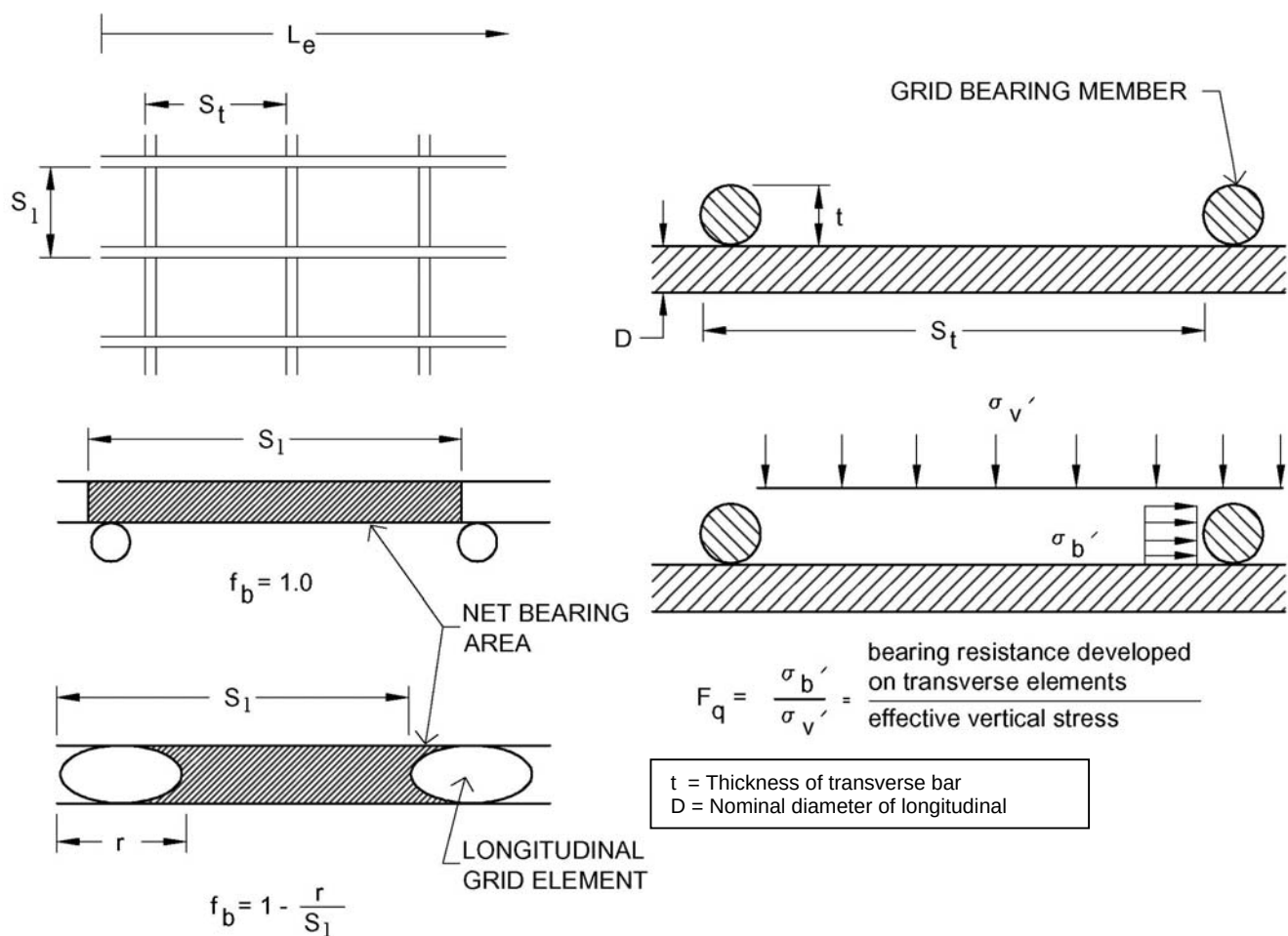
Table 3-6. Summary of Pullout Capacity Design Parameters.

Reinforcement Type	S_{opt}	Grid Spacing	Tan ρ	F_q	α_β	α Default Value
Inextensible strips		NA	Obtain Tan ρ from tests, or use default values	NA	NA	1.0
Inextensible grids (bar mats and welded wire)	$\frac{t F_q}{2 \tan \phi}$	$S_t \leq S_{opt}$	Obtain Tan ρ from tests	NA	NA	1.0*
	$\frac{t F_q}{2 \tan \phi}$	$S_t > S_{opt}$	NA	Obtain F_q from tests, or use default values	$\frac{t}{2 S_t}$	1.0*
Extensible grids with min. grid opening $d_{50} > 1$	$\frac{t F_q}{2 \tan \phi}$	$S_t \leq S_{opt}$	Obtain Tan ρ from tests	NA	NA	0.8
	$\frac{t F_q}{2 \tan \phi}$	$S_t > S_{opt}$	NA	Obtain F_q from tests, or use default values	$\frac{f_b t}{2 S_t}$	0.8
Extensible grids with min. grid opening $d_{50} < 1$		NA	Obtain Tan ρ from tests	NA	NA	0.8
Extensible sheets		NA	Obtain Tan ρ from tests	NA	NA	0.6

NOTES:

- (i) It is acceptable to use the empirical values provided in or referenced by this table to determine F^* in the absence of product and backfill specific test data, provided granular reinforced fill as specified in Table 3-1 for MSE walls is used and $C_u \geq 4$. For fill outside these limits, tests must be run.
- (ii) Pullout testing to determine α is recommended if α shown in table is less than 1.0. These values of α represent highly extensible geosynthetics.
- (iii) For grids where Tan ρ is applicable, apply Tan ρ to the entire surface area of the reinforcement sheet (i.e., soil and grid), not just the surface area of the grid elements.
- (iv) NA means "not applicable." ϕ is the soil friction angle. ρ is the interface friction angle mobilized along the reinforcement. S_{opt} is the optimum transverse grid element spacing to mobilize maximum pullout resistance as obtained from pullout tests (typically 6 in. (150 mm) or greater). S_t is the spacing of the transverse grid elements. t is the thickness of the transverse elements. F_q is the embedment (or surcharge) bearing capacity factor. α_β is a structural geometric factor for passive resistance. f_b is the fraction of the transverse member on which bearing can be fully developed (typically ranging from 0.6 to 1.0) as obtained from an evaluation of the bearing surface shape. D_{50} is the backfill grain size at 50% passing by weight. α is the scale effect correction factor.
- (v) Definitions of the geometric variables are illustrated in Figure 3-4.

* For longitudinal bars/wires spacing greater than 6 inches, α may be less than 1.0 and pullout tests are required.



S_l = Distance between longitudinal bars.

S_t = Distance between transverse bars.

Notes:

1. Transverse bar thickness does not need to be reduced for corrosion.
2. This is applicable up to a maximum transverse bar spacing of 24 in. (610 mm).

Figure 3-4. Definition of grid dimensions for calculating pullout capacity.

The Pullout Resistance Factor F^* can be obtained most accurately from laboratory or field pullout tests performed in the specific backfill to be used on the project. Test procedures for determining pullout parameters are presented in Appendix B. Alternatively, F^* can be derived from empirical or theoretical relationships developed for each soil-reinforcement interaction mechanism and provided by the reinforcement supplier. For any reinforcement, F^* can be estimated using the general equation:

$$F^* = \text{Passive Resistance} + \text{Frictional Resistance}$$

$$\text{or, } F^* = F_q \cdot \alpha_\beta + \tan \rho \quad (3-3)$$

where:

F_q = the embedment (or surcharge) bearing capacity factor

α_β = a bearing factor for passive resistance which is based on the thickness per unit width of the bearing member.

ρ = the soil-reinforcement interaction friction angle.

The pullout capacity parameters for Equation 3-3 are summarized in Table 3-6 and Figure 3-4 for the soil reinforcement systems considered in this manual.

A significant number of laboratory pullout tests have been performed for many commonly used reinforcement backfill combinations and correlated to representative field pullout tests. Therefore, the need for additional laboratory and/or field pullout tests, is generally limited to reinforcement/reinforced fill combinations where this data is sparse or nonexistent (e.g., uniform and marginal reinforced fill discussed in Section 3.1). Where applicable, laboratory pullout tests should be made in accordance with the device and procedures in ASTM D6706 and Appendix B of this manual. Note that this test procedure provides a short-term pullout capacity and does not account for soil or reinforcement creep deformations, which may be significant in MSE wall and RSS structures utilizing fine grained soil fills.

When using laboratory pullout tests to determine design parameters, vertical stress variations and reinforcement element configurations for the actual project should be used. Tests should be performed on samples with a minimum embedded length of 24 in. (600 mm). The pullout resistance is the greater of the peak pullout resistance value prior to, or the value achieved at, a maximum deformation of 3/4-in. (20 mm) as measured at the front of the embedded section for inextensible reinforcements and 5/8-in. (15 mm) as measured at the end of the embedded sample for extensible reinforcements. This allowable deflection criterion is based on a need to limit the structure deformations, which are necessary to develop sufficient pullout capacity.

Long-term pullout tests to assess soil/reinforcement creep behavior should be conducted when silt or clay reinforced fill is being used. Soil properties and reinforcement type will determine if the allowable pullout resistance is governed by creep deformations. The placement and compaction procedures for both short-term and long-term pullout tests should simulate field conditions. The allowable deformation criteria in the previous paragraph should be applied.

A summary of the procedures for evaluating laboratory tests to obtain pullout design parameters is outlined in Appendix B of this manual.

Most specialty system suppliers have developed recommended pullout parameters for their products when used in conjunction with the select backfill detailed in this chapter for MSEW and RSS structures. The semi-empirical relationships summarized below are consistent with results obtained from laboratory and field pullout testing at a 95 percent confidence limit, and generally consistent with suppliers developed data. Some additional economy can be obtained from site/product specific testing, where the source of the backfill in the reinforced volume has been identified during design.

In the absence of site-specific pullout testing data, it is reasonable to use the semi-empirical relationships described in the following paragraphs in conjunction with the standard specifications for reinforced fill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, the Pullout Resistance Factor F^* is commonly taken as:

$$F^* = \tan \rho = 1.2 + \log C_u \text{ at the top of the structure} = 2.0 \text{ maximum} \quad (3-4)$$

$$F^* = \tan \phi \text{ at a depth of 20 ft (6 m) and below} \quad (3-5)$$

where C_u is the uniformity coefficient of the backfill (D_{60}/D_{10}). **If the specific C_u for the wall backfill is unknown at design time, a $C_u = 4$ should be assumed (i.e., $F^* = 1.8$ at the top of the wall) for reinforced fills meeting the requirements of Section 3.1 of this chapter.**

For steel grid reinforcements with transverse spacing of $S_t \geq 6$ inches (150 mm) (see Figure 3-4), F^* is a function of a bearing or embedment factor, F_q , applied over the contributing bearing α_β , as follows:

$$F^* = F_q \alpha_\beta = 40 \alpha_\beta = 40 (t/2S_t) = 20 (t/S_t) \text{ at the top of the structure} \quad (3-6)$$

$$F^* = F_q \alpha_\beta = 20 \alpha_\beta = 20 (t/2S_t) = 10 (t/S_t) \text{ at a depth of 20 ft (6 m) and below} \quad (3-7)$$

Where, t is the thickness of the transverse bar. S_t must be uniform throughout the length of the reinforcement rather than having transverse grid members concentrated only in the resistant zone. The maximum S_t distance is 24 in. (610 mm). For sloping backfills see Figure 4-15.

For geosynthetic (i.e., geogrid and geotextile) sheet reinforcement, the pullout resistance is based on a reduction in the available soil friction with the reduction factor often referred to as an Interaction Factor, C_i . In the absence of test data, the F^* value for geosynthetic reinforcement should conservatively be taken as:

$$F^* = \frac{2}{3} \tan \phi \quad (3-8)$$

Where used in the above relationships, ϕ is the peak friction angle of the soil which for MSE walls using select granular backfill, is taken as a maximum of 34 degrees unless project specific test data substantiates higher values. For RSS structures, the ϕ angle of the reinforced backfill is normally established by test, as a reasonably wide range of backfills can be used. A lower bound value of 28 degrees is often used.

3.4.3 Interface Shear

The interface shear between sheet type geosynthetics (geotextiles, geogrids and geocomposite drains) and the soil is often lower than the friction angle of the soil itself and can form a slip plane. Therefore the interface friction coefficient $\tan \rho$ must be determined in order to evaluate sliding along the geosynthetic interface with the reinforced fill and, if appropriate, the foundation or retained backfill soil. The interface friction angle ρ is determined from soil-geosynthetic direct shear tests in accordance with ASTM D 5321. In the absence of test results, the interface friction coefficient can be conservatively taken as:

$$\rho = \frac{2}{3} \tan \phi \quad (3-9)$$

for geotextiles, geogrids and geonet type drainage composites. Other geosynthetics such as geomembranes and some geocomposite drain cores may have much lower interface values and tests should accordingly be performed.

3.5 ESTABLISHMENT OF STRUCTURAL DESIGN PROPERTIES

The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The two most commonly used reinforcement materials, steel and geosynthetics, must be considered separately as follows:

3.5.1 Strength Properties of Steel Reinforcements

For steel reinforcements, the design life is achieved by reducing the cross-sectional area of the reinforcement used in design calculations by the anticipated corrosion losses over the design life period as follows:

$$E_c = E_n - E_R \quad (3-10)$$

where E_c is the thickness of the reinforcement at the end of the design life, E_n the nominal thickness at construction, and E_R the sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure.

The nominal long-term tensile strength of the reinforcement, T_{al} , is obtained for steel strips and grids as shown in the following equations. T_{al} in units force per unit width is used to provide a unified strength approach, which can be applied to any reinforcement. Tensile strength of a known steel or grid reinforcement can also be expressed in terms of the tensile load carried by the reinforcement, P_{tal} . The desired designation of reinforcement tensile strength (T_{al} or P_{tal}) varies depending on whether one is designing with a known system, designing with an undefined reinforcement, checking a design layout, performing connection design, or performing reinforcement pullout calculations. Thus, nominal tensile strength may be calculated and expressed in the following terms:

$$T_{al} = \frac{F_y A_c}{b} \quad (\text{in strength per unit reinforcement width \{kips/ft\}}) \quad (3-11a)$$

$$P_{tal} = F_y A_c \quad (\text{in strength per reinforcement element \{kips\}}) \quad (3-11b)$$

where:

b = the gross width of the strip, sheet or grid (see Figure 3-5)

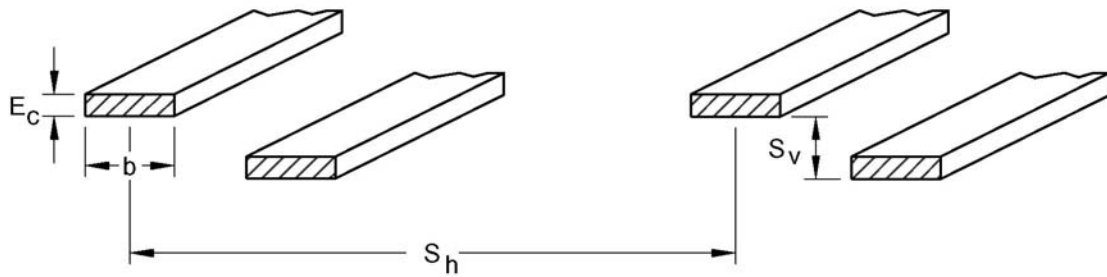
F_y = yield stress of steel

A_c = design cross section area of the steel, defined as the original cross section area minus corrosion losses anticipated to occur during the design life of the wall.

The LRFD resistance factors for steel reinforcements in MSE walls are listed in Table 4-8. The resistance factor for strip reinforcements under static conditions is 0.75. The resistance factors for steel grid MSE wall reinforcements, for static loading, is 0.65 when reinforcement is connected to a rigid facing element and is 0.75 when connected to a flexible facing. The lower resistance factor for grid reinforcing members connected to a rigid facing element (e.g., a concrete panel or block) is used to account for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars. Transverse and longitudinal grid members are sized in accordance with ASTM A185.

The quantities needed to determine A_c for steel strips and grids are shown in Figure 3-5. Typical dimensions for common steel reinforcements are provided in Appendix C. The use of hardened and otherwise low strain (very high strength) steels may increase the potential for catastrophic failure; therefore, a lower resistance factor may be warranted with such materials.

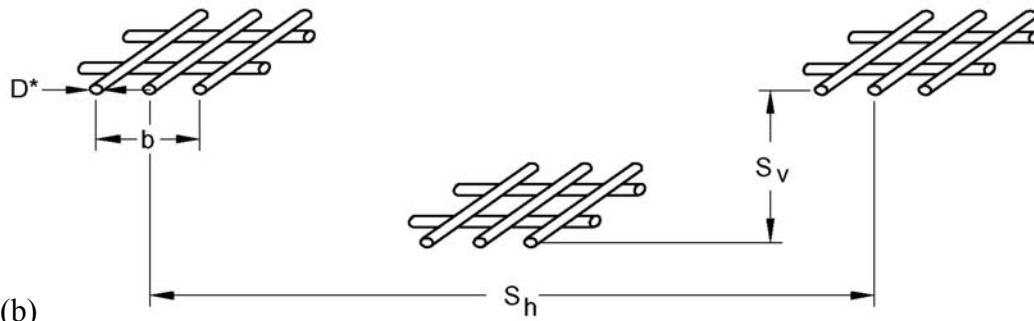
For metallic reinforcement, the life of the structure will depend on the corrosion resistance of the reinforcement. Practically all the metallic reinforcements used in construction of embankments and walls, whether they are strips, bar mats, or wire mesh, are made of galvanized mild steel. Woven meshes with PVC coatings provide some corrosion protection, provided the coating is not significantly damaged during construction. Epoxy coatings can be used for corrosion protection, but are susceptible to construction damage, which can significantly reduce the coatings effectiveness. When PVC or epoxy coatings are used, the maximum particle size of the backfill should be restricted to $\frac{3}{4}$ -inch (19 mm) or less to reduce the potential for construction damage. For a more detailed discussion of requirements, refer to the Corrosion/Degradation manual, FHWA NHI-09-087 (Elias et al., 2009).



$$A_c = b E_c$$

E_c = strip thickness corrected for corrosion loss.

(a)



(b)

$$A_c = (\text{No. of longitudinal bars}) \times \left(\frac{(D^*)^2}{4} \right)$$

D^* = Diameter of bar or wire corrected for corrosion loss

b = Unit width of reinforcement (if reinforcement is continuous count the number of bars for reinforcement width of 1 unit)

$$T_{\max} = \phi R_c T_{al} = \frac{\phi R_c F_y A_c}{b}$$

where $T_{\max}$ = Maximum factored load applied to reinforcement (load/unit wall width)

T_{al} = Nominal long-term tensile strength of the reinforcement (strength/unit reinforcement width)

ϕ = 0.75 for steel strip

ϕ = 0.65 for steel grid and rigid face

F_y = Yield strength of steel

R_c = Reinforcement coverage ratio = $\frac{b}{S_h}$

Use R_c = 1 for continuous reinforcement (i.e., $S_h = b = 1$ unit width)

Figure 3-5. Parameters for metal reinforcement nominal strength calculations showing (a) steel strips and (b) metallic grids and bar mats.

Several DOTs have used resin-bonded epoxy coated steel reinforcing elements. The effectiveness of these coatings in MSE wall structures has not been well documented. If used, the minimum coating thickness should be on the order of 18 mils (45 μm), and applied in accordance with ASTM A884 for grid reinforcement and AASHTO M284 for strip reinforcement. The in-ground design life of the coating should be considered as equal to that of a galvanized reinforcement with a coating thickness of 3.4 mils (85 μm), unless durability exposure testing has been performed on the specific coating that identifies a longer effective life as discussed in FHWA NHI-09-087 (Elias et al., 2009). Where other metals, such as aluminum alloys or stainless steel have been used, corrosion, unexpectedly, has been a severe problem, and their use has been discontinued.

The in-ground degradation resistance of PVC coated mesh has not been sufficiently demonstrated. Anecdotal evidence of satisfactory performance in excess of 25 years does not exist.

Extensive studies have been made to determine the rate of corrosion of galvanized mild steel bars or strips buried in different types of soils commonly used in reinforced soil. Based on these studies, deterioration of steel strips, mesh, bars and mats can be estimated and accounted for by using increased metal thickness.

The majority of MSE walls constructed to date have used galvanized steel and backfill materials with low corrosive potential. A minimum galvanization coating of 2.0 oz/ft² (605 g/m²) or 3.4 mils (85 μm) thickness is required per Article 11.10.6.4.2a (AASHTO, 2007). Galvanization shall be applied in accordance with AASHTO M 111 (ASTM A 123) for strip type, bar mat, or grid type reinforcements and ASTM A 153 for accessory parts such as bolts and tie strips. Galvanization shall be applied after fabrication in accordance with ASTM A123. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion (which can occur in non-galvanized or “black” steel). After the zinc is oxidized (consumed), corrosion of the base metal starts.

The ASTM and AASHTO standards for galvanization provide different required minimum galvanization coating thickness as a function of the bar or wire thickness. However, as noted previously AASHTO (2007) requires a minimum thickness of 3.4 mils (85 μm) for MSE walls. Galvanization requirements using this minimum and AASHTO M 111 are summarized in Table 3-7.

Table 3-7. Minimum Galvanization Thickness by Steel Thickness
(after AASHTO M 111 and ASTM A123).

Category	Steel Thickness	Minimum Galvanization Thickness
Strip	< ¼ in. (6.4 mm)	3.4 mils (85 µm)
	> ¼ in. (6.4 mm)	3.9 mils (100 µm)
Wire*	All diameters	3.4 mils (85 µm)
* For bar mats fabricated from uncoated steel wire.		

The corrosion rates presented in Table 3-8 are suitable for conservative design. **These rates assume a moderately corrosive backfill material having the controlled electrochemical property limits** that are discussed under electrochemical properties in this chapter.

Table 3-8 Steel Corrosion Rates for Moderately Corrosive Reinforced Fill.

For zinc/side:	0.58 mils/yr (15 µm/year) (first 2 years)
	0.16 mils/yr (4 µm/year) (thereafter)
For residual carbon steel/side:	0.47 mils/yr (12 µm/year) (thereafter)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 3.4 mils (85 µm) (AASHTO, 2007) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 0.055 in. to 0.08 in. (1.42 mm to 2.02 mm) would be anticipated over the remaining years of a 75 to 100 year design life, respectively. Galvanization can also be damaged during handling and construction by abrasion, scratching, notching, and cracking. Care must be taken during handling and construction to avoid damage. Construction equipment should not travel directly on reinforcing elements and elements should not be dragged, excessively bent, or field cut. Galvanized reinforcement should be well supported during lifting and handling to prevent excessive bending. Any damaged section should be field repaired by coating the damaged area with a field grade zinc-rich paint.

The look of galvanized WWM face may not be desired on some projects due to aesthetic requirements. As previously noted, black (ungalvanized) steel is not allowed on permanent structures. Staining of galvanized WWM has been used to achieve desired aesthetics on some projects.

The designer of an MSE structure should also consider the potential for changes in the reinforced fill environment during the structure's service life. In certain parts of the United States, it can be expected that deicing salts, coastal storm surges, or contaminated runoff or groundwater might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern such that additional protective measures may be required.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 8 ft (2.5 m) of the reinforced backfill (as measured from the roadway surface) or greater depths, depending on the gradation and compaction of the fill, are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates as shown in the Design Details section in Chapter 5. Alternatively free draining reinforced fill (e.g., AASHTO No. 57 stone) has also been found to allow salts to “flush out” and limit corrosion as discussed in FHWA NHI-09-087 (Elias et al., 2009). Note that value of “higher” corrosion rate for deicing salt exposure is not defined.

The following project situations lie outside the scope of the previously presented values:

- Structures exposed to a marine or other chloride-rich environment. (Excluding locations where de-icing salts are used.) For marine saltwater structures, carbon steel losses on the order of 3.2 mils (80 μm) per side or radius should be anticipated in the first few years, reducing to 0.67 to 0.7 mils (17 to 20 μm) thereafter. Zinc losses are likely to be quite rapid as compared to losses in reinforced fills meeting the MSE electrochemical criteria. Total loss of zinc (3.4 mils {85 μm }) should be anticipated in the first year.
- Structures exposed to stray currents, such as from nearby underground power lines, and structures supporting or located adjacent to electrical railways.
- Structures exposed to acidic water emanating from mine waste, abandoned coal mines, or pyrite-rich soil and rock strata.

Each of these situations creates a special set of conditions that should be specifically analyzed by a corrosion specialist.

3.5.2 Strength Properties of Geosynthetic Reinforcement

Selection of long-term nominal tensile strength, T_{al} , for geosynthetic reinforcement is determined by thorough consideration of all possible strength time dependent strength losses over the design life period. The tensile properties of geosynthetics are affected by factors such as creep, installation damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer can vary widely requiring a T_{al} determination for each individual product with consideration of all these factors.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on polymer type. In addition, these materials are susceptible to installation damage and the effects of high temperature at the facing and connections. Temperature acts to accelerate creep and aging processes and temperature effects are accounted for through their determination. While the normal range of in-ground temperature vary from 55° F (12° C) in cold and temperate climates to 85° F (30° C) in arid desert climates, temperatures at the facing and reinforcement connections can be as high as 120° F (50° C). Confining stress is not directly taken into account other than indirectly when installation damage is evaluated. For creep and durability, confining stress generally will tend to improve the long-term strength of the reinforcement.

The available long-term strength, T_{al} , is calculated as follows:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D} \text{ (in strength per unit reinforcement width)} \quad (3-12)$$

where,

T_{ult} = Ultimate Tensile Strength (strength per unit width). The tensile strength of the reinforcement is determined from wide strip tests per ASTM D4595 (geotextiles) or D6637 (geogrids) based on the minimum average roll value (MARV) for the product.

RF = Reduction Factor. The product of all applicable reduction factors.

RF_{ID} = Installation Damage Reduction Factor. A reduction factor that accounts for the damaging effects of placement and compaction of soil or aggregate over the

geosynthetic during installation. A minimum reduction factor of 1.1 should be used to account for testing uncertainties.

RF_{CR} = Creep Reduction Factor. A reduction factor that accounts for the effect of creep resulting from long-term sustained tensile load applied to the geosynthetic.

RF_D = Durability Reduction Factor. A reduction factor that accounts for the strength loss caused by chemical degradation (aging) of the polymer used in the geosynthetic reinforcement (e.g., oxidation of polyolefins, hydrolysis of polyesters, etc.).

RF_{ID} , RF_{CR} , and RF_D reflect actual long-term strength losses, analogous to loss of steel strength due to corrosion. This long-term geosynthetic reinforcement strength loss concept is illustrated in Figure 3-6. As shown in the figure, some strength losses occur immediately upon installation, and others occur throughout the design life of the reinforcement. Much of the long-term strength loss does not begin to occur until near the end of the reinforcement design life.

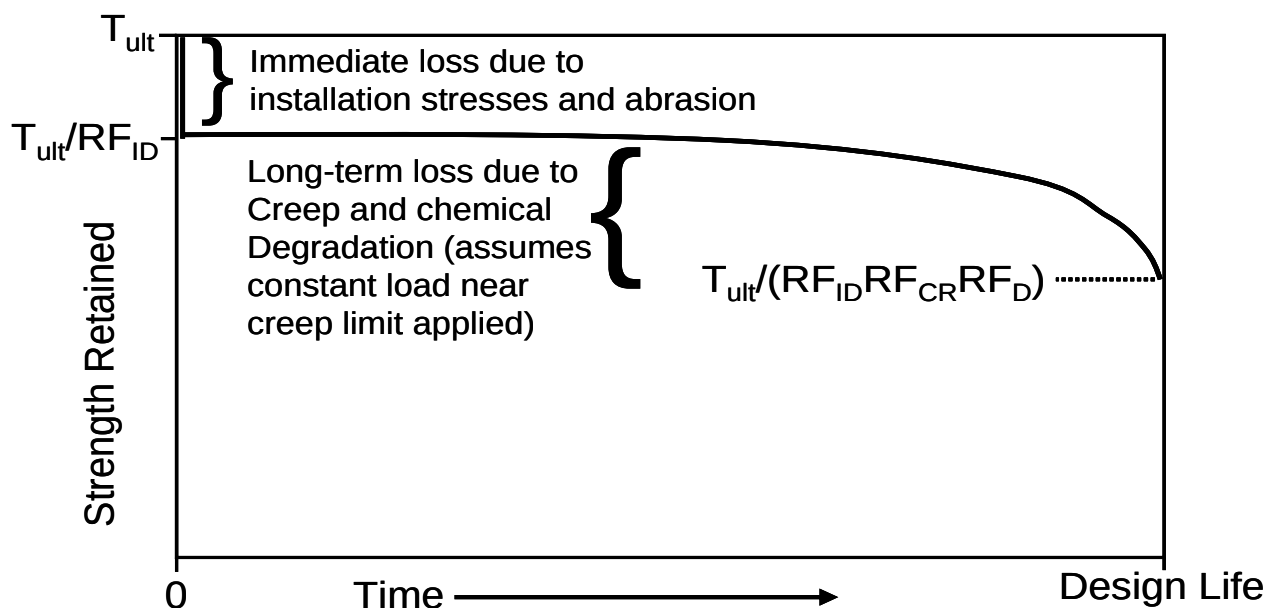


Figure 3-6. Long-term geosynthetic reinforcement strength concepts.

Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical agents. Therefore, each product must be investigated individually, or in the context of product line where the same polymer source and additives are used, and the manufacturing process is the same for all products in the product line. This product line approach makes it possible to interpolate reduction factors for products in the product line not specifically tested using the reduction factors determined for the products in the product line that are specifically tested for each degradation mechanism.

The AASHTO LRFD Bridge Design Specifications provide minimum requirements for the assessment of T_{al} for use in the design of geosynthetic reinforced soil structures. Protocols for evaluating T_{al} are included in Appendix D with supporting information on testing procedures provided in the companion Corrosion/Degradation document (Elias et al., 2009).

It is recommended that T_{al} values for specific products be determined from in-house, agency evaluation or third-party evaluation of independent test results such as the Highway Innovative Technology Evaluation Center (HITEC) or AASHTO National Transportation Product Evaluation Program (NTPEP). Agencies can approve reduction factors and allowable strength values based these reports or require that vendor designs use reduction factors substantiated by these or equivalent third party reports. Alternatively, T_{al} could be obtained directly from the manufacturer based on independent test results, though third party testing is the preferred approach. If manufacturer data is used, it should meet the same standard of quality and completeness that can be obtained from the third party testing programs such as NTPEP, and the designer should check to make sure that the manufacturer data are representative of the products likely to be received at the project site (i.e., the product test data should be current, and the product manufacturing process, polymer source, etc., should not have changed since the testing was conducted). In all cases, the geosynthetic product line must be reevaluated on a periodic to assess any changes that may affect the product and corresponding reduction values (e.g., NTPEP requires that the geosynthetic reinforcement product/product line be retested every 3 years).

In lieu of third party testing or manufacturer generated data, in-house agency testing to establish T_{al} with regard to the full suite of tests is generally not practical. However, agencies are encouraged to at least perform some of the index testing themselves, both for product qualification purposes (i.e., development of a qualified product or approved products list) as well as project specific product acceptance purposes. Agencies should also consider site specific installation damage testing, especially if relatively coarse, uniformly graded crushed or otherwise angular aggregate is used as backfill, or if other relatively severe installation conditions are anticipated.

The determination of reduction factors for each geosynthetic product and product line requires extensive field and/or laboratory testing which can take a year or more to complete. Background regarding the determination of each long-term strength reduction factor is briefly summarized as follows:

3.5.2.a Ultimate Tensile Strength, T_{ult}

The value selected for T_{ult} , for design purposes, is the minimum average roll value (MARV) for the product. This minimum average roll value, accounts for statistical variance in the material strength. Other sources of uncertainty and variability in the long-term strength result from installation damage, creep extrapolation, and the chemical degradation process. It is assumed that the observed variability in the creep rupture envelope is 100% correlated with the short-term tensile strength, as the creep strength is typically directly proportional to the short-term tensile strength within a product line. Therefore, the MARV of T_{ult} adequately takes into account variability in the creep strength. Note that the MARV of T_{ult} is the minimum certifiable wide width tensile strength provided by the product manufacturer.

3.5.2.b Installation Damage Reduction Factor, RF_{ID}

Damage during handling and construction, such as from abrasion and wear, punching and tear or scratching, notching, and cracking may occur in geosynthetics. These types of damage can only be avoided by using care during handling and construction. Construction equipment should not travel directly on geosynthetic materials.

Damage during reinforced fill placement and compaction operations is a function of the severity of loading imposed on the geosynthetic during construction operations and the size and angularity of the reinforced fill. For MSE walls and RSS construction, lightweight, low strength geotextiles and geogrids should be avoided to minimize damage with ensuing loss of strength.

Protocols for field testing for this reduction factor are detailed in the companion Corrosion/Degradation document (Elias et al., 2009) and in ASTM D-5818 (see also WSDOT T925). These protocols require that the geosynthetic material be subjected to a reinforced fill placement and compaction cycle, consistent with field practice. The ratio of the initial strength, to the strength of retrieved samples defines this reduction factor. For reinforcement applications, a minimum weight of 8.0 oz/yd² (270 g/m²) for geotextiles is recommended to minimize installation damage. This roughly corresponds to a Class 1 geotextile as specified in AASHTO M 288. In general, the combination of geosynthetic reinforcement, and backfill placement and gradation characteristics, should not result in a value of RF_{ID} greater than 1.7. If testing indicates that RF_{ID} will be greater than 1.7 (approximately a 40 percent strength loss), that combination of geosynthetic and backfill

conditions should not be used, as this or greater levels of damage will cause the remaining strength to be highly variable and therefore not adequately reliable for design.

Table 3-9 provides a summary of typical RF_{ID} values for a range of soil gradations and geosynthetic types.

In general, RF_{ID} is strongly dependent on the backfill soil gradation characteristics and its angularity, especially for lighter weight geosynthetics. Provided a minimum of 6 inches of backfill material is placed between the reinforcement surface and the compaction and spreading equipment wheels/tracks, the backfill placement and compaction technique will have a lesser effect on RF_{ID} . Regarding geosynthetic characteristics, the geosynthetic weight/thickness or tensile strength may have a significant effect on RF_{ID} . However, for coated polyester geogrids, the coating thickness may overwhelm the effect of the product unit weight or thickness on RF_{ID} .

Table 3-9. Installation Damage Reduction Factors.

Reduction Factor, RF_{ID}		
Geosynthetic	Type 1 Backfill Max. Size 4 in. (100mm) D_{50} about 1 1/4-in. (30 mm)	Type 2 Backfill Max. Size 3/4 -in. (20mm) D_{50} about #30 (0.7 mm)
HDPE uniaxial geogrid	1.20 - 1.45	1.10 - 1.20
PP biaxial geogrid	1.20 - 1.45	1.10 - 1.20
PVC coated PET geogrid	1.30 - 1.85	1.10 - 1.30
Acrylic coated PET geogrid	1.30 - 2.05	1.20 - 1.40
Woven geotextiles (PP&PET) ^a	1.40 - 2.20	1.10 - 1.40
Non woven geotextiles (PP&PET) ^a	1.40 - 2.50	1.10 - 1.40
Slit film woven PP geotextile ^a	1.60 - 3.00	1.10 - 2.00
a. Minimum weight 8.0 oz/yd ² (270 g/m ²).		

3.5.2.c Creep Reduction Factor, RF_{CR}

The creep reduction factor is required to limit the load in the reinforcement to a level known as the creep limit, that will preclude excessive elongation and creep rupture over the life of the structure. The creep limit strength is thus analogous to yield strength in steel. Creep is essentially a long-term deformation process. As load is applied, molecular chains move relative to each other through straightening out of folded or curved/kinked chains or through breaking of inter-molecular bonds, resulting in no strength loss, but increased elongation.

Eventually, if the load levels are sufficiently high i.e., constant load near the creep limit), the molecular chains can straighten/elongate no more without breaking the molecular chains. Significant strength loss occurs only when the straightening/slipping process is exhausted. If the load is high enough, molecular chains break, and both elongation and strength loss occur at an accelerating rate, eventually resulting in rupture. Generally this strength loss occurs only near the end of the design life of the geosynthetic under a given load level.

The creep reduction factor is obtained from long term laboratory creep testing as detailed in Appendix D. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. For creep testing one of two approaches may be used: 1) “conventional” creep testing per ASTM D5262, or 2) a combination of Stepped Isothermal Method (SIM) per ASTM D6992, which is an accelerated method using stepped increases in temperature to allow tests to be performed in a matter of days, and “conventional” creep testing. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep rupture limit) within the design life of the structure (e.g., several years for temporary structures, 75 to 100 years for permanent structures).

Typical ranges of RF_{CR} as a function of polymer type are:

<u>Polymer Type</u>	<u>Creep Reduction Factors</u>
Polyester (PET)	2.5 to 1.6
Polypropylene (PP)	5 to 4.0
High Density Polyethylene (HDPE)	5 to 2.6

3.5.2.d Durability Reduction Factor, RF_D

This reduction factor is dependent on the susceptibility of the geosynthetic to attack by chemicals, thermal oxidation, hydrolysis, environmental stress cracking, and microorganisms, and can vary typically from 1.1 to 2.0.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water must be available). Hydrolysis and the resulting fiber dissolution are accelerated in alkaline regimes, percent of water saturation in the surrounding soil, and temperature. Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen). The level of oxygen in reinforced fills is a function of soil porosity, groundwater location and other factors, and has been found to be slightly less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in-ground may proceed at a rate equal those used above ground. Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the reinforced fill as found in acid sulphate soils (e.g., pyrite), slag and cinder fills, other industrial wastes or mine tailings

containing transition metals, and elevated temperature. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin.

The relative resistance of polymers to these identified regimes is shown in Table 3-10 and a choice can be made, therefore, consistent with the in-ground regimes indicated.

Table 3-10. Anticipated Resistance of Polymers to Specific Environments.

<u>Soil Environment</u>	<u>Polymer</u>		
	<u>PET</u>	<u>PE</u>	<u>PP</u>
Acid Sulphate Soils	NE	ETR	ETR
Organic Soils	NE	NE	NE
Saline Soils pH < 9	NE	NE	NE
Ferruginous	NE	ETR	ETR
Calcareous Soils	ETR	NE	NE
Modified Soils/Lime, Cement	ETR	NE	NE
Sodic Soils, pH > 9	ETR	NE	NE
Soils with Transition Metals	NE	ETR	ETR
NE =No Effect			
ETR =Exposure Tests Required			

Most geosynthetic reinforcement is buried, and therefore ultraviolet (UV) stability is only of concern during construction and when the geosynthetic is used to wrap the wall or slope face. If used in exposed locations, the geosynthetic should be protected with coatings or facing units to prevent deterioration. UV tests (ASTM D4355) extended beyond the normal 500 hour test duration should be performed on materials that will be directly exposed for long periods of time (more than several months) in order to evaluate the materials anticipated design life. Vegetative covers can also be considered in the case of open weave geotextiles or geogrids. Thick geosynthetics with ultraviolet stabilizers can be left exposed for several years or more without protection; however, long-term maintenance should be anticipated because of both UV deterioration and possible vandalism.

Protocols for testing to obtain this reduction factor have been proposed and are detailed in FHWA RD-97-144 (Elias et al. 1999). In general, for polyolefins, they consist of oven aging polyolefins (PP and HDPE) samples to accelerate oxidation and measure their strength reduction, as a function of time, temperature and oxygen concentration. This high

temperature data must then be extrapolated to a temperature consistent with field conditions. For polyesters (PET) the aging is conducted in an aqueous media at varying pHs and relatively high temperature to accelerate hydrolysis, with data extrapolated to a temperature consistent with field conditions. For more detailed explanations, see the companion Corrosion/Degradation manual, FHWA NHI-09-087 (Elias et al., 2009).

Due to the long-term nature of these durability evaluation protocols (2 to 3 years could be required to complete such tests), it is generally not practical to conduct such tests for typical geosynthetic reinforcement design, but are generally more suited for research activities. However, short-term index type tests can be conducted as indicators of good long-term durability performance, based on correlation to the long-term research results obtained and reported by Elias et al. (1999). Such index test results, combined with a criteria applied to the test results that can be considered to indicate good long-term performance, can be used to justify the use of a default value for RF_D that can be used for the determination of T_{al} .

The following recommendations are stated in this companion document in regards to defining a RF_D factor. With respect to aging degradation, current research results suggest the following.

Polyester Geosynthetics

PET geosynthetics are recommended for use only in environments characterized by $3 < \text{pH} < 9$. The reduction factors for PET aging (RF_D) listed in Table 3-11 are developed for a 100-year design life in the absence of long-term product specific testing. Based on these research results, for polyester reinforcement, the AASHTO LRFD specifications recommend a minimum number average molecular weight of 25,000 and a maximum carboxyl end group content (CEG) of 30 to allow the use of a default reduction factor for durability.

Polyolefin Geosynthetics

To mitigate thermal and oxidative degradative processes, polyolefin (i.e., PP and HDPE) products are stabilized by the addition of antioxidants for both processing stability and long-term functional stability. These antioxidant packages are proprietary to each manufacturer and their type, quantity, and effectiveness varies. Without residual antioxidant protection (after processing), PP products are vulnerable to oxidation and significant strength loss within a projected 75 to 100 year design life at 20°C. Current data suggests that unstabilized PP has a half-life of less than 50 years.

Table 3-11. Durability (Aging) Reduction Factors for PET.

Product ^a	Durability Reduction Factor, RF _D	
	$5 \leq \text{pH} \leq 8$	$3^b < \text{pH} \leq 5$ $8 \leq \text{pH} < 9$
Geotextiles $M_n < 20,000$, $40 < \text{CEG} < 50$	1.6	2.0
Coated geogrids, Geotextiles $M_n > 25,000$, $\text{CEG} < 30$	1.15	1.3
M_n = number average molecular weight CEG = carboxyl end group <u>Notes:</u> a. Use of materials outside the indicated molecular property range requires specific product testing. Use of products outside of $3 < \text{pH} < 9$ range is not recommended. b. Lower limit of pH for permanent applications is 4.5 and lower limit for temporary applications is 3, per Article 11.10.6.4.2b (AASHTO, 2007).		

Therefore the anticipated functional life of a PP geosynthetic is to a great extent a function of the type and post-production antioxidant levels, and the rate of subsequent antioxidant consumption. Antioxidant consumption is related to the in-ground oxygen content, which in fills is only slightly less than atmospheric.

A detailed discussion of the effectiveness of oven aging and other protocols to allow estimation of long-term strength loss due to the combination of heat aging and oxidative degradation of various polyolefins is provided in Elias et al. (1999) and Elias et al. (2009). At present, index tests and associated test result criteria that can be considered indicative of sufficient long-term durability consist of shorter-term relatively high temperature oven aging tests (ENV ISO 13438:1999 and UV degradation tests (i.e., ASTM D4355). The current AASHTO LRFD specifications currently only specify a requirement for the UV test as an indirect indicator of the presence of long-term residual antioxidant protection, requiring polyolefins to have a minimum of 70 percent strength retained after 500 hours in a weatherometer per ASTM D4355. In addition, in Europe and in the NTPEP testing program, oven aging test are also required to justify the use of a default value for RF_D for polyolefins.

For both polyester and polyolefins, if these index test criteria are met, a default value for RF_D of 1.3 could be used to determine T_{al} for design purposes. These index criteria are summarized in Table 3-12. If the effective in-soil site temperature is anticipated to be approximately 85° F (30° C) plus or minus a few degrees, a higher default reduction factor for RF_D should be considered.

Environmental stress cracking is an aging phenomenon that is really as much related to creep as it is to durability. In certain environments, such as when surfactants are present, the creep rupture process, through making it easier for the tie molecules to pull out of the crystalline structure, can be accelerated, allowing cracks in the polymer to form, and premature rupture. Additional information on this phenomenon is provided in Elias et al. (2009). For most in ground conditions, the chemicals necessary to cause this to happen are generally not present, and the results from laboratory creep testing are sufficient to address strength loss under constant load.

Table 3-12. Minimum Requirements for Use of Default Durability Reduction Factors (RF_D) for Primary Geosynthetic Reinforcement.

Type	Property	Test Method	Criteria to Allow Use of Default RF
Polypropylene and Polyethylene	UV Oxidation Resistance	ASTM D4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyester	UV Oxidation Resistance	ASTM D4355	Min. 50% strength retained after 500 hrs. in weatherometer if geosynthetic will be buried within one week, 70% if left exposed for more than one week
Polypropylene and Polyethylene	Thermo-Oxidation Resistance	ENV ISO 13438:1999, Method A (Polypropylene) or B (Polyethylene)	Min. 50% strength retained after 28 days (PP) or 56 days (HDPE)
Polyester	Hydrolysis Resistance	Inherent Viscosity Method (ASTM D4603 and GRI Test Method GG8), or Determine Directly Using Gel Permeation Chromatography	Min. Number (Mn) Molecular Weight of 25,000
Polyester	Hydrolysis Resistance	GRI GG7	Max. Carboxyl End Group Number of 30
All Polymers	Survivability	¹ Weight per Unit Area, ASTM D5261	Min. 8 oz/yd ² (270 g/m ²)
All Polymers	% Post Consumer Recycled Material by Weight	Certification of Material used	Maximum 0%
¹ Alternatively, a default RF _D = 1.3 may be used if product specific installation damage testing is performed and it is determined that RF _{ID} is 1.7 or less, and if the other requirements in Table 3-12 are met.			

Note that biological degradation due to micro-organisms is rarely a concern, as most geosynthetic reinforcement products only contain high molecular weight polymers, and the biological agents have great difficulty in finding the molecular chain endings that would allow them to begin consuming the polymer. Therefore, biological degradation is usually not considered in the determination of RF_D .

3.5.2.e Durability Reduction Factor, RF_D , at Wall Face Unit

As noted in Section 4.4.7.i Connection Strength, the long-term environmental aging factor (RF_D) may be significantly different than that used in computing the in-soil nominal long-term reinforcement strength T_{al} . For these applications, it is recommended that the use of polyesters be limited to a pH range of > 3 and < 9 , as noted in Table 3-11.

Of particular concern is the use of polyester geogrid and geotextile reinforcements with concrete facings because of the potential high pH environment. PET geogrids and geotextiles should not be cast into concrete for connections, due to the potential for chemical degradation.

Use of PET reinforcements connected to dry-cast MBW units by laying the reinforcement between units may be subject to additional strength reductions. An FHWA sponsored field monitoring study to examine pH conditions within and adjacent to MBW units was performed (Koerner et al., 2000), which provided a large database of pH measurements of 25 MSE wall structures in the United States. The results indicated that the pH regime within the blocks in the connection zone is only occasionally above 9 and then for only the first few years. The pH subsequently decreases to the pH of the ambient backfill (Koerner et al., 2000). It therefore appears that for coated PET geogrids no further reduction is warranted. For geotextiles a small further reduction should be considered to account for a few years at a pH in excess of 9.

Caution is advised in situations where the MBW units will be saturated for extended periods of time such as structures in lakes or streams. For such cases, long-term pH tests should be performed on saturated block. If the pH exceeds 9, polyester reinforcements should not be used in the section of the structure.

3.5.2.f LRFD Geosynthetic Reinforcement Resistance Factor, ϕ

The resistance factor for geosynthetic reinforcement accounts for potential of local overstress due to load nonuniformity and uncertainties in long-term reinforcement strength. For Strength I limit state conditions, a resistance factor (ϕ) equal to 0.90 is used for geosynthetic reinforcements (see Table 4-7). This is higher than the resistance factors for steel reinforcements due to the ductile nature of geosynthetic systems at failure.

The recommended resistance factor of ϕ of 0.90 can be further justified by considering the following:

- For geosynthetic reinforcements, the reinforced fill soil controls the amount of strain in the reinforcement which for granular fills is limited to considerably less than the rupture strain of the reinforcement. Therefore even at a limit state, overstress of the geosynthetic reinforcement would cause visible, time-dependent strain in the wall system rather than sudden collapse.
- The long-term properties of geosynthetics, based on limited data, are significantly improved when confined in soil. Confinement is presently not considered in developing nominal long-term strength.
- Measurements of stress in MSE walls reinforced with geosynthetics have consistently indicated lower stress levels than used for design as developed in Chapter 4.

Note that T_{al} is used for RSS structures design with limit equilibrium analysis and computation of a factor of safety against instability.

3.5.2.g Preliminary Design Reduction Factor, RF For preliminary design of permanent structures or for applications defined by the user as not having severe consequences should poor performance or failure occur, the nominal long-term tensile strength T_{al} , may be evaluated without product specific data, as:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{7} \quad (3-13)$$

Further, $RF = 7$ should be limited (i.e., do not use Eq. 3-13 where following requirements are not met) to projects where the project environment meets the following requirements:

- Granular soils (sands, gravels) used in the reinforced volume.
- $4.5 \leq pH \leq 9$
- Site temperature $< 85^\circ \text{ F}$ (30° C)
- Maximum backfill particle size of $\frac{3}{4}$ -inch (19 mm)
- Maximum MSEW height is 35 ft (10 m) and
- Maximum RSS height is 50 ft (15 m)

Site temperature is defined as the temperature which is halfway between the average yearly air temperature and normal daily air temperature for the highest month at the site.

RF = 7 has been established by multiplying lower bound partial reduction factors obtained from currently available test data, for products which meet the minimum requirements in Table 3-13. *It should be noted that the total Reduction Factor may be reduced significantly with appropriate test data. It is not uncommon for products with creep, installation damage and aging data, to develop total Reduction Factors in the range of 3 to 6 or even less with the development of new materials.*

For temporary applications not having severe consequences should poor performance or failure occur, a default value for RF of 3.5 rather than 7 may be considered.

Table 3-13. Minimum Requirements for use of Preliminary Design Reduction Factor for Primary Geosynthetic Reinforcement.

Type	Property	Test Method	Criteria to Allow Use of Default RF
Polypropylene	UV Oxidation Resistance	ASTM D4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyethylene	UV Oxidation Resistance	ASTM D4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyester	Hydrolysis Resistance	Inherent Viscosity Method (ASTM D4603) with Correlation or Determine Directly Using Gel Permeation Chromatography	Min. Number (Mn) Molecular Weight of 25,000
Polyester	Hydrolysis Resistance	GRI GG7	Max. Carboxyl End Group Number of 30
All Polymers	Survivability	Weight per Unit Area, ASTM D5261	Min. 8 oz/yd ² (270 g/m ²)
All Polymers	% Post Consumer Recycled Material by Weight	Certification of Material used	Maximum 0%

3.5.2.h Serviceability Limit State

Serviceability limit state deflection requirements for geosynthetic reinforcements are met through the use of low stress levels resulting from reduction factors combined with the inherent constraining effects of granular soils. With regard to strain limits on the reinforcement, methods for estimating of strain vary widely with no present consensus on an appropriate analytical method capable of modeling strains in the structure. Measurements

from instrumented field structures have consistently measured much lower strain levels in the reinforcement (typically less than 1 percent) than predicted by most current analytical methods. **Therefore, until an appropriate method of determination is agreed upon, it is recommended that strain limit requirements not be imposed on the reinforcement.**

3.6 FACING MATERIALS

The material aspects of the various facings used with MSE walls structures are discussed below, by facing type. Typical dimensions, manufacturing process and controls, details, durability, and associated materials are discussed. Aesthetics were discussed in Chapter 2. Tolerances of precast panels to settlement were presented in Section 2.8.3. Design aspects of the more commonly used facings are addressed in Section 4.4.8. Specifications are addressed in Chapter 10.

3.6.1 Precast Concrete Panels

3.6.1.a Segmental Panels

Segmental, precast concrete panels are commonly square or rectangular in shape with typical dimensions of 5 to 8-in. (125 to 200 mm) thick and 5-foot (1.5-m) high and a front face width of 5 or 10-ft (1.5 or 3-m). Panels with cruciform, diamond, and hexagonal face geometry are also used. The panels are typically cast with the exposed face down, so they may have a smooth or a form-liner finish. Panels may also be prepared with an exposed aggregate finish. The edges of adjacent panels are cast with a butt, shiplap, or tongue-and-groove joint.

Agencies should check the raw materials, mix design, and precasting operation as they do for other precast, structural items. Generally, agencies have reviewed and approved these items for a particular precaster. Panels are usually produced by a local precaster for, and with forms provided by, the wall vendor. Form dimensions, concrete steel reinforcement placement, and connection hardware placement should be examined for conformance to the vendor's quality control and tolerances. The units must be fully supported until the concrete reaches a minimum compressive strength of 1,000 psi (6.9 MPa). The units may be shipped after reaching a minimum compressive strength of 3,400 psi (23.4 MPa). At the option of the contractor, the units may be installed after the concrete reaches a minimum compressive strength of 3,400 psi (23.4 MPa). The concrete must have a minimum 4,000 psi (27.6 MPa) compressive strength at 28 days. Temperature and tensile steel reinforcement should be designed in accordance with Section 5 of AASHTO LRFD Specifications for Highway Bridges (2007).

Metal connection hardware that is cast into the panel and extends out the back face of the panel for attachment to the soil reinforcement should not be placed in direct contact with the concrete steel reinforcement. This type of placement could accelerate corrosion of metal soil reinforcement. Direct contact is permissible if both have the same protection (e.g., galvanized).

Bearing pads are placed on all horizontal (and diagonal, if applicable) joints of adjacent segmental precast panels as they are erected. Two pads are usually used on 5-foot (1.5-m) wide panels and at least three bearing pads with 10-foot (3-m) wide panels. A minimum of two bearing pads are used per horizontal panel joint. The bearing pads are used to prevent or minimize point loadings or stress concentrations between adjacent panels, and to accommodate small vertical deformation of the panels as the wall height increases and the reinforced wall fill compresses.

Bearing pads shall meet or exceed the following material requirements:

- Preformed EPDM (Ethylene Propylene Diene Monomer) rubber pads conforming to ASTM D2000 Grade 2, Type A, Class A with a Durometer Hardness of 60 ± 5 .
- Preformed HDPE (High Density Polyethylene) pads with a minimum density of 0.946 grams per cubic centimeter in accordance with ASTM D 1505.

The stiffness (axial and lateral), size, and number of bearing pads must be determined such that the final joint opening is not less than the required joint width after compression (e.g., $\frac{1}{2}$ in.) unless otherwise shown on the plans. The MSE wall designer must submit substantiating calculations verifying the stiffness (axial and lateral), size, and number of bearing pads assuming, as a minimum, a vertical loading at a given joint equal to 2 times the weight of facing panels directly above that level. As part of the substantiating calculations, the MSE wall designer must submit results of certified laboratory tests in the form of vertical load-vertical strain and vertical load-lateral strain curves for the specific bearing pads proposed by the MSE wall designer. The vertical load-vertical strain curve should extend beyond the first yield point of the proposed bearing pad.

3.6.1.b Full-Height Panels

Typical dimensions of full-height panels are 6 to 8-in. (150 to 200-mm) thick and 8 or 10-ft (2.4 to 3-m) wide. Single, full-height panel walls have been constructed to a height of approximately 32 ft (10 m). Full-height panels are externally braced until the reinforced soil reaches $\frac{2}{3}$ to full height of the wall.

Full-height panels do not provide the same ability to adjust face panel alignment and rotation during construction, as do segmental panels. Nor are bearing pads used to accommodate elastic settlement of the reinforced fill so the connection detailing and strength must accommodate this deformation. Therefore, if the full-height panels will be used, Agencies should specify experience requirements for the wall vendor, wall designer (if different than the wall vendor), and the wall contractor. Additionally, the maximum height should be limited to about 32 feet (10 m), or less.

Agency controls are the same as for segmental, with the exception that taller, full-height panels have multiple heights of pick-up point hardware cast into the panel. Handling of the panels for shipping and erection should be monitored to ensure panels are not cracked by these operations.

No bearing pads are used with full-height panels. Therefore, high quality reinforced fill should always be used with full-height panel walls. Individual wall systems should address how the reinforcement connection is designed to tolerate elastic fill settlement.

3.6.2 Modular Block Wall Units

Modular block wall (MBW) MSE face units have typical dimensions of 4 to 15-in. (100 to 375-mm) high and 8 to 18-in. (200 to 450-mm) in exposed face length, and 8 to 24-in. (200 to 600-mm) in depth (perpendicular to wall face). MBW units are produced in a masonry manufacturing process. Therefore, the concrete is dry-cast, and unlike wet-cast panels cannot be air entrained or reinforced with steel. These units are also known as “segmental retaining wall” units.

There are a wide variety of commercially available MBW units, as noted in Section 2.4.3. These units are normally produced near the project site by a licensed manufacturer. Quality control requirements and quality assurance vary by licensor and licensee. Therefore, Agencies should control the raw materials, mix design, and casting operation as they do for wet-cast concrete, structural items. Form or cast units should be examined for dimensional tolerances. Many of these units have the face sheared off after casting to create a roughened, rock-like texture for aesthetic reasons.

Dry-cast concrete MBW units are susceptible to freeze-thaw degradation with exposure to deicing salts and cold temperatures. This is a concern in northern tier states that use deicing salts. Some vendors have developed mix designs, with additive(s), and manufacturing processes that result in units that are very durable and resistant to freeze-thaw degradation.

The current specifications in Chapter 10 have been developed to address this issue and clarify requirements depending on the susceptibility to freeze-thaw conditions and salt exposure.

Based on good performance experience by several agencies, ASTM C1372, Standard Specification for Segmental Retaining Wall Units should be used as a model, except that the compressive strength for units should be increased to 4,000 psi (28 MPa) to increase durability, maximum water absorption be limited to 5 percent, requirements for freeze-thaw testing modified, and tolerance limits expanded.

Note that more stringent durability requirements are being used by the Minnesota Department of Transportation (Mn/DOT) based upon their experience, research, climatic conditions and de-icing salt usage. The Mn/DOT criteria (2008) state that wall and cap units shall conform to ASTM C1372, except for the items in Table 3-14.

Several research projects investigating the freeze-thaw durability and degradation of MBW units have been performed. Reports are available from FHWA (Chan et al., 2007) and the University of Minnesota (Embacher et al., 2001a,b).

Freeze-thaw resistance of MBW units is tested following ASTM C1262. These tests generally take more than 3 months to perform. Therefore, the testing is not suited for approval of materials on an individual project basis. The testing is better suited to an agency evaluating and placing MBW units on an approved products list.

MBW units are erected using a running bond configuration. Full-height cores are filled with aggregate during erection units are normally dry-stacked (i.e. without mortar), and erected using a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys.

Geogrid soil reinforcement is typically used with MBW units, though some systems use geotextile and some use steel mat soil reinforcement. The soil reinforcement is connected to the MBW units via a frictional, mechanical, or combination mechanical and frictional-type connection. Bearing pads between vertically adjacent units are not used with MBW units. Therefore, the connection detailing and strength, and the soil placement and compaction must accommodate deformation caused by elastic compression of the reinforced fill. On certain systems, geosynthetic soil reinforcement sandwiched between vertically adjacent units provides some cushioning to distribute bearing loads between blocks.

**Table 3-14. Additional MBW Cold Weather Requirements
Recommended by Mn/DOT (2008).**

Item	Test Standard	<u>Requirement</u>
Compressive strength	ASTM C140, except that Section 6.2.4 shall be deleted and replaced with: “The specimens shall be coupons cut from a finished side or back shell of each unit and sawn to remove any face shell projections. The coupon size shall have a height to thickness ratio of 2 to 1 before capping and a length to thickness ratio of 4 to 1. The coupon shall be cut from the unit such that the coupon height dimension is in the same direction as the unit height dimension. Compressive testing of full size units will not be permitted. The compressive strength of the coupon shall be assumed to represent the net area compressive strength of the whole unit.”	5,500 psi (38 MPa) min. 5,800 psi (40 MPa) min. Average for 3 units
Freeze-thaw durability of wall units	The freeze/thaw durability of wall units tested in accordance with ASTM C1262 in a 3% saline solution shall be the minimum of the following:	(1) the mean weight loss of five test specimens at the conclusion of 90 cycles shall not exceed 1% of its initial weight; (2) the mean weight loss of the 4 lowest out of 5 test specimens at the conclusion of 100 cycles shall not exceed 1.5% of its initial weight. Test results shall be recorded and reported in 10 cycle intervals showing the weight of all specimens and not just the mean value.
Freeze-thaw durability of cap units	The freeze/thaw durability of cap units tested in accordance with ASTM C1262 in a 3% saline solution shall be the minimum of the following:	(1) the mean weight loss of five test specimens at the conclusion of 40 cycles shall not exceed 1% of its initial weight; (2) the mean weight loss of the 4 lowest out of 5 test specimens at the conclusion of 50 cycles shall not exceed 1.5% of its initial weight. Test results shall be recorded and reported in 10 cycle intervals showing the weight of all specimens and not just the mean value.
Cap unit	—	Top surface sloped at 1 inch fall per 10 inch run (1 mm fall per 10 mm run) front to back or crowned at the center.
Surface sealer	Contact Mn/DOT Concrete Engineering Unit, or www.mrr.dot.state.mn.us/pavement/concrete_products.asp , for requirements.	All segmental masonry retaining walls shall have their surfaces sealed. Apply surface sealer to the top, exposed front face, and backside of the upper three courses of all walls.

Therefore, it is recommended that agencies specify wall height experience requirements for the wall vendor, wall designer (if different than the wall vendor), and the wall contractor when MBW unit faced walls are to be used. Additionally, it is recommended that the maximum height typically be limited to about 32 feet (10 m), or less, unless setbacks are used to separate wall facing loads. Taller walls without setbacks require that bearing between units and possible stress concentrations due to geometric variations along the length of the wall be specifically addressed in the design and detailing. Typically, this can be accomplished with horizontal bearing pads or other compression members in the lower portion of the wall and/or vertical joints to separate geometric variations.

The use of polyester geogrid or geotextile soil reinforcements connected to the dry-cast MBW concrete units are discussed in Section 3.5.2.e. Recommendations for design as addressed in Section 3.5.3.e3, Durability Reduction Factor, RF_D , at the Wall Face Unit.

3.6.3 Welded Wire Mesh Facing

Welded wire mesh (WWM) is a popular facing for temporary walls and slopes, and is used in permanent walls and slopes. In permanent walls and slopes, the WWM may be the primary face soil retention element. For these cases, galvanized steel is used. The reinforcements in temporary structures should be galvanized if contact between reinforcements of the temporary structure and of a permanent (galvanized) structure is possible. In some permanent, geosynthetic-reinforced slopes and walls, the WWM is used as a forming device that is left in-place. The geosynthetic is the primary face soil retention element, and for these cases, plain (a.k.a, black) steel is typically used. A temporary WWM wall with a geotextile for retention at the face is shown in Figure 2-3.

Steel facings should be galvanized consistent with the use of galvanized reinforcements. Hot dip galvanizing of at least 2 oz/ft² should protect the steel in atmospheric conditions for a period between 20 and 50 years (AGA, 2004). Forty to 50 years are expected in rural and suburban environments, 25 to 30 years in coastal areas, and approximately 20 years if located in proximity to industrial areas where the atmosphere may be acidic. A typical corrosion rate for temporary, non-galvanized steel facing is 1.0 mil/yr (25 µm/yr). Substantially higher rates should be used if the wall face will be vegetated, where road salts are used, if atmospheric conditions are corrosive such as marine environments or when air quality may be compromised by nearby industrial activity. Corrosion potential can be reduced by using open graded stone in the facing. Note that a corrosion rate of 28 µm/yr should be applied to plain steel soil reinforcements, if the reinforced fill is not corrosive or only mildly aggressive, for temporary walls.

Hardware cloth that is sometimes used with welded wire facings to contain fill material may be vulnerable to corrosion (if steel) or degradation from UV radiation (if geosynthetic). Designers should assume that the hardware cloth will degrade over time, in permanent walls, and that the WWM will have to retain the wall fill adjacent to the face or maintenance (i.e., repair, replace) of the hardware cloth

For permanent walls, vertical and horizontal spacing of metallic reinforcements for flexible face (welded wire or similar) wall systems should not exceed 18 inches. The stiffness of the facing and spacing of reinforcements must be such that the maximum local horizontal deformation between soil reinforcement layers is limited to less than 1 to 2 in. as specified by the agency. The maximum local horizontal deformation between soil reinforcement layers should also be limited to less than 1 to 2 in. for temporary walls, i.e., walls with up to 36 months service life. This recommendation is particularly important if the temporary wall will be incorporated into a permanent feature, e.g., buried within an embankment fill.

The look of galvanized WWM face may not be desired on some projects due to aesthetic requirements. On some projects, staining of galvanized WWM has been used to achieve desired aesthetics.

3.6.4 Geosynthetic Wrap-Around Facing

Geosynthetic facing elements should not be left exposed to sunlight (specifically ultraviolet radiation) for permanent walls. If geosynthetic facing elements must be left exposed to sunlight, for permanent or temporary structures, the geosynthetic must be stabilized to be resistant to ultraviolet radiation. Furthermore, product specific test data should be provided which can be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment. Vegetative covers provide some protection from UV and in many cases, a healthy vegetative cover can prevent exposure altogether. Alternately, a protective facing must be constructed in addition (e.g., concrete, shotcrete, etc.). A temporary wrap-around wall is shown in Figure 2-3.

3.6.5 Other Facings

Other facings being used on permanent walls, and sometimes on slopes, include: large, up to 3-ft (0.9-m) high and 3 to 10-ft (0.9 to 3-m) in width, wet cast concrete units, gabions, and geocells.

The large wet cast units are typically stacked, similar to MBW units. Generally, geosynthetic soil reinforcements are used with these units. The reinforcement is usually connected to the

facing by friction, i.e., sandwiched between vertically adjacent units, as shown in Figure 3-7. Attachments may also be cast into the units and mechanical connection used, as shown in Figure 3-8.

Rock filled gabions are another large face unit used with MSE walls. One system uses woven-wire soil reinforcement that is integral with the gabion face, so no connection is required. Other systems connect reinforcement to the facing by friction by sandwiching the reinforcement between vertically adjacent units. Connecting the reinforcement by mechanically clipping it to the back of a gabion should be avoided. Most gabions are 3 ft by 3 ft (0.9 by 0.9 m), thus vertical reinforcement exceeds the 32 in. (0.8 m) recommended maximum spacing. This greater spacing may be offset by the size/mass of the facing. Although 36-in. reinforcement spacing has been used successfully on many projects, it is not in agreement with the 32-in. limit to ensure a coherent MSE mass. The Owner should exercise caution in the evaluation of the maximum reinforcement spacing when specific loading conditions, unusual geometries, or soft foundation exist. The Owner and/or wall designer should consider use of secondary reinforcement layers placed in at the center of the unit heights to reduce reinforcement vertical spacing.

Geocells are used to face reinforced soil walls and slopes. Eight-inch (200-mm) high geocells and nominally about 3 ft (0.9 m) wide are typically used. Connection to the soil reinforcement is by friction, i.e., sandwiched between vertically adjacent mats of geocells. The lifts of geocells may be offset and the outer cells filled with topsoil and vegetated, as shown in Figure 3-9.



Figure 3-7. Large, wet-cast concrete face unit with reinforcement placed between units.



Figure 3-8. Large, wet-cast concrete face unit with embedded reinforcement connectors.



Figure 3-9. Geocell face unit with vegetation.

3.6.6 Two-Stage Facings

Two-stage MSE wall construction is used to construct walls on foundations that will undergo significant settlement. The first stage is construction of an MSE wall with a flexible facing (i.e., WWM or geosynthetic wrap). Connectors or form anchors are embedded in the first stage construction. The foundation soils are allowed to settle under the load of the first stage, with or without an additional surcharge load. The second stage consists of facing the first stage with cast-in-place or precast concrete panels. Either full height or segmental precast panels are used and are mechanically connected to the first-stage reinforced soil mass. Connection mechanisms and details may be proprietary to the wall vendor. For cast in place facings, the design of the connection mechanism must consider fluid pressure that develops during pouring of the concrete, which may require staging to avoid connection overstressing.

Precast material control is discussed in Section 3.6.1. Design issues include; 1) estimation of settlement and establishing tolerance limits for the first-stage wall construction, 2) estimating additional long term settlement after construction of the second stage including additional loading from the facing system, and 3) evaluating the long-term durability of the connection hardware between the concrete and MSE mass with consideration for long term differential settlement. Corrosion needs to be addressed for steel connectors and durability for any geosynthetic connectors.

CHAPTER 4

DESIGN OF MSE WALLS

This chapter details design guidelines common to all MSE wall structures. It is limited to MSE walls having a near-vertical face, and uniform length of soil reinforcements. MSE wall design details are addressed in Chapter 5. Design guidelines for complex structures, or structures with unusual features are covered in Chapter 6. Detailed example calculations for both routine and complex structures are presented in Appendix E of this reference manual.

This chapter is organized sequentially as follows:

- Overview of design methods.
 - o LRFD
 - o Other methods
- Loads and load combinations.
 - o LRFD design of MSE walls
- MSE wall design guidelines (step-by-step)
 - o Sizing for external stability
 - o Sizing for internal stability
- Temporary walls
- Design checklist
- Computer aided design
- Standard MSE wall designs

4.1 DESIGN METHODOLOGY AND ANALYSIS METHODS

4.1.1 Load and Resistance Factor Design (LRFD) Platform

Traditionally, the MSE wall design has been performed using the Allowable Stress Design (ASD) methodology. The LRFD methodology is the latest advancement in transportation structures design practice. The LRFD method in various forms is now being applied throughout the world. For example, EuroCode uses the limit state design (LSD) methodology, which is very similar to the LRFD methodology. Regardless of the design methodology, the core analytical methods for MSE walls such as external and internal stability evaluation remain unchanged. The assumption of a coherent gravity mass for external stability, the shape of the internal failure planes, and treatment of reinforcements as discrete elements remains unchanged. The primary change is in the way the loads and resistances are compared and how uncertainty is incorporated into the design process.

Specific to the topic of MSE walls the following points regarding LRFD methodology should be noted to prevent any confusion in application of the various theories and equations presented in this chapter:

- The symbol ϕ (phi) is used for both the soil friction angle and the LRFD resistance factor.
- The symbol γ (gamma) is used for both soil unit weight and the LRFD load factor.
- Load and resistance factors for MSE walls are currently calibrated by fitting to ASD results. Therefore, designs using LRFD procedure should not significantly vary from past, expected ASD designs.
- For most MSE wall system designs, strength limit states generally control the member sizes. Service limit states may control aspects such as joint width openings and construction sequence based on the anticipated deformations. Extreme event limit states may affect both the member sizes as well as deformations.

4.1.2 Analysis Methods

As noted earlier, the core analysis methods for MSE walls are unchanged relative to ASD practice. AASHTO (2002), which is based on the ASD method, recommended the use of the *Simplified Method* (a.k.a., *Simplified Coherent Gravity Method*) provided in the previous version of this manual. {Note: The AASHTO (2002) and FHWA (Elias et al., 2001) ASD references will not be updated by AASHTO or FHWA, respectively.}

It is acknowledged that other analytical methods are also available in the literature as follows:

- Allowable Stress Design (ASD) Procedure and the Simplified Method (AASHTO, 2002 and FHWA NHI-00-043 {Elias et al., 2001})
- Coherent Gravity Method Analysis Model
- National Concrete Masonry Association (NCMA) Procedure (NCMA, 2009)
- Geosynthetic Reinforced Soil (GRS) Method (Wu et al., 2006)
- K-Stiffness Method (Allen and Bathurst, 2003; Allen et al., 2003; Allen et al., 2004; WSDOT, 2006; and Bathurst et al., 2008a)

The LRFD methodology permits consideration of any of the above methods as long as appropriate calibrations are performed for resistance factors using acceptable quality statistical data. This chapter concentrates on application of the Simplified Method which is recommended due to its applicability to a variety of soil reinforcement types (in contrast to the limited applicability of the alternative methods to specific type of reinforcements, e.g., GRS method is strictly applicable to geosynthetic reinforcements), and it is a methodology that has been successfully used in practice for many years. Brief descriptions of these other analytical methods are included in Appendix F.

4.2 LOADS AND LOAD COMBINATIONS

A complete list of various loads, load factors and load combinations that need to be considered in design of bridge structures and associated transportation structures such as retaining walls and culverts is presented in Section 3 of AASHTO (2007). Many load types are commonplace to design of bridge structures and not applicable to retaining walls as noted in Section 11 of AASHTO (2007). With respect to MSE wall structures, only a few of the loads and load combinations are applicable on a routine basis. The applicable loads for most MSE wall applications are summarized below followed by a summary of applicable load combinations in Tables 4-1 and 4-2. Complete load combination and load factor tables (per AASHTO, 2007) are contained in Appendix A.

Applicable Loads

Permanent Loads

- EH = Horizontal earth loads
- ES = Earth surcharge load
- EV = Vertical pressure from dead load of earth fill

Transient Loads

- CT = Vehicular collision force
- EQ = Earthquake load
- LL = Vehicular live load
- LS = Live load surcharge

An example of an ES load on an MSE wall is the pressure from a spread footing above the reinforced mass. An example EV load is a sloping fill above the top of an MSE wall. Further distinction is made under the external and the internal design steps that follow.

Table 4-1. Typical MSE Wall Load Combinations and Load Factors
(after Table 3.4.1-1, AASHTO {2007}).

Load Combination Limit State	EH ES EV	LL LS	Use One of These at a Time	
			EQ	CT
STRENGTH I	γ_p	1.75	—	—
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	—
EXTREME EVENT II	γ_p	0.50	—	1.00
SERVICE I	1.00	1.00	—	—
Notes: γ_p = load factor for permanent loading. May subscript as γ_{p-EV} , γ_{p-EH} , etc. γ_{EQ} = load factor for live load applied simultaneously with seismic loads				

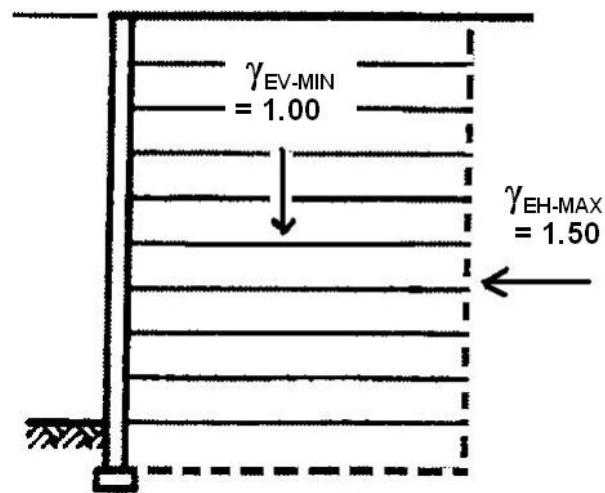
Table 4-2. Typical MSE Wall Load Factors for Permanent Loads, γ_p
(after Table 3.4.1-2, AASHTO {2007}).

Type of Load	Load Factor	
	Maximum	Minimum
DC: Component and Attachments	1.25	0.90
EH: Horizontal Earth Pressure • Active	1.50	0.90
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments	1.00 1.35	N/A 1.00
ES: Earth Surcharge	1.50	0.75
Note: May subscript as γ_{EV-MIN} , γ_{EV-MAX} , γ_{EH-MIN} , γ_{EH-MAX} , etc.		

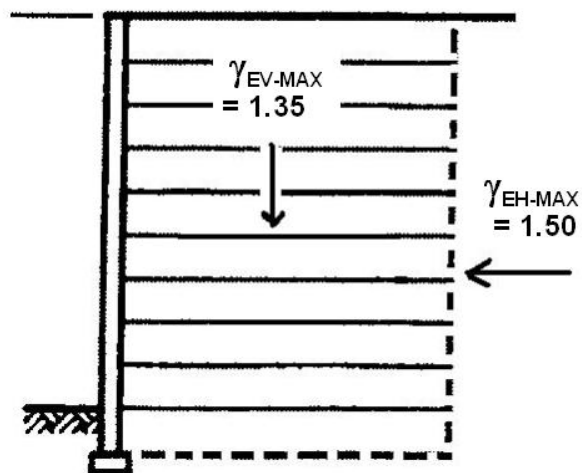
Maximum and Minimum Load Factors

Two load factors, a maximum and a minimum, are listed in Table 4-2. It is important to understand the application of these load factors in context of MSE walls. Article 3.4.1 AASHTO (2007) states that: *“The factors shall be selected to produce the total extreme factored force effect. For each load combination, both positive and negative extremes shall be investigated. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect. For permanent force effects, the load factor that produces the more critical combination shall be selected. . . . Where the permanent load increases the stability or load-carrying capacity of a component or bridge, the minimum value of the load factor for that permanent load shall also be investigated.”*

In general, AASHTO's guidance can be applied by using minimum load factors if permanent loads increase stability and use maximum load factors if permanent loads reduce stability. For simple walls, e.g., level backfill with or without surcharges due to traffic, or sloping backfill, the load factor (minimum or maximum) to use for a particular stability check may be readily identifiable. The load factors to use for such simple walls for external stability calculations are illustrated in Figure 4-1. The maximum EV load factor should be used for internal stability calculations.



a. Typical load factors for sliding stability and eccentricity check.



b. Typical load factors for bearing calculations.

Figure 4-1. External stability load factors for simple walls.

The basic concept of load combinations using maximum and minimum load factors is applicable to more complex MSE wall configurations, such as those that may be experienced at bridge abutments or walls with complex geometries (see Chapter 6). Therefore, different combinations of load factors will need to be investigated to determine the total extreme factored (critical) force effect for each applicable limit state.

While the positive and negative extremes are the two bounds, an intermediate combination of maximum and minimum load factors can create the critical force effect for design purposes. This is particularly applicable to retaining walls, wherein various components within a wall system may separately experience maximum or minimum loads. For example, in MSE walls, while the reinforced soil mass may be constructed such that it results in a maximum load, it is conceivable that the construction of retained fill may be at a minimum load level. Therefore, a critical combination of loads needs to be evaluated based on applicable maximum and minimum load factors. The detailed design examples complex MSE wall configurations in Appendix E use the concept of using minimum and maximum load factors.

4.3 DESIGN OF MSE WALLS USING LRFD METHODOLOGY

The procedure for design of MSE walls using LRFD methodology is very similar to that using ASD methodology. In LRFD, the external and internal stability of the MSE wall is evaluated at all appropriate strength limit states and overall stability and lateral/vertical wall movement are evaluated at the service limit state. Extreme event load combinations are used to design and analyze for conditions such as vehicle impact and seismic loading (see Chapter 7 for extreme event design). The specific checks for the strength and service limit states required for MSE wall design are listed below.

Strength Limit States for MSE walls

- External Stability
 - Limiting Eccentricity
 - Sliding
 - Bearing Resistance
- Internal Stability
 - Tensile Resistance of Reinforcement
 - Pullout Resistance of Reinforcement
 - Structural Resistance of Face Elements
 - Structural Resistance of Face Element Connections

Service Limit States for MSE walls

- External Stability
 - o Vertical Wall Movements
 - o Lateral Wall Movements

Global Stability of MSE walls

- Overall Stability
- Compound Stability

The external stability of an MSE wall is evaluated assuming that the reinforced soil zone acts as a rigid body. This is because, when properly designed, the wall facing and the reinforced soil act as a coherent block with lateral earth pressures acting on the back side of that block.

The internal stability of the reinforced soil zone is dependent on three fundamental characteristics:

- the soil-reinforcement interaction (resistance to pullout and to sliding, for sheet-type reinforcements);
- the tensile resistance of the reinforcement; and
- the durability of the reinforcing material.

Therefore, the internal stability analyses of an MSE wall in LRFD is evaluated by (a) determining the maximum factored load in each reinforcement and (b) comparing this maximum factored load to the factored pullout resistance and to the factored tensile resistance of the reinforcement for all applicable strength, service, and extreme event limit states.

Capacity to Demand Ratio (CDR)

With LRFD, the goal is to have the factored resistance greater than the factored load. The term capacity to demand ratio, CDR, is used to quantify the ratio of the factored resistance to the factored load. This term is useful in identifying critical and controlling limit states.

4.3.1 Design Steps

There are eleven basic design steps for an MSE wall, as listed in Table 4-3. Some of these steps have several sub-steps in the design process. These steps are for walls with simple geometries, as discussed in this chapter. Steps can vary somewhat depending on on type of reinforcement and/or whether or not type of reinforcement is initially defined. Additional steps are required for more complex cases such as true bridge abutments, as discussed in Chapter 6.

Table 4-3. Basic LRFD Design Steps for MSE Walls.

Step 1.	Establish Project Requirements – including all geometry, loading conditions (permanent, transient, seismic, etc.), performance criteria, and construction constraints.	
Step 2.	Establish Project Parameters – evaluate existing topography, site subsurface conditions, reinforced wall fill properties, and retained backfill properties.	
Step 3.	Estimate Wall Embedment Depth, Design Height(s), and Reinforcement Length	
Step 4.	Define nominal loads	
Step 5.	Summarize Load Combinations, Load Factors, and Resistance Factors	
Step 6.	Evaluate External Stability	
	a.	Evaluate sliding
	b.	Evaluate eccentricity
	c.	Evaluate bearing on foundation soil
	d.	Settlement analysis (at service limit state)
Step 7.	Evaluate Internal Stability	
	a.	Select type of soil reinforcement
	b.	Define critical failure surface (for selected soil reinforcement type)
	c.	Define unfactored loads
	d.	Establish vertical layout of soil reinforcements
	e.	Calculate factored horizontal stress and maximum tension at each reinforcement level.
	f.	Calculate nominal and factored long-term tensile resistance of soil reinforcements
	g.	Select grade (strength) of soil reinforcement and/or number of soil reinforcement elements at each level.
	h.	Calculate nominal and factored pullout resistance of soil reinforcements, and check established layout
	i.	Check connection resistance requirements at facing
	j.	Estimate lateral wall movements (at service limit state)
	k.	Check vertical movement and compression pads
Step 8.	Design of Facing Elements	
Step 9.	Assess Overall Global Stability	
Step 10.	Assess Compound Stability	
Step 11.	Design Wall Drainage Systems.	
	a.	Subsurface drainage
	b.	Surface drainage

4.4 MSE WALL DESIGN GUIDELINES

4.4.1 Step 1 – Establish Project Requirements

Prior to proceeding with the design, the following parameters must be defined:

- Geometry
 - Wall heights
 - Wall batter
 - Backslope
 - Toe slope
- Loading Conditions
 - Soil surcharges
 - Live (transient) load surcharges
 - Dead (permanent) load surcharges
 - Loads from adjacent structures that may influence the internal or external stability of MSE wall system, e.g., spread footings, deep foundations, etc.
 - Seismic
 - Traffic barrier impact
- Performance Criteria
 - Design code (e.g., AASHTO LRFD)
 - Maximum tolerable differential settlement
 - Maximum tolerable horizontal displacement
 - Design life
 - Construction Constraints

The chosen performance criteria should reflect site conditions and agency or AASHTO code requirements, which are discussed in detail in Chapters 2 and 3 of this manual.

4.4.2 Step 2 – Establish Project Parameters

The following must be defined by the agency (Owner) and/or its designer:

Existing and proposed topography

- Subsurface conditions across the site
 - Engineering properties of foundation soils (γ_f , c'_f , ϕ'_f , c_u)
 - Groundwater conditions
- Reinforced wall fill – engineering properties of the reinforced soil volume (γ_r , ϕ'_r)
- Retained backfill – engineering properties of the retained fill (γ_b , c'_b , ϕ'_b), addressing all possible fills (e.g., in-situ, imported, on-site, etc.). Cohesion in the retained backfill is usually assumed to be equal to zero. See FHWA Earth Retaining Structures reference

manual (Tanyu et al., 2008) for guidance on value of cohesion and calculation of the lateral pressure if a cohesion value is used in design.

Note that AASHTO uses the subscript f for both the foundation and retained backfill soils. In the text of this reference manual, the subscript f is used for foundation soil and subscript b is used for the retained backfill.

The reinforced wall fill should be a select granular material, as detailed in Chapter 3 of this manual and in Article 7.3.6.3 AASHTO LRFD Bridge Construction Specifications (2004). Per Article 11.10.6.2 (AASHTO, 2007) the maximum friction angle of the select granular reinforced fill should be assumed to be 34° , unless the project specific fill is tested for frictional strength by triaxial or direct shear testing methods. A design friction angle greater than 40° should not used, even if the measured friction angle is greater than 40° . Note, that while 34° is a maximum value in absence of testing, some soils such as semi-rounded to round, uniform sands, that meet the specified gradation have a friction angle lower than 34° . In geologic areas where such soils are found (e.g., Florida, Wisconsin, Minnesota, etc.), it is recommended that project specific fill shear strength tests be performed. Similarly, where soils are micaceous, project specific shear strength tests should be performed. Also note, it is assumed that the select granular reinforced fill is noncohesive, i.e., cohesion is assumed equal to zero.

For the foundation soil, Article 11.10.5.3 (AASHTO, 2007) notes that in absence of specific data, a maximum friction angle, ϕ'_f , of 30° may be used. The use of an assumed, non-specific parameter is recommended only for preliminary sizing. As discussed in Chapter 2, a project specific site evaluation, that defines subsurface conditions and properties, is required for design of MSE wall structures.

An assumed friction angle, ϕ'_b , of 30° is often used for the retained (i.e., behind the reinforced zone) backfill. The use of an assumed, non-specific parameter is recommended only for preliminary sizing. As discussed in Chapter 2, a project specific site evaluation, that defines subsurface conditions and properties, is required for the design of MSE wall structures; or the use of a backfill specification that assures that the minimum friction angle is obtained. Most agencies have defined allowable property ranges for the retained fill (may be classified as an *embankment* fill material) and have appropriate friction angle(s) established for design.

4.4.3 Step 3 – Estimate Wall Embedment Depth and Reinforcement Length

The process of sizing the structure begins by determining the required embedment, established under Project Criteria (Section 2.8.3, see Table 2-2), and the final exposed wall height, the combination of which is the full design height, H , for each section or station to be investigated. Use of the full height condition is required for design as this condition usually prevails in bottom-up constructed structures, at least to the end of construction.

A preliminary length of reinforcement is chosen to initiate design. The length should be the greater of $0.7H$ or 8 ft (2.5 m), where H is the design height of the structure. Structures with sloping surcharge fills or other concentrated loads, such as abutments, generally require longer reinforcements for stability, often on the order of $0.8H$ to $1.1H$ (see Table 2-1). This preliminary reinforcement length is checked in the external and the internal stability calculations.

Generally, the reinforcement length should be uniform throughout the entire height of the wall. One exception is special structures with shorter reinforcement lengths at the base of the wall; these are addressed in Chapter 6. Another exception is the use of longer layers of reinforcement at the top of a wall. It is recommended that the upper two layers of soil reinforcement be extended 3 ft (0.9 m) beyond the other layers where post-construction movements at the reinforced zone and retained backfill have been observed on previous, similar projects or if a seismic loading could lead to tension cracks in the backfill soil immediately behind the reinforcement. The design can be completed assuming uniform lengths, and the extra length added to the top two layers when detailing and specifying.

The 8 ft (2.5 m) minimum is used to accommodate the typical size of fill spreading and compaction equipment used on transportation works. As noted in Commentary C.11.10.2.1 AASHTO (2007), a minimum soil reinforcement length, on the order of 6.0 ft (1.8 m) can be considered for short walls if smaller compaction equipment is used and other wall design requirements are met. But, the minimum of $0.7H$ should be maintained. This shorter minimum length of 6 ft (1.8 m) is generally used only for landscape features (e.g., walls not supporting traffic).

4.4.4 Step 4 – Define Nominal Loads

The primary sources of external loading on an MSE wall are the earth pressure from the retained backfill behind the reinforced zone and any surcharge loadings above the reinforced zone. Thus, the loads for MSE walls may include loads due to horizontal earth pressure (EH), vertical earth pressure (EV), live load surcharge (LS), and earth surcharge (ES). Water

(WA) and seismic (EQ) should also be evaluated if applicable. Stability computations for walls with a near vertical face are made by assuming that the MSE wall acts as a rigid body with earth pressures developed on a vertical pressure plane at the back end of the reinforcements, as shown in Figures 4-2, 4-3, and 4-4. Estimation of earth pressures on MSE walls for three different conditions (i.e., horizontal backslope with traffic surcharge, sloping backslope, and broken backslope) follows.

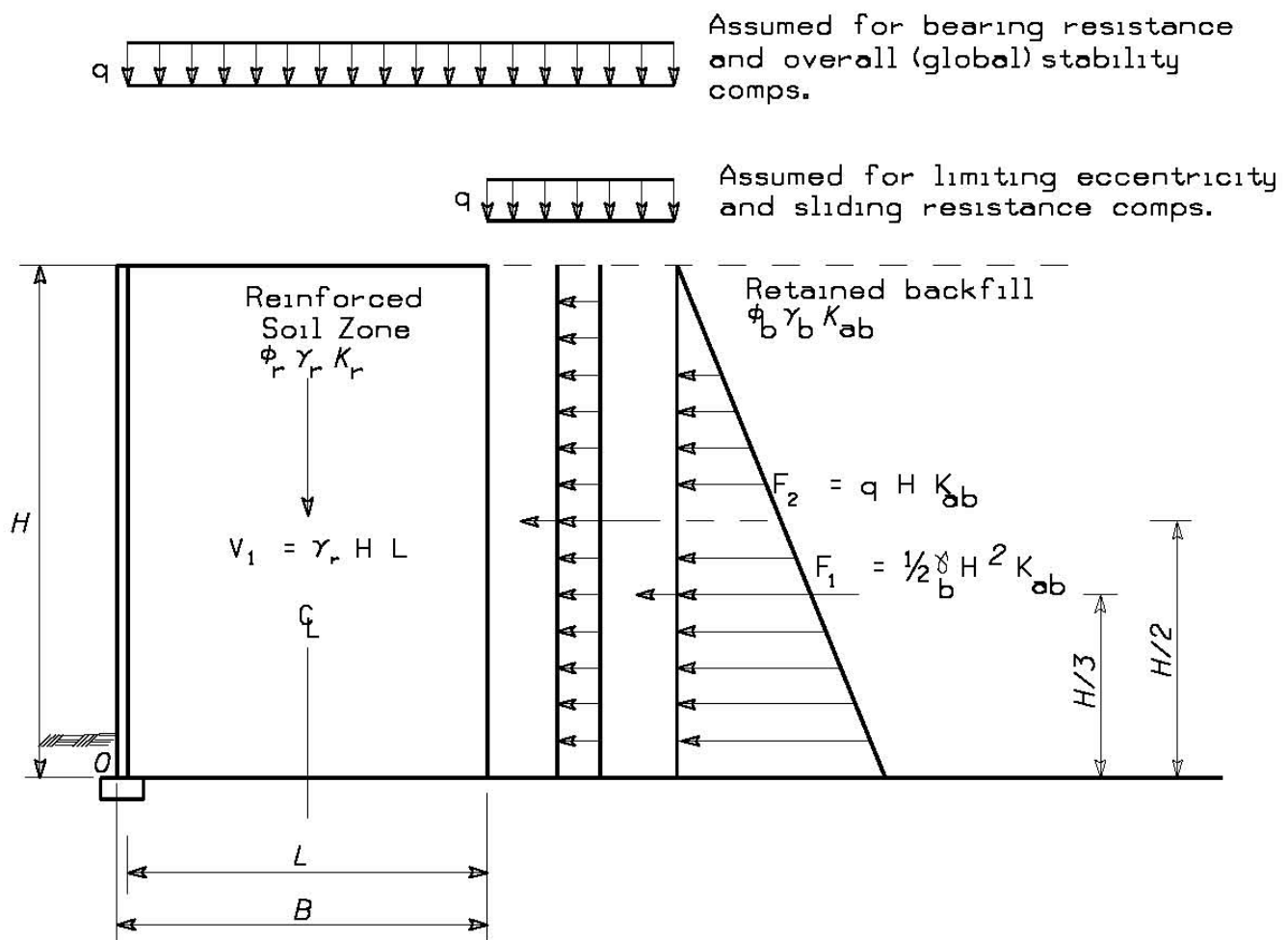


Figure 4-2. External analysis: nominal earth pressures; horizontal backslope with traffic surcharge (after AASHTO, 2007).

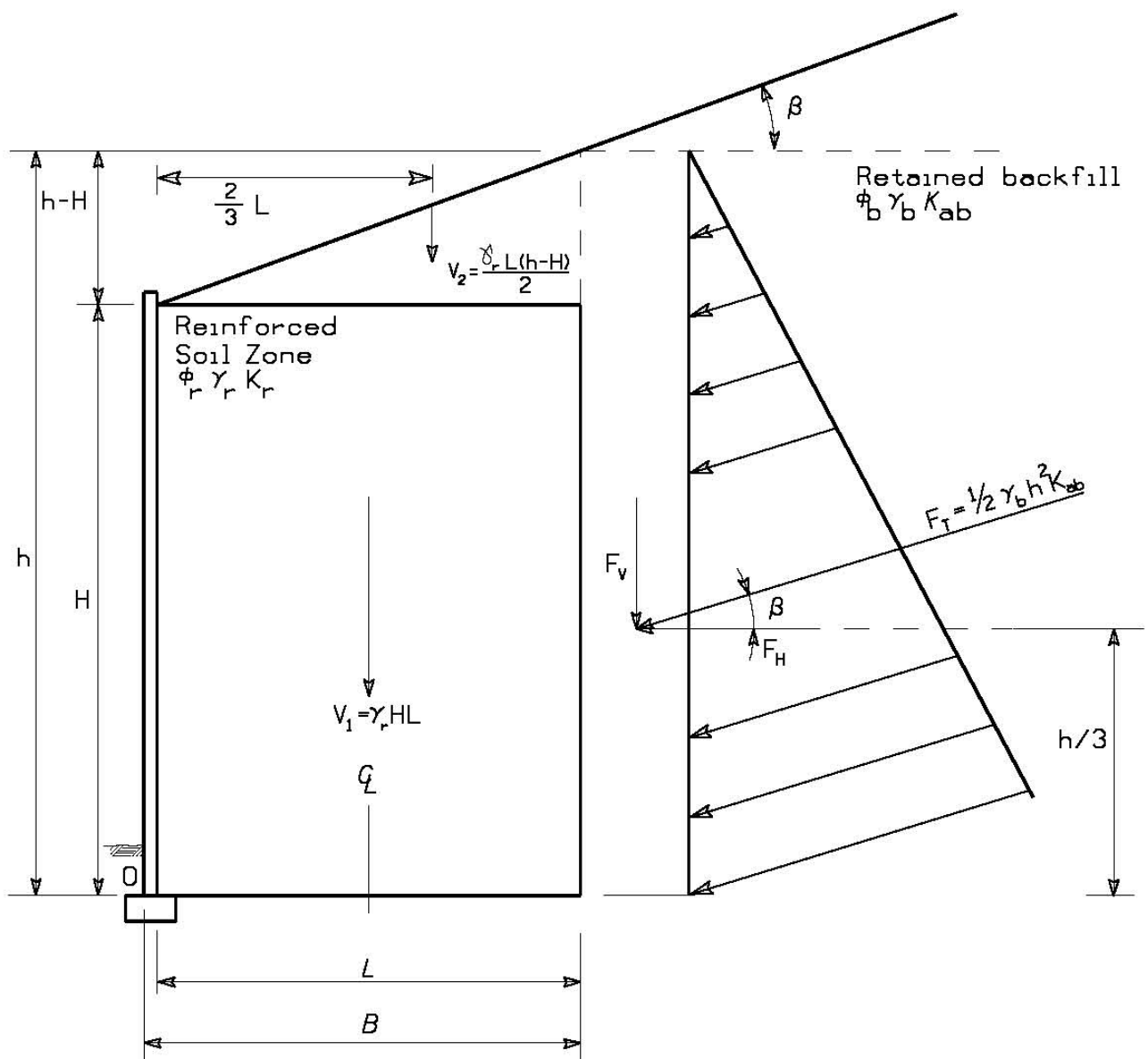


Figure 4-3. External analysis: earth pressure; sloping backfill case (after AASHTO, 2007).

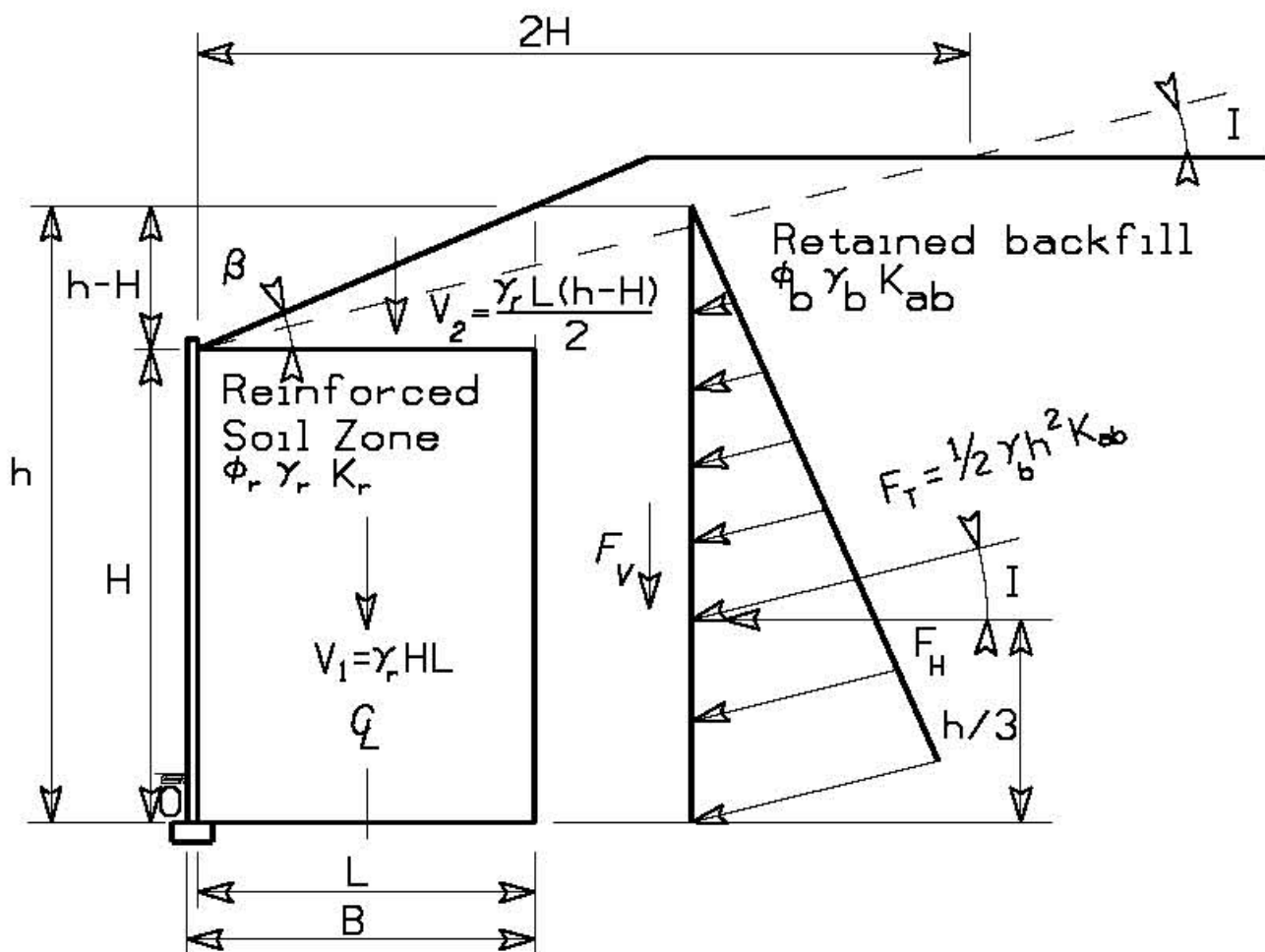


Figure 4-4. External analysis: earth pressure; broken backslope case (after AASHTO, 2007).

Vertical Wall and Horizontal Backslope: The active coefficient of earth pressure is calculated for near vertical walls (defined as walls with a face batter of less than 10 degrees from vertical) and a horizontal backslope from:

$$K_{ab} = \tan^2 \left(45 - \frac{\phi'_b}{2} \right) \quad (4-1)$$

where: ϕ'_b = friction angle of retained backfill.

Vertical Wall and a Surcharge Slope: The active coefficient of earth pressure is calculated for near vertical walls (defined as walls with a face batter of less than 10 degrees from vertical) and a sloping backfill from:

$$K_{ab} = \frac{\sin^2 (\theta + \phi'_b)}{\Gamma \sin^2 \theta \sin (\theta - \delta)} \quad (4-2)$$

where:

$$\Gamma = \left[1 + \sqrt{\frac{\sin (\phi'_b + \delta) \sin (\phi'_b - \beta)}{\sin (\theta - \delta) \sin (\theta + \beta)}} \right]^2 \quad (4-3)$$

β = Nominal slope of backfill behind wall (deg)

δ = Angle of friction between retained backfill and reinforced soil, set equal to β (deg)

ϕ'_b = effective friction angle of retained backfill (deg)

θ = 90° for vertical, or near (< 10°) vertical, wall (deg)

Note that the earth pressure force, (F_T) in Figure 4-3, is oriented at the same angle as the backslope, β , as it is assumed that $\delta = \beta$.

Vertical Wall with Broken Backslope: The active earth pressure coefficient (K_a) for this condition is computed using Equations 4-2 and 4-3, with the design β angle and the interface angle δ both set equal to I, as defined in Figure 4-4.

Battered Wall with or without Backslope: For an inclined front face and reinforced zone (i.e., batter) equal or greater than 10 degrees from vertical, the coefficient of earth pressure can be calculated using Equations 4-2 and 4-3 where θ is the face inclination from horizontal, and β the surcharge slope angle as shown in Figure 4-5. The wall friction angle δ is assumed to be equal to β .

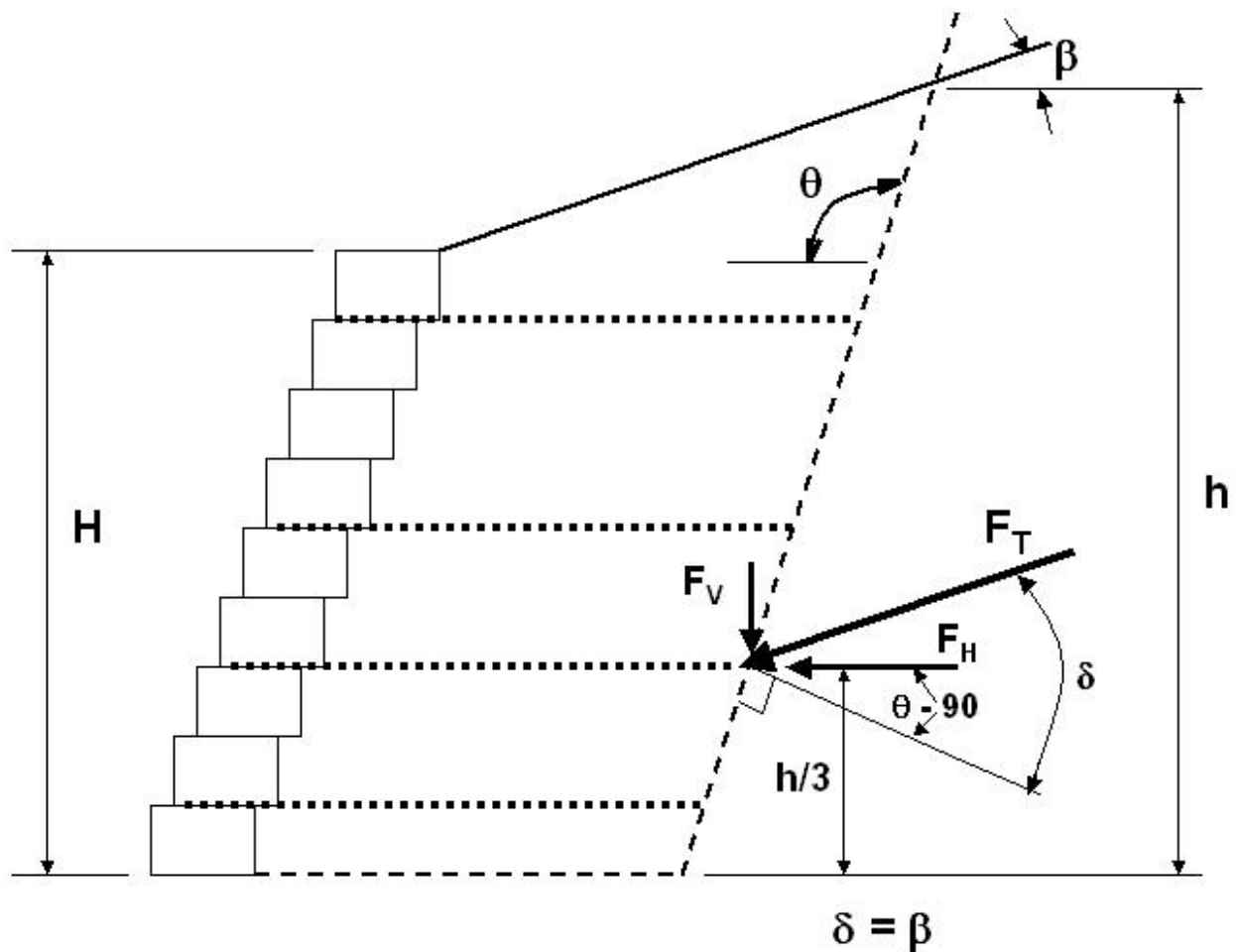


Figure 4-5. Notation for Coulomb active earth pressures used with wall batters, θ , greater than 100° (after AASHTO, 2007).

Traffic Loads

Traffic loads should be treated as uniform surcharge live load of not less than 2.0 ft (0.6 m) of earth (Article 11.10.10.2, AASHTO {2007}). For external stability, traffic load for walls parallel to traffic will have an equivalent height of soil, h_{eq} equal to 2.0 ft. For internal stability, traffic load for walls parallel to traffic will have a h_{eq} , equal to 2.0 ft unless traffic is allowed within 1.0 ft of the back of the wall facing. Commonly the wheel path is more than 1-ft behind the wall backface due to the presence of a traffic barrier and, therefore, a h_{eq} value of 2 ft is applicable.

Equivalent heights of soil, h_{eq} , for uniform surcharge loadings on retaining wall abutments with traffic running perpendicular to the wall may be taken from Table 4-4. Linear

interpolation is used for intermediate wall heights. Typically, the abutment h_{eq} will be acting on the stub abutment that sits on top of the reinforced soil zone (see Figure 4-13). If a structural approach slab is used and is supported on the backwall of the abutment (and not by the soil), the load is directly transmitted to the abutment; in this case $h_{eq} = 0$ is used unless otherwise mandated by an owner.

If the surcharge is for other than highway vehicular loading, the owner should specify or approve different surcharge load.

Table 4-4. Equivalent Height of Soil, h_{eq} , for Traffic Loading on Abutments Perpendicular to Traffic (Table 3.11.6.4-1, AASHTO {2007}).

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥ 20.0	2.0

Soil Compaction-Induced Earth Pressures

Compaction stresses are already included in the design model and specified compaction procedures for MSE walls (Article C3.11.2, AASHTO {2007}). Therefore, no additional design considerations are required.

4.4.5 Step 5 – Summarize Load Combinations, Load Factors, and Resistance Factors

Load combinations were discussed in Section 4.2, and typically may include Strength I, Extreme I and/or II, and Service I limits. Note however, that in certain states, the Strength II limit state is more critical than the Strength I limit state because owner prescribed legal loads are greater than those provided in the AASHTO specifications (2007). Maximum permanent loads, minimum permanent loads, and total extremes should be checked for a particular load combination for walls with complex geometry and/or loadings to identify the critical loading. Examination of only the critical loading combination, as described in Section 4.2, is sufficient for simple walls. Load factors typically used for MSE walls are listed in Tables 4-1 and 4-2. Refer to the information in Appendix E or Section 3 of AASHTO (2007) for load factors to use with complex MSE wall configurations and loadings.

Live loads are not used on specific design steps since they contribute to stability. These are identified in subsequent design steps.

Resistance factors for external stability and for internal stability are presented in respective design step discussions that follow. Internal stability resistance factors are listed later in Table 4-7.

4.4.6 Step 6 – Evaluate External Stability

As with classical gravity and semigravity retaining structures, four potential external failure mechanisms are usually considered in sizing MSE walls, as shown in Figure 4-6. They include:

- Sliding on the base
- Limiting eccentricity (formerly known as overturning)
- Bearing resistance
- Overall/global stability (see Step 8)

The resistance factor for external stability analyses of MSE walls are listed in Table 4-5.

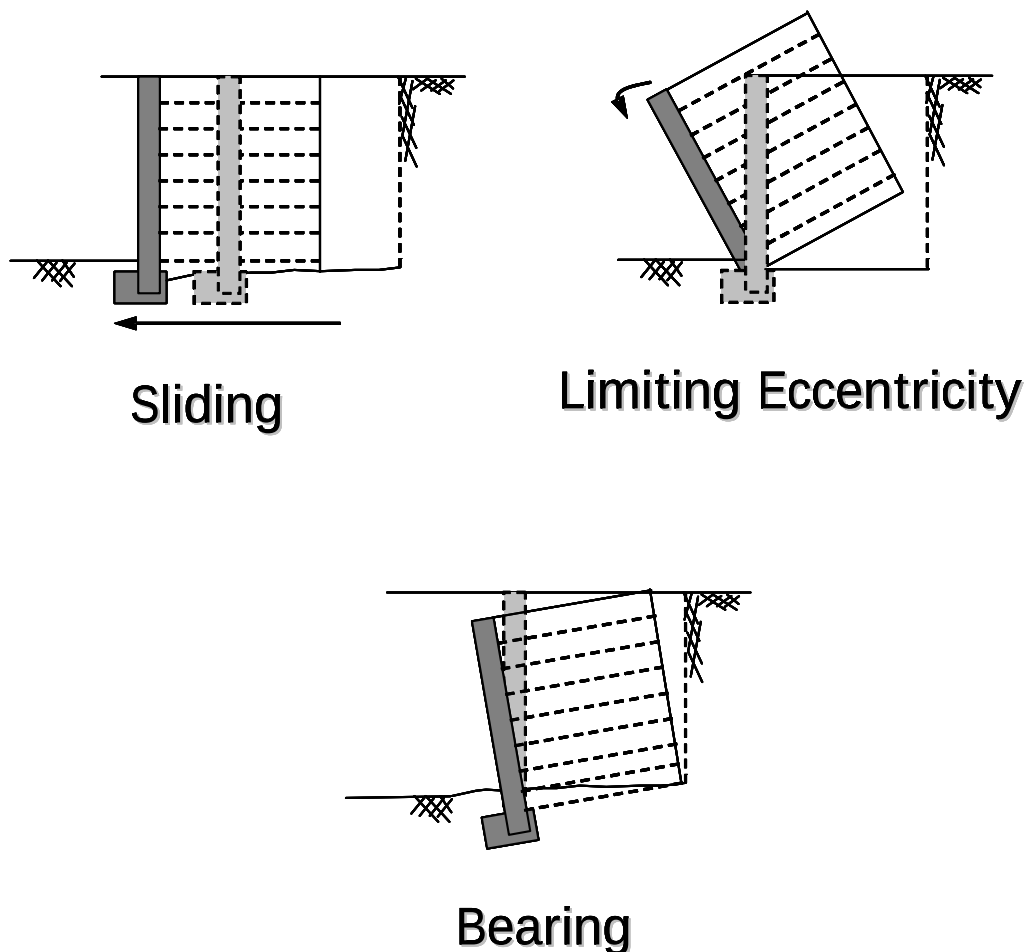


Figure 4-6. Potential external failure mechanisms for a MSE wall.

Table 4-5. External Stability Resistance Factors for MSE Walls
(Table 11.5.6-1, AASHTO {2007}).

Stability Mode	Condition	Resistance Factor
Bearing Resistance		0.65
Sliding		1.0
Overall (global) Stability	Where geotechnical parameters are well defined, and the slope does not support or contain a structural element	0.75
	Where geotechnical parameters are based on limited information, or the slope contains or supports a structural element	0.65

4.4.6.a Evaluate Sliding Stability

Check the preliminary sizing with respect to sliding of the reinforced zone where the resisting force is the lesser of the shear resistance along the base of the wall or of a weak layer near the base of the MSE wall, and the sliding force is the horizontal component of the thrust on the vertical plane at the back of the wall (see Figures 4-2 through 4-4). The live load surcharge is not considered as a stabilizing force when checking sliding, i.e., the sliding stability check only applies the live load above the retained backfill, as shown in Figure 4-2. The driving forces generally include factored horizontal loads due to earth, water, seismic, and surcharges.

Sliding resistance along the base of the wall is evaluated using the same procedures as for spread footings on soil as per Article 10.6.3.4 (AASHTO, 2007). The factored resistance against failure by sliding (R_R) can be estimated by:

$$R_R = \phi_\tau R_\tau \quad (4-4)$$

where:

ϕ_τ = resistance factor for shear resistance between soil and foundation
(equal to 1.0 for sliding of soil-on-soil, see Table 4-5)

R_τ = nominal sliding resistance between reinforced fill and foundation soil

Note that any soil passive resistance at the toe due to embedment is ignored due to the potential for the soil to be removed through natural or manmade processes during its service life (e.g. erosion, utility installation, etc.). Also, passive resistance is usually not available during construction. The shear strength of the facing system is also conservatively neglected.

Calculation steps and equations to compute sliding for two typical cases follow. These equations should be extended to include other loads and geometries, for other cases, such as additional live and dead load surcharge loads.

- 1) Calculate nominal thrust, per unit width, acting on the back of the reinforced zone.

Wall with Horizontal Backslope: (see Figure 4-2)

The retained backfill resultant, F_1 , is:

$$F_1 = \frac{1}{2} K_{ab} \gamma_b H^2 \quad (4-5)$$

For a uniform surcharge, the resultant is:

$$F_2 = K_{ab} q H \quad (4-6)$$

where:

K_{ab} = active earth pressure coefficient for the retained backfill

γ_b = moist unit weight of the retained backfill soil

H = height of the retaining wall

q = uniform live load surcharge = $(\gamma_r)(h_{eq})$

Wall with Sloping Backfill: (see Figure 4-3)

Calculate nominal retained backfill force resultant per unit width, F_T

$$F_T = \frac{1}{2} K_{ab} \gamma_b h^2 \quad (4-7)$$

where:

K_{ab} = active earth pressure coefficient for the sloping backfill, see Eq. 4-2

h = total height of wall, H , and slope at the back of the reinforced zone
 $= H + L \tan \beta$

For a broken backslope (see Figure 4-4), $h - H$ should not exceed the height of the upper crest. If the broken backslope height is defined as “S”, then $(H + L \tan \beta) \leq (H + S)$; use $(H + S)$ if $(L \tan \beta) > S$.

- 2) Calculate the nominal and the factored horizontal driving forces. For a horizontal backslope and uniform live load surcharge:

$$\sum \mathbf{F} = \mathbf{F}_1 + \mathbf{F}_2 \quad (4-8)$$

$$\mathbf{P}_d = \gamma_{EH} \mathbf{F}_1 + \gamma_{LS} \mathbf{F}_2 \quad (4-9)$$

For a sloping backfill condition:

$$\mathbf{F}_H = \mathbf{F}_T \cos\beta \quad (4-10)$$

$$\mathbf{P}_d = \gamma_{EH} \mathbf{F}_H = \gamma_{EH} \mathbf{F}_T \cos\beta \quad (4-11)$$

Use the maximum EH load factor (= 1.50) in these equations because it creates the maximum driving force effect for the sliding limit state.

- 3) Determine the most critical frictional properties at the base. Choose the minimum soil friction angle, ϕ for three possibilities:
 - i) Sliding along the foundation soil, if its shear strength (based on $c'_f + \tan \phi'_f$ and/or c_u for cohesive soils) is smaller than that of the reinforced fill material shear strength ($\tan \phi'_r$).
 - ii) Sliding along the reinforced fill (ϕ'_r).
 - iii) For sheet type reinforcement, sliding along the weaker of the upper and lower soil-reinforcement interfaces. The soil-reinforcement friction angle ρ , should preferably be measured by means of interface direct shear tests. In absence of testing, it may be taken as $\frac{2}{3} \tan \phi'_r$.
- 4) Calculate the nominal components of resisting force and the factored resisting force per unit length of wall. For a horizontal backslope and uniform live load surcharge, the live load is excluded since it increases sliding stability:

$$\mathbf{R}_r = \gamma_{EV} \mathbf{V}_1 \times \mu \quad (4-12)$$

For a sloping backfill condition:

$$\mathbf{R}_r = [\gamma_{EV} (\mathbf{V}_1 + \mathbf{V}_2) + \gamma_{EH} (\mathbf{F}_T \sin\beta)] \times \mu \quad (4-13)$$

where

μ = minimum soil friction angle ϕ [$\tan \phi'_f$, $\tan \phi'_r$, or (for continuous reinforcement) $\tan \rho$]

External loads that increase sliding resistance should only be included if those loads are permanent.

Use the minimum EV load factor (= 1.00) in these equations because it results in minimum resistance for the sliding limit state.

- 5) Compare factored sliding resistance, R_r , to the factored driving force, P_d , to check that resistance is greater.
- 6) Check the capacity demand ratio (CDR) for sliding, $CDR = R_r/P_d$. If the $CDR < 1.0$, increase the reinforcement length, L , and repeat the calculations.

4.4.6.b Eccentricity Limit Check

The system of forces for checking the eccentricity at the base of the wall is shown on Figure 4-7. It should be noted that the weight and width of the wall facing is typically neglected in the calculations. Limiting eccentricity is a strength limit state check. The eccentricity limit check only applies the live load above the retained backfill, as shown in Figure 4-2.

The eccentricity, e , is the distance between the resultant foundation load and the center of the reinforced zone (i.e., $L/2$), as illustrated in Figure 4-7. The quantity e is calculated by summing the overturning and the resisting moments about the bottom, center of the base length, and dividing by the vertical load.

$$e = \frac{\sum M_D - \sum M_R}{\sum V} \quad (4-14)$$

Equations to compute eccentricity for two typical cases follow. These equations should be extended to include other loads and geometries, for other cases.

Wall with Horizontal Backslope: Calculation steps for the determination of the eccentricity beneath a wall with a horizontal backslope and a uniform live load surcharge are as follows, with respect to Figure 4-7.

Calculate nominal retained backfill and surcharge force resultants per unit width. See Equations 4-5 and 4-6 for walls with a horizontal backslope and uniform live load surcharge. See Equation 4-7 for walls with sloping backfill.

For a vertical wall, with horizontal backslope and uniform live load surcharge, calculate the eccentricity e as follows:

$$e = \frac{\gamma_{EH-MAX} F_1 \left(\frac{H}{3}\right) + \gamma_{LS} F_{q-Ls} \left(\frac{H}{2}\right)}{\gamma_{EV-MIN} V_1} \quad (4-15)$$

Wall with Sloping Backfill: The eccentricity beneath a wall with a sloping backfill, and no surcharges, is calculated as follows, with respect to Figure 4-8.

Calculate e with factored loads. For a wall with a sloping backfill the eccentricity is equal to:

$$e = \frac{\gamma_{EH-MAX} F_T \cos\beta \left(\frac{h}{3}\right) - \gamma_{EH-MAX} F_T \sin\beta \left(\frac{L}{2}\right) - \gamma_{EV-MIN} V_2 \left(\frac{L}{6}\right)}{\gamma_{EV-MIN} V_1 + \gamma_{EV-MIN} V_2 + \gamma_{EH-MAX} F_T \sin\beta} \quad (4-16)$$

Eccentricity Check Criteria: The eccentricity, e , is considered acceptable if the calculated location of the resultant vertical force (based on factored loads) is within the middle one-half of the base width for soil foundations (i.e., $e_{max} = L / 4$) and middle three-fourths of the base width for rock foundations (i.e., $e_{max} = 3/8 L$). Therefore, for each strength limit load group, e must be less than e_{max} . If e is greater, than a longer length of reinforcement is required.

Examination of only the critical loading combination, as describe in Section 4.2, (i.e., use the minimum EV and maximum EH load factors) is sufficient for simple walls. Maximum permanent loads, minimum permanent loads, and total extremes should be checked for complex (geometry and/or loadings) walls to identify the critical loading.

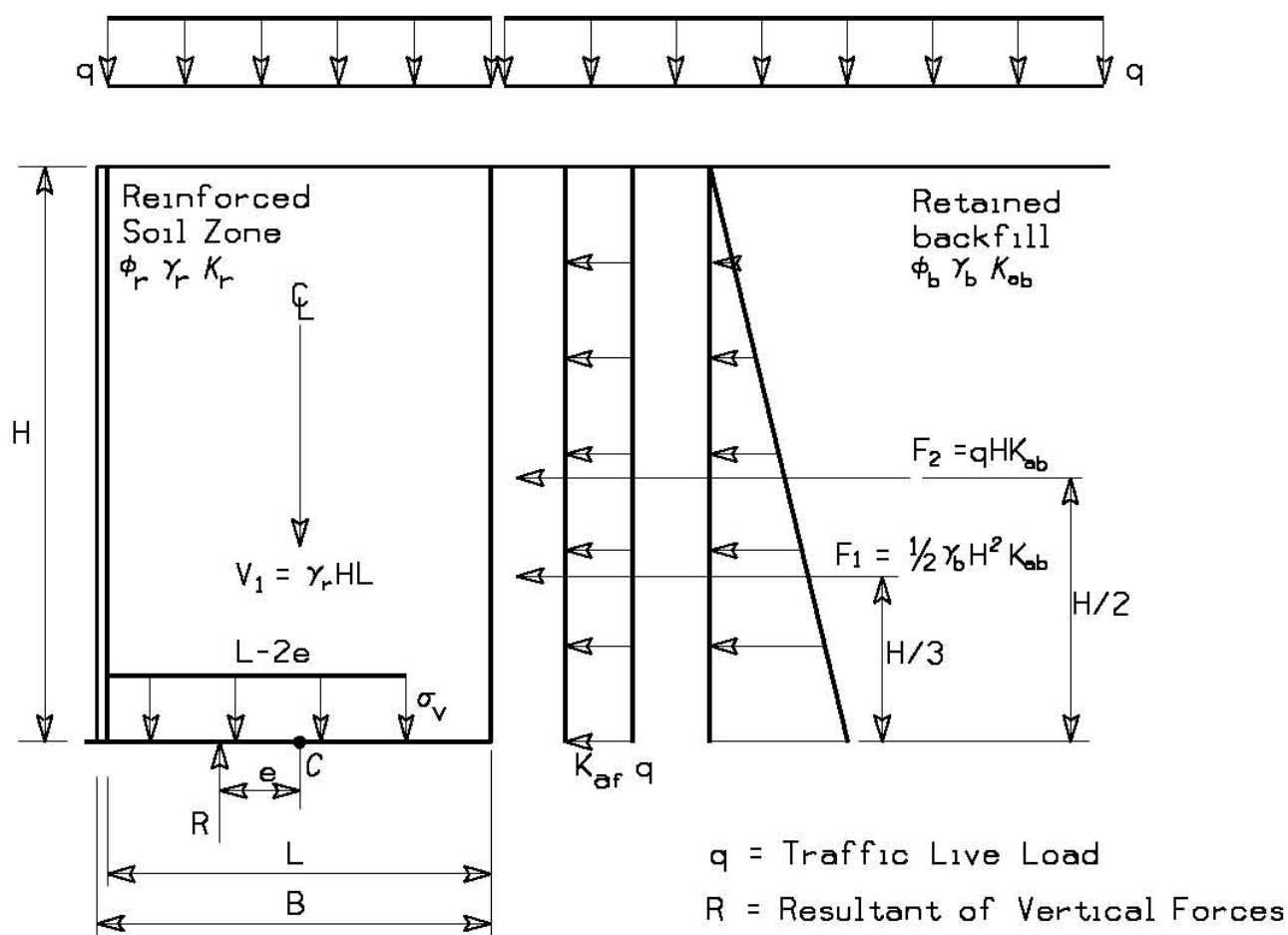


Figure 4-7. Calculation of eccentricity and vertical stress for bearing check, for horizontal backslope with traffic surcharge condition.

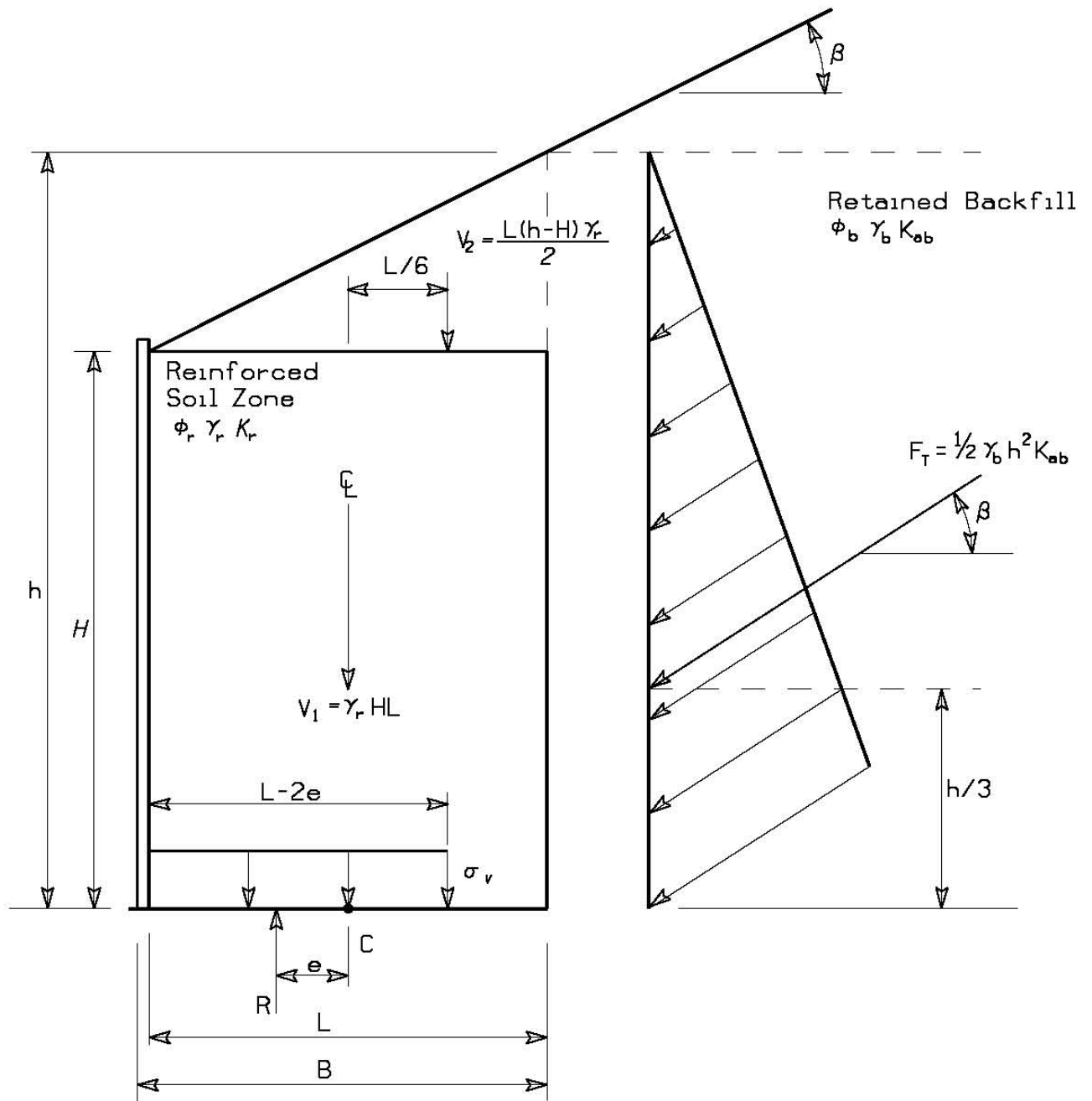


Figure 4-8. Calculation of eccentricity and vertical stress for bearing check, for sloping backslope condition.

4.4.6.c Evaluate Bearing on Foundation

Two modes of bearing capacity failure exist, general shear failure and local shear failure. Local shear is characterized by a punching or *squeezing* of the foundation soil when soft or loose soils exist below the wall.

Bearing calculations require both a strength limit state and a service limit state calculation. Strength limit calculations check that the factored bearing pressure is less than the factored bearing resistance. Service limit calculations are used to compute nominal bearing pressure for use in settlement calculations. It should be noted that the weight and width of the wall facing is typically neglected in the calculations. The bearing check applies live load above both the reinforced zone and the retained backfill, as shown in Figure 4-2.

General Shear. To prevent bearing failure on a uniform foundation soil, it is required that the factored vertical pressure at the base of the wall, as calculated with the uniform Meyerhof-type distribution, does not exceed the factored bearing resistance of the foundation soil:

$$q_R \geq q_{\text{uniform}} \quad (4-17)$$

The uniform vertical pressure is calculated as:

$$\sigma_v = \frac{\sum V}{L - 2e_B} \quad (4-18)$$

where:

ΣV = summation of vertical forces

L = width of foundation, equal to reinforcement length L

e_B = eccentricity for bearing calculation (not equal to eccentricity check e)

This step, 6.c, requires a different computation of the eccentricity value computed in Step 6.a because different, i.e., maximum in lieu of minimum, load factor(s) are used. Also note that the bearing check applies the live load above both the reinforced zone and the retained backfill, as shown in Figure 4-2. In addition to walls founded on soil, a uniform vertical pressure is also used for walls founded on rock due to the flexibility of MSE walls and their limited ability to transmit moment (Article C11.10.5.4 {AASHTO, 2007}).

Calculation steps for MSE walls with either a horizontal backslope and uniform live load surcharge and for sloping backfills follow. Again, note that these equations should be extended to include other loads and geometries, for other cases.

- 1) Calculate the eccentricity, e_B , of the resulting force at the base of the wall. The e value from the eccentricity check, Step 6.a, cannot be used, Calculate e with factored loads. For a wall with horizontal backslope and uniform live load surcharge centered about the reinforced zone, the eccentricity is equal to:

$$e_B = \frac{\gamma_{EH-MAX} F_1 \left(\frac{H}{3} \right) + \gamma_{LS} F_{q-Ls} \left(\frac{H}{2} \right)}{\gamma_{EV-MAX} V_1 + \gamma_{LS} q L} \quad (4-19)$$

where terms were previously defined. The maximum load factors for γ_{EH} and γ_{EV} are used to be consistent with the computation for σ_v (below) where maximum load factors results in the maximum vertical stress.

For walls with sloping backfill see Equation 4-16. Again, note that these equations should be extended to include other loads and geometries, for other cases.

Note that when checking the various load factors, and load combinations, the value of eccentricity, e_B , will vary. Also note that when the calculated value of eccentricity, e_B , is negative, a value of 0 should be carried forward in the design stress equation, i.e., set $L' = L$, per AASHTO C11.10.5.4 (2007).

- 2) Calculate the factored vertical stress σ_{v-F} at the base assuming Meyerhof-type distribution. For a horizontal backslope and uniform live load surcharge the factored bearing pressure is:

$$\sigma_{v-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{LS} q L}{L - 2e_B} \quad (4-20)$$

This approach, proposed originally by Meyerhof, assumes that a stress distribution due to eccentric loading can be approximated by a uniform stress distribution over a reduced area at the base of the wall. This area is defined by a width equal to the wall width minus twice the eccentricity as shown in Figures 4-7 and 4-8. The effect of eccentricity and load inclination is addressed with use of the effective width, $L - 2e_B$, in lieu of the full width, L .

For wall with sloping backfill the factored bearing stress is:

$$q_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{EV-MAX} V_2 + \gamma_{EH-MAX} F_T \sin\beta}{L - 2e_B} \quad (4-21)$$

Note that $(L - 2e_B)$ is set equal to L when the value of eccentricity is negative. A negative value of eccentricity may be found for some extreme geometries, e.g. a wall section with very long reinforcements and a steep, infinite backslope. **Note that when checking the various load factors and load combinations the value of eccentricity, e_B , will vary and a critical value must be determined by comparisons of applicable load combinations.**

Where applicable, in the computation of bearing stress, σ_{V-F} , include the influence of factored surcharge and factored concentrated loads. Maintain consistency with loads and load factors used in the eccentricity calculation and corresponding bearing stress calculation.

- 3) Determine the nominal bearing resistance, q_n , Equation 10.6.3.1.2a-1 of AASHTO (2007). For a level grade in front of a MSE wall and no groundwater influence:

$$q_n = c_f N_c + 0.5L'\gamma_f N_\gamma \quad (4-22)$$

where: c_f = the cohesion of the foundation soil

γ_f = the unit weight of the foundation soil

N_c and N_γ = dimensionless bearing capacity coefficients

L' = effective foundation width, equal to $L - 2e_B$; set L' equal to L if e_B is a negative value

The dimensionless bearing capacity factors can be obtained from Table 10.6.3.1.2a-1 of AASHTO (2007) and, for convenience, are shown in Table 4-6. Modifications to q_n (Equation 4-22) for a ground surface slope and for high groundwater level are provided in 10.6.3.1.2 AASHTO (2007). The beneficial effect of wall embedment is neglected. (Note: for excessive embedment (i.e., embedment greater than the minimum requirements, see Table 2-2), partial embedment may be considered in the determination of q_n provided that the fill in front of the wall is placed and compacted as the reinforced fill is placed and all possible failure modes are examined. Bearing capacity is addressed in detail in the following two NHI courses: 132037 Shallow Foundations, and reference manual FHWA NHI-01-023 (Munfakh et al., 2001); and 132012 Soils & Foundations, and reference manual Volume I, FHWA NHI-06-089 (Samtani and Nowatzki, 2006).

Table 4-6. Bearing Resistance Factors
(Table 10.6.3.1.2a-1, AASHTO {2007}).

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.0	0.0	23	18.1	8.7	8.2
1	5.4	1.1	0.1	24	19.3	9.6	9.4
2	5.6	1.2	0.2	25	20.7	10.7	10.9
3	5.9	1.3	0.2	26	22.3	11.9	12.5
4	6.2	1.4	0.3	27	23.9	13.2	14.5
5	6.5	1.6	0.5	28	25.8	14.7	16.7
6	6.8	1.7	0.6	29	27.9	16.4	19.3
7	7.2	1.9	0.7	30	30.1	18.4	22.4
8	7.5	2.1	0.9	31	32.7	20.6	25.9
9	7.9	2.3	1.0	32	35.5	23.2	30.2
10	8.4	2.5	1.2	33	38.6	26.1	35.2
11	8.8	2.7	1.4	34	42.2	29.4	41.1
12	9.3	3.0	1.7	35	46.1	33.3	48.0
13	9.8	3.3	2.0	36	50.6	37.8	56.3
14	10.4	3.6	2.3	37	55.6	42.9	66.2
15	11.0	3.9	2.7	38	61.4	48.9	78.0
16	11.6	4.3	3.1	39	37.9	56.0	92.3
17	12.3	4.8	3.5	40	75.3	64.2	109.4
18	13.1	5.3	4.1	41	83.9	73.9	130.2
19	13.9	5.8	4.7	42	93.7	85.4	155.6
20	14.8	6.4	5.4	43	105.1	99.0	186.5
21	15.8	7.1	6.2	44	118.4	115.3	224.6
22	16.9	7.8	7.1	45	133.9	134.9	271.8
Note: N_c (Prandtl, 1921), N_q (Reisner, 1924), and N_γ (Vesic, 1975). N_q is embedment term, which is typically not used in MSE wall design.							

- 4) Check that factored bearing resistance is greater than the factored bearing stress, i.e., $q_R \geq q_{V-F}$. The factored bearing resistance (q_R) is given as:

$$q_R = \phi q_n \quad (4-23)$$

where:

ϕ = resistance factor, for MSE walls this factor is 0.65 (Table 11.5.6-1, AASHTO {2007})

- 5) As indicated in step 2) and step 3), q_{V-F} can be decreased and q_R increased by lengthening the reinforcements, though only marginally. The nominal bearing resistance often may be increased by additional subsurface investigation and better definition of the foundation soil properties. If adequate support conditions cannot be achieved or lengthening reinforcements significantly increases costs, improvement of the foundation soil may be considered (dynamic compaction, soil replacement, stone columns, precompression, etc.) – see FHWA Ground Improvement Methods reference manuals NHI-06-019 and NHI-06-020 (Elias et al., 2006).

Local Shear, Punching Shear And Lateral Squeeze. Local shear is a transition between general shear and punching shear, which can occur in loose or compressible soils, in weak soils under slow (drained) loading. If local shear or punching shear failure is possible, Section 10.6.3.1.2b of AASHTO (2007) requires the use of reduced shear strength parameters for calculating the nominal bearing resistance. The reduced effective stress cohesion, c^* is set equal to $0.67c'$. The reduced effective stress soil friction angle, ϕ^* is set equal to $\tan^{-1}(0.67 \tan \phi'_f)$.

Lateral squeeze is a special case of local shear that can occur when bearing on a weak cohesive soil layer overlying a firm soil layer. Lateral squeeze failure results in significant horizontal movement of the soil under the structure.

To prevent local shear of structures bearing on weak cohesive soils it is required that:

$$\gamma_r H \leq 3 c_u \quad (4-24)$$

where γ_r is the nominal unit weight of the reinforced fill, H is the height of the wall and c_u is the nominal total stress cohesion of the foundation soil.

If adequate support conditions cannot be achieved, either the soft soils should be removed or ground improvement of the foundation soils is required. Local shear, as well as bearing on

two layered soil systems in undrained and drained loading, are addressed in Section 10.6.3.1.2 of AASHTO (2007). Local shear and lateral squeeze is addressed in detail in NHI course 132012 Soils & Foundations, and reference manual Volume II, FHWA NHI-06-088 (Samtani and Nowatzki, 2006).

4.4.6.d Settlement Estimate

Conventional settlement analyses should be carried out to ensure that immediate, consolidation, and secondary settlement of the wall are less than the performance requirements of the project (see FHWA NHI-06-088 and NHI-06-089, *Soils and Foundations Reference manuals* {Samtani and Nowatzki, 2006}). Settlement is evaluated under bearing pressure computed at a Service I limit state.

Significant estimated post-construction foundation settlements indicate that the planned top of wall elevations need to be adjusted. This can be accomplished by increasing the top of wall elevations during wall design, or by providing height adjustment within the top of wall coping, and/or by delaying the casting of the top row of panels to the end of erection. The required height of the top row, would then be determined with possible further allowance for continuing settlements. Significant differential settlements (greater than 1/100), indicate the need of slip joints, which allow for independent vertical movement of adjacent precast panels. Where the anticipated settlements and their duration, cannot be accommodated by these measures, consideration must be given to ground improvement techniques such as wick drains, stone columns, dynamic compaction, the use of lightweight fill or the implementation of two-phased construction in which the first phase facing is typically a wire facing.

4.4.7 Step 7 –EVALUATE INTERNAL STABILITY

Internal failure of a MSE wall can occur in two different ways:

- The tensile forces (and, in the case of rigid reinforcements, the shear forces) in the inclusions become so large that the inclusions elongate excessively or break, leading to large movements and/or possible collapse of the structure. This mode of failure is called failure by elongation or breakage of the reinforcements.
- The tensile forces in the reinforcements become larger than the pullout resistance, leading to large movements and/or possible collapse of the structure. This mode of failure is called failure by pullout.

The process of sizing and designing to preclude internal failure, therefore, consists of determining the maximum developed tension forces, their location along a locus of critical slip surfaces and the resistance provided by the reinforcements both in pullout capacity and tensile strength. Internal stability also includes an evaluation of serviceability requirements

such as tolerable lateral movement of supported structures and control of downdrag stress on reinforcement connections.

4.4.7.a Select Type of Soil Reinforcement

Soil reinforcements are either inextensible (i.e., mostly metallic) or extensible (i.e., mostly polymeric materials), as discussed in Chapter 2. The internal wall design model varies by material type due to their extensibility relative to soil at failure. Therefore, the choice of material type should be made at this step of the design. The variations are: whether life prediction is based on metal corrosion or polymer degradation; critical failure plane geometry assumed for design; and lateral stress used for design. Distinction can be made between the characteristics of inextensible and extensible reinforcements, as follows.

Design Methods, Inextensible (e.g., Metallic) Reinforcements

The current method of limit equilibrium analysis uses a coherent gravity structure approach to determine external stability of the reinforced mass, similar to the analysis for any conventional or traditional gravity structure. For internal stability evaluations, it considers a bi-linear failure surface that divides the reinforced zone in active and resistant zones and requires that an equilibrium state be achieved for successful design.

The lateral earth pressure distribution for **external stability**, is assumed to be based on Coulomb's method with a wall friction angle δ assumed to be zero. For **internal stability** lateral pressure varying from a multiple of K_a to an active earth pressure state, K_a is used for design. Previous research (FHWA RD 89-043) has focused on developing the state of stress for internal stability, as a function of K_a , type of reinforcement used (geotextile, geogrid, metal strip or metal grid), and depth. The results from these and more recent (Allen et al., 2001) efforts have been synthesized in a *simplified method*, which will be used throughout this manual.

Design Methods, Extensible (e.g., Geosynthetic) Reinforcements

For **external stability** calculations, the current method assumes an earth pressure distribution, consistent with the method used for inextensible reinforcements.

For **internal stability** computations using the *simplified method*, the internal coefficient of earth pressure is again a function of the type of reinforcement, where the minimum coefficient (K_a) is used for walls constructed with continuous sheets of geotextiles and geogrids. For internal stability, a Rankine failure surface is considered, because the extensible reinforcements can elongate more than the soil, before failure, and do not significantly modify the shape of the soil failure surface.

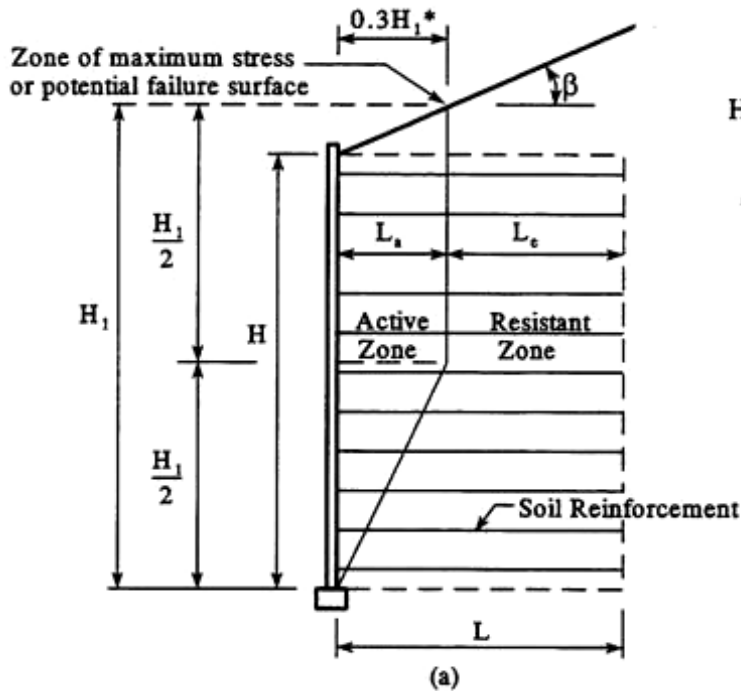
4.4.7.b Define Critical Slip Surface

The critical slip surface in a simple reinforced soil wall is assumed to coincide with the locus of the maximum tensile force, T_{MAX} , in each reinforcement layer. The shape and location of the critical failure surface is based upon instrumented structures and theoretical studies.

This critical failure surface has been assumed to be approximately bilinear in the case of inextensible reinforcements (Figure 4-9), approximately linear in the case of extensible reinforcements (Figure 4-9), and passes through the toe of the wall in both cases.

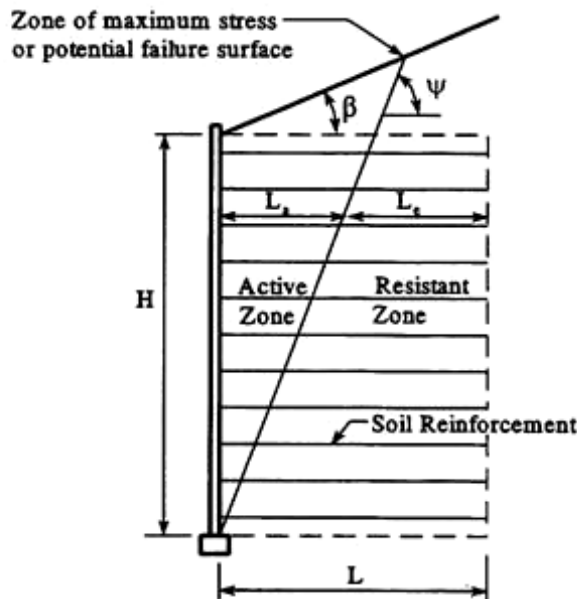
When failure develops, the reinforcement may elongate and be deformed at its intersection with the failure surface. As a result, the tensile force in the reinforcement would increase and rotate. Consequently, the component in the direction of the failure surface would increase and the normal component may increase or decrease. Elongation and rotation of the reinforcements may be negligible for stiff inextensible reinforcements such as steel strips but may be significant with geosynthetics. Any reinforcement rotation is ignored for internal wall stability calculations with the simplified method. However, reinforcement rotation may be considered in compound slope stability analysis (see Chapters 8 and 9).

For extensible reinforcements, the Coulomb earth pressure relationship shown on Figure 4-5 should be used to define the failure surface, per AASHTO Figure 11.10.6.3.1-1 (2007), where the wall front batter from vertical is greater than 10 degrees.



$$H_1 = H + \frac{\tan \beta \times 0.3H}{1 - 0.3 \tan \beta}$$

* If wall face is battered, an offset of $0.3H_1$ is still required, and the upper portion of the zone of maximum stress should be parallel to the wall face



For vertical face

$$\psi = 45 + \frac{\phi}{2}$$

For walls with a face batter angle (θ) 10° or more from the vertical,

$$\tan(\psi - \theta) = \frac{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)][1 + \tan(\delta + 90 - \theta)\cot(\phi + \theta - 90)]}}{1 + \tan(\delta + 90 - \theta)[\tan(\phi - \beta) + \cot(\phi + \theta - 90)]}$$

with $\delta = \beta$
 θ = wall batter angle (b)

For wall with a broken backslope, use $\delta = \beta_{ca}$

Figure 4-9. Location of potential failure surface for internal stability design of MSE Walls (a) inextensible reinforcements and (b) extensible reinforcements.

4.4.7.c Define Unfactored Loads

The primary sources of internal loading of an MSE wall is the earth pressure from the reinforced fill and any surcharge loadings on top of the reinforced zone. The unfactored loads for MSE walls may include loads due to, vertical earth pressure (EV), live load surcharge (LS), and earth surcharge (ES). Water, seismic, and vehicle impact loads should also be evaluated, as appropriate.

Research studies (Collin, 1986; Christopher et al., 1990; Allen et al., 2001) have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE wall, which, in turn, is a function of the modulus, extensibility and density of reinforcement. Based on this research, a relationship between the type of the reinforcement and the overburden stress has been developed, and shown in Figure 4-10. The K_r/K_a ratio for metallic (inextensible) reinforcements decreases from the top of the reinforced wall fill to a constant value 20 ft (6 m) below this elevation. In contrast to inextensible reinforcements, the K_r/K_a for extensible (e.g., geosynthetic) reinforcement is a constant. Note that the resulting K_r/K_a ratio is referenced to the top of the wall at the face, excluding any copings and appurtenances (i.e., the top of the reinforced soil zone at the face) for both walls with level and with sloping backfills. The K_r/K_a starting elevation for an MSE wall supporting a spread footing bridge abutment is the top of the backfill, see Chapter 6 and appended design example.

The simplified approach used herein was developed in order to avoid iterative design procedures required by some of the complex refinements of the available methods i.e., the coherent gravity method (AASHTO, 1994 Interims) and the structural stiffness method (FHWA RD 89-043, Christopher et al., 1990). The *simplified method* (a.k.a. *simplified coherent gravity method*) (Elias and Christopher, 1997; Allen et al., 2001) is based on the same empirical data used to develop these two methods.

Figure 4-10 was prepared by back analysis of the lateral stress ratio K_r from available field data where stresses in the reinforcements were measured and normalized as a function of the Rankine active earth pressure coefficient, K_a . The Rankine active earth pressure theory assumes lateral pressure is independent of backfill slope and interface friction. The ratios shown on Figure 4-10 correspond to values representative of the specific reinforcement systems that are known to give satisfactory results assuming that the vertical stress is equal to the weight of the overburden (γH). This provides a simplified evaluation method for all cohesionless reinforced fill walls. Future data may lead to modifications in Figure 4-10, including relationships for newly developed reinforcement types, effect of full height panels, etc. These relationships can be developed by instrumenting structures and using numerical models to verify the K_r/K_a ratio for routine and complex walls.

The lateral earth pressure coefficient K_r is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship, assuming no wall friction and a β angle equal to zero (i.e., equivalent to the Rankine earth pressure coefficient). For a vertical wall the earth pressure therefore reduces to the Rankine equation:

$$K_a = \tan^2 \left(45 - \frac{\phi'_r}{2} \right) \quad (4-25)$$

For wall face batters equal to or greater than 10° from the vertical, the following simplified form of the Coulomb equation can be used:

$$K_a = \frac{\sin^2(\theta + \phi'_r)}{\sin^3 \theta \left[1 + \frac{\sin \phi'_r}{\sin \theta} \right]} \quad (4-26)$$

where θ is the inclination of the back of the facing as measured from the horizontal starting in front of the wall, as shown in Figure 4-5. Commentary C11.10.6.2.1 AASHTO (2007) states that above equation can be used for battered walls. The 10° value recommendation is consistent with the equation to determine the failure surface location for walls with 10° or greater batter (C11.10.6.3.1, AASHTO {2007}).

The stress, σ_2 , due to a sloping backfill on top of an MSE wall can be determined as shown in Figure 4-11. An equivalent soil height, S , is computed based upon the slope geometry. The value of S_{eq} should not exceed the slope height for broken back sloping fills. A reinforcement length of $0.7H$ is used to compute the sloping backfill stress, σ_2 , on the soil reinforcement, as a greater length would only have minimal effect on the reinforcement. The vertical stress is equal to the product equivalent soil height and the reinforced fill unit weight, and is uniformly applied across the top of the MSE zone.

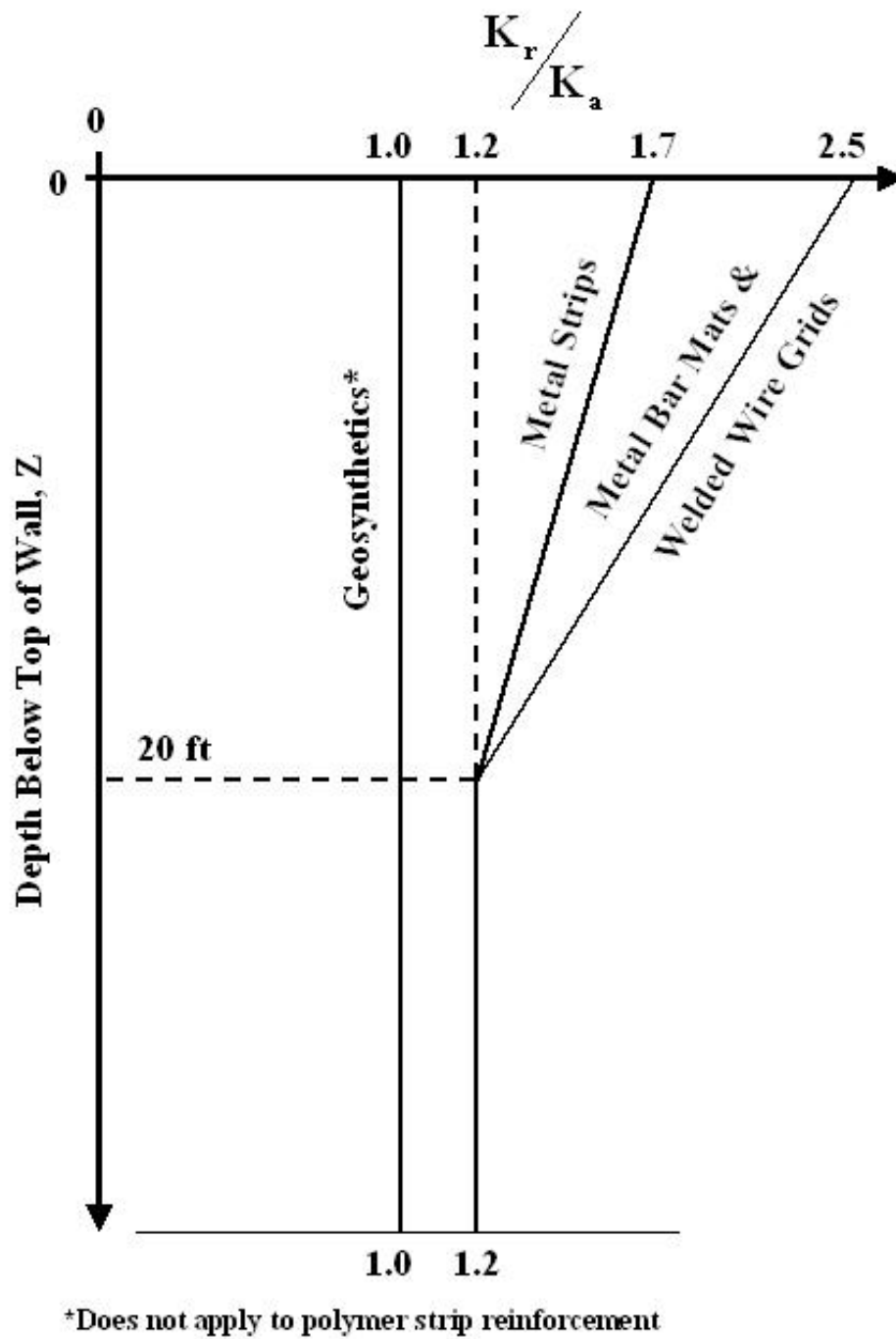


Figure 4-10. Variation of the coefficient of lateral stress ratio (K_r/K_a) with depth in a MSE wall (Elias and Christopher, 1997; AASHTO, 2002; & after AASHTO, 2007).

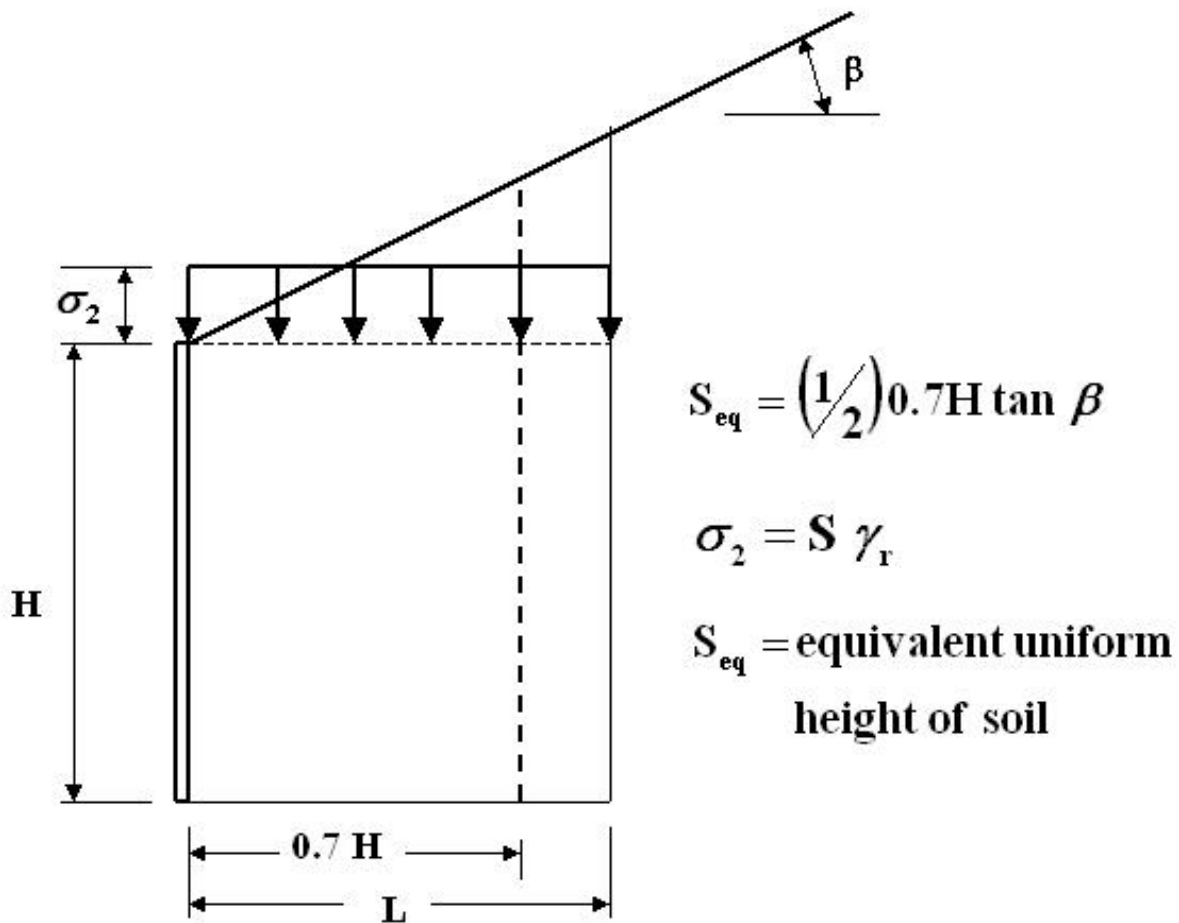


Figure 4-11. Calculation of vertical stress for sloping backfill conditions for internal stability.

4.4.7.d Establish Vertical Layout of Soil Reinforcements

Use of a constant reinforcement section and spacing for the full height of the wall usually gives more reinforcement in the upper portion of the wall than is required for stability. Therefore, a more economical design may be possible by varying the reinforcement density with depth. However, to provide a coherent reinforced soil zone, vertical spacing of reinforcement should not exceed 32 in. (800 mm).

There are generally two practical ways to accomplish this for MSE walls:

- For reinforcements consisting of strips, grids, or mats used with segmental precast concrete facings, the vertical spacing is maintained constant and the reinforcement

density is increased with depth by increasing the number and/or the size of the reinforcements. For instance, the typical horizontal spacing of 2-in. (50 mm) x 5/32-in. (4 mm) strips is 30 in. (0.75 m), but this can be decreased by adding horizontal reinforcement locations.

- For continuous sheet reinforcements, made of geotextiles or geogrids, a common way of varying the reinforcement density T_{al}/S_v is to change the vertical spacing S_v , especially if wrapped facing is used, because it easily accommodates spacing variations. The range of acceptable spacing is governed by consideration of placement and compaction of the backfill (e.g., S_v taken as 1, 2 or 3 times the compacted lift thickness). The reinforcement density T_{al}/S_v can also be varied by changing the strength (T_{al}) especially if wrapped facing techniques requiring a constant wrap height are used.

Low-to medium-height walls (e.g., < 16 ft {5 m}) are usually constructed with one strength geosynthetic. Taller walls use multiple strength geosynthetics. For example the 41 ft (12.6 m) high Seattle preload wall used four strengths of geotextiles (Allen et al., 1992). A maximum spacing of 16 in. (400 mm) is typical for wrapped faced geosynthetic walls, although a smaller spacing may be desirable to minimize bulging.

For walls constructed with modular blocks, the maximum vertical spacing of reinforcement should be limited to two times the block depth (front face to back face) or 32 in. (810 mm), which ever is less, to assure construction and long-term stability. The top row of reinforcement should be limited to 1.5 the block depth (e.g. one unit plus a cap unit). (AASHTO 11.10.2.3.1 {AASHTO, 2007}).

For large face units, such as 3 ft by 3 ft (0.9 m by 0.9 m) gabions, a vertical spacing equal to the face height (i.e., 3 ft {0.9 m}) is typically used. This spacing slightly exceeds the limit noted above, but this may be offset by the contributions of the large facing unit to internal (i.e., bulging) stability.

4.4.7.e Calculate Factored Tensile Forces in the Reinforcement Layers

e.1 Calculate Horizontal Stress

For internal stability analysis, the distribution of horizontal stress, σ_H , is first established. The horizontal stress at any given depth within the reinforced soil zone is expressed as follows:

$$\sigma_H = K_r [\sigma_v] + \Delta\sigma_H \quad (4-27)$$

where K_r is the coefficient of lateral earth pressure in the reinforced soil zone and is obtained from Figure 4-10, σ_v is the factored vertical pressure at the depth of interest, and $\Delta\sigma_h$ is the supplemental factored horizontal stress due to external surcharges.

For internal stability analysis, the following assumptions are made in the computation of factored vertical pressure, σ_v :

1. Vertical pressure due to the weight of the reinforced soil zone is assigned a load type “EV” with a corresponding (maximum) load factor, $\gamma_{P-EV} = 1.35$. The maximum load factor of 1.35, and not the minimum load factor of 1.00, is always used to find the critical stress.
2. Any vertical surcharge above the reinforced soil zone that is due to soil or considered as an equivalent soil surcharge is assigned a load type “EV.” In this scenario, a live load traffic surcharge that is represented by an equivalent uniform soil surcharge of height h_{eq} is assumed as load type “EV.” This is in contrast to the external stability analysis where the live load traffic surcharge is assumed as load type “LS” because in external stability analysis the MSE wall is assumed to be a rigid block. For internal stability analysis, the assumption of load type “EV” is used so that the amount of soil reinforcement within the reinforced soil zone is approximately the same as obtained using past working stress design approach (i.e., calibration by fitting).
3. The unit weight of the equivalent soil surcharge is assumed to be the same as the unit weight of the reinforced soil zone, γ_r , which is generally greater than or equal to the unit weight of the retained backfill.
4. Any vertical surcharge that is due to non-soil source is assigned a load type “ES.” Example of such a load is the bearing pressure under a spread footing on top of reinforced soil zone. However, the application of the load factor of $\gamma_{P-ES} = 1.50$ that is assigned to load type “ES” is a function of how the vertical pressures are computed as follows:
 - If the vertical pressures are based on nominal (i.e., unfactored) loads, then use $\gamma_{P-ES} = 1.50$.
 - If the vertical pressures were based on factored loads, then use $\gamma_{P-ES} = 1.00$. This is because once the loads are factored they should not be factored again.

It is recommended that the factored vertical pressure be evaluated using both the above approaches and the larger value chosen for analysis.

The supplemental factored horizontal pressure, $\Delta\sigma_h$, could be from a variety of sources. Two examples of supplemental horizontal pressures are as follows:

1. Horizontal pressures due to the horizontal (shear) stresses at the bottom of a spread footing on top of reinforced soil zone.
2. Horizontal pressures from deep foundation elements extending through the reinforced soil zone.

Supplemental horizontal pressures are assigned a load type “ES” since they represent surcharges on or within the reinforced soil zone. However, similar to the vertical pressures due to non-soil loads, the application of the maximum load factor of $\gamma_{P-ES} = 1.50$ that is assigned to load type “ES” is a function of how the horizontal pressures are computed as follows:

- If the horizontal pressures are based on nominal (i.e., unfactored) loads, then use $\gamma_{ES-MAX} = 1.50$.
- If the horizontal pressures were based on factored loads, then use $\gamma_{P-ES} = 1.00$. This is because once the loads are factored they should not be factored again.

As with vertical pressure, it is recommended that the factored horizontal pressure be evaluated using both the above approaches and the larger value chosen for analysis.

The application of the above guidance is illustrated below for four MSE wall configurations ranging from simple to complex geometries. The logic used in development of these equations can be extended to any other MSE wall configuration with complex system of surcharges.

Example 1: MSE wall with level backfill and no surcharge. This represents the simplest MSE wall configuration for which the horizontal stress at any given depth Z below the top of the reinforced soil zone is given as follows:

$$\sigma_H = K_r [(\gamma_r Z) \gamma_{EV-MAX}] \quad (4-28)$$

where, γ_r is the unit weight of soil in the reinforced soil zone, and γ_{EV-MAX} is the maximum load factor (=1.35) for load type “EV.” The value of K_r is obtained by assuming that: (i) the variation of K_r/K_a ratio shown in Figure 4-10 starts from the top of the reinforced soil zone, and (ii) K_a is computed using the Rankine formula (Eq. 4-25).

Example 2: MSE wall with sloping backfill. This configuration is commonly used for side-hill retaining wall applications. Example of this configuration is shown in Figure 4-10. As shown in Figure 4-11, the sloping surcharge is approximated by an equivalent uniform soil surcharge of height, S_{eq} . For this case, the horizontal stress at any depth Z below the top of the reinforced soil zone can be written as follows:

$$\sigma_H = K_r [\gamma_r (Z + S_{eq}) \gamma_{EV-MAX}] \quad (4-29)$$

The value of K_r is obtained by assuming that: (i) the variation of K_r/K_a ratio shown in Figure 4-10 starts from the top of the reinforced soil zone, and (ii) K_a is computed using the Rankine formula (Eq. 4-25) assuming that the backfill is level. Use of Equation 4-29 is demonstrated in Example Problem E-3 in Appendix E.

Example 3: MSE wall with level backfill and live load surcharge. This configuration is commonly used for grade-separated roadways. Assuming that the live load is expressed as an equivalent uniform soil surcharge of height, h_{eq} , (equal to 2 ft) the horizontal stress at any depth Z below the top of the reinforced soil zone can be written as follows:

$$\sigma_H = K_r [\gamma_r (Z + h_{eq}) \gamma_{EV-MAX}] \quad (4-30)$$

The value of K_r is obtained by assuming that: (i) the variation of K_r/K_a ratio shown in Figure 4-10 starts from the top of the reinforced soil zone, and (ii) K_a is computed using the Rankine formula (Eq. 4-25). Use of Equation 4-30 is demonstrated in Example Problem E-4 in Appendix E.

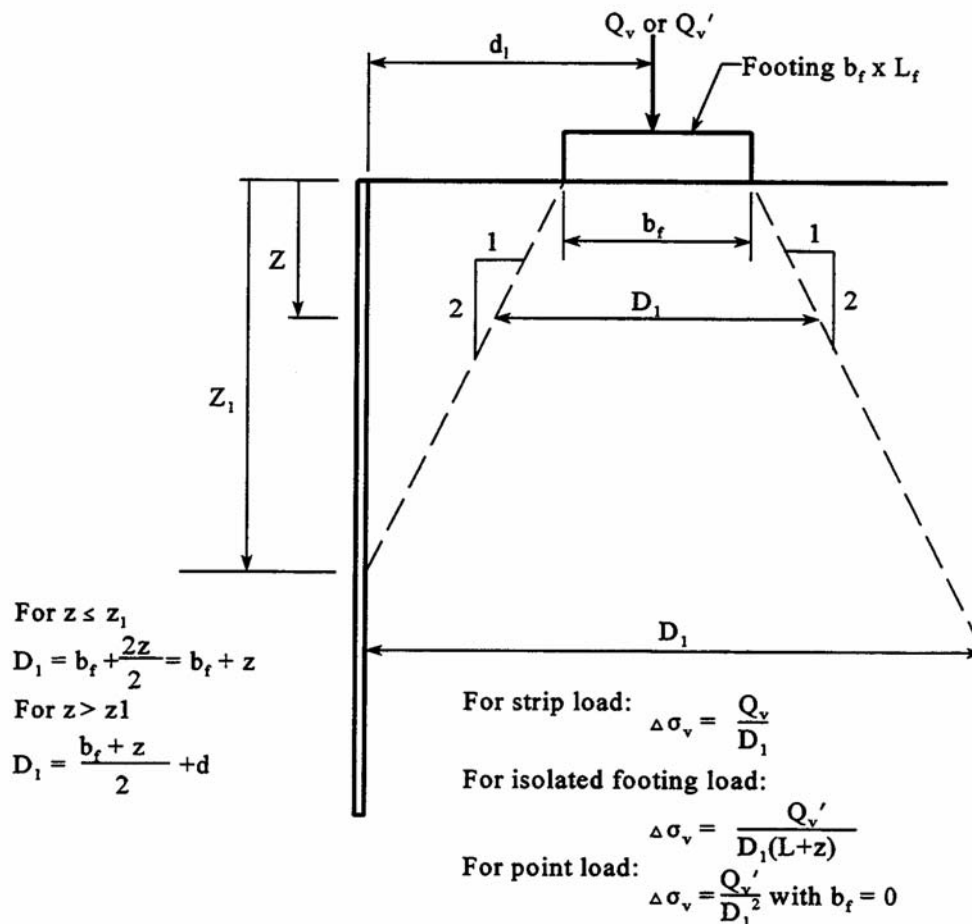
Example 4: Bridge abutment with a spread footing on top of MSE wall. In this configuration the bridge superstructure rests on a spread footing on top of a MSE wall. This configuration is discussed in detail in Chapter 6. It is included here as an example of a complex system of surcharges that can be used to explain the computation of horizontal stress for such cases. For development of the equation of horizontal stress, refer to Figures 4-12 and 4-13. Assumptions are that the live load is expressed as an equivalent uniform soil surcharge of height, h_{eq} , as per Table 4-4, the height of the roadway fill above the reinforced soil zone is h , and $\Delta\sigma_v$ and $\Delta\sigma_H$ increase T_{MAX} . Then,

the horizontal stress at any depth Z below the top of the reinforced soil zone can be written as follows:

$$\sigma_H = K_r [\gamma_r (Z + h + h_{eq}) \gamma_{EV-MAX} + (\Delta\sigma_{v-footing}) \gamma_{P-ES}] + (\Delta\sigma_H) \gamma_{P-ES} \quad (4-31)$$

where $\Delta\sigma_v$ and $\Delta\sigma_H$ are the vertical (normal) and horizontal (shear) pressures at the bottom of the spread footing. As noted earlier, the value of γ_{P-ES} is 1.50 if nominal (i.e., unfactored) pressures are used, and is 1.00 if factored pressures are used with the final value being chosen based on larger values of $(\Delta\sigma_{v-footing})\gamma_{P-ES}$ and $(\Delta\sigma_H)\gamma_{P-ES}$.

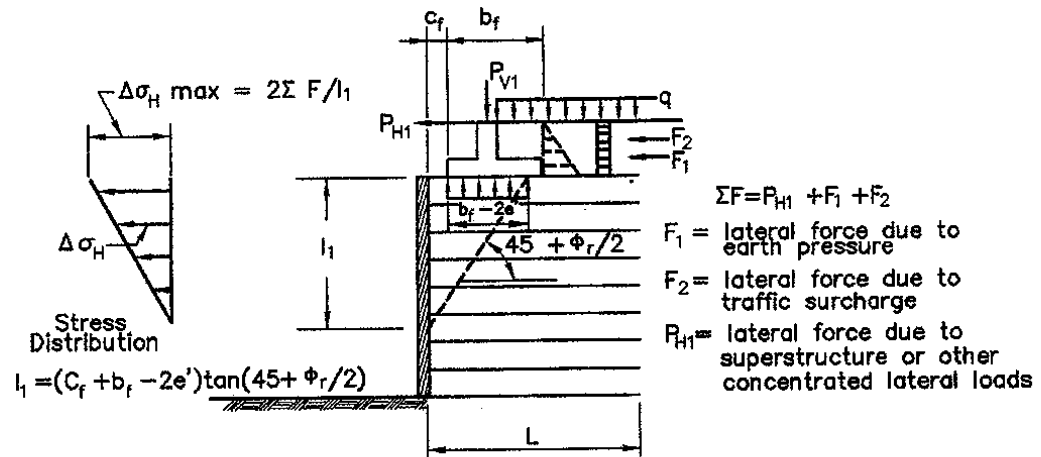
The value of K_r is obtained by assuming that: (i) the variation of K_r/K_a ratio shown in Figure 4-10 starts from the finished pavement grade behind the spread footing, and (ii) K_a is computed using the Rankine formula (Eq. 4-25). Use of Equation 4-31 is demonstrated in Example Problem E-5 in Appendix E.



Where: D_1 = Effective width of applied load at any depth, calculated as shown above
 b_f = Width of applied load. For footings which are eccentrically loaded (e.g., bridge abutment footings), set b_f equal to the equivalent footing width B' by reducing it by $2e'$, where e' is the eccentricity of the footing load (i.e., $b_f - 2e'$).
 L_f = Length of footing
 Q_v = Load per linear feet of strip footing
 Q_v' = Load on isolated rectangular footing or point load
 z_1 = Depth where effective width intersects back of wall face = $2d_1 - b_f$

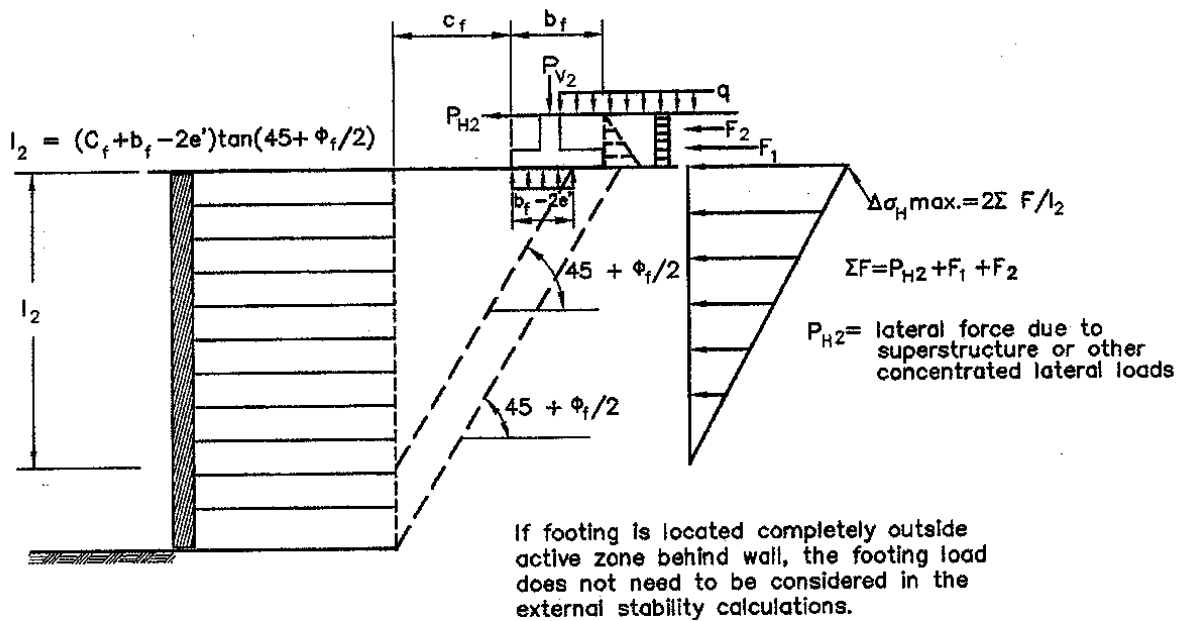
Assume the increased vertical stress due to the surcharge load has no influence on stresses used to evaluate internal stability if the surcharge load is located behind the reinforced soil mass. For external stability, assume the surcharge has no influence if it is located outside the active zone behind the wall.

Figure 4-12. Distribution of stress from concentrated vertical load for internal and external stability calculations.



e' = eccentricity of load on footing

a. Distribution of Stress for Internal Stability Calculations.



b. Distribution of Stress for External Stability Calculations.

Figure 4-13. Distribution of stresses from concentrated horizontal loads.

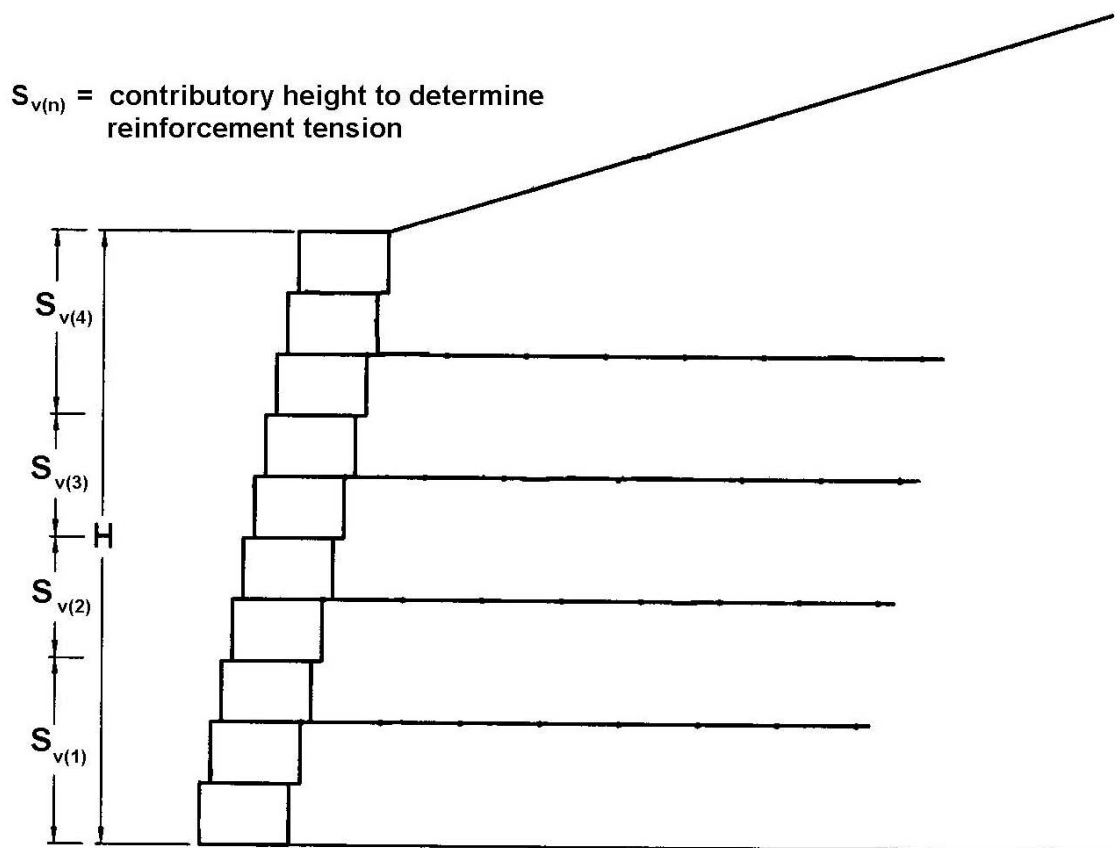


Figure 4-14. Reinforcement load contributory height.

e.2 Calculate Maximum Tension, T_{MAX}

Calculate the maximum factored tension T_{MAX} in each reinforcement layer per unit width of wall based on the vertical spacing S_v from:

$$T_{MAX} = \sigma_H S_v \quad (\text{in force per unit reinforcement width \{kips/ft\}}) \quad (4-32a)$$

The term S_v is equal to the vertical reinforcement spacing for a layer where vertically adjacent reinforcements are equally spaced from the layer under consideration. In this case, σ_H , calculated at the level of the reinforcement, is at the center of the contributory height. The contributory height is defined as the midpoint between vertically adjacent reinforcement elevations, except for the top and bottom layers reinforcement.

For the top and bottom layers of reinforcement, S_v is the distance from top or bottom of wall, respectively, to the midpoint between the first and second layer (from top or bottom of wall, respectively) of reinforcement. S_v distances are illustrated in Figure 4-14.

The maximum reinforcement tension, T_{MAX} , for the top and bottom layers of reinforcement, and for intermediate layers that do not have equally spaced adjacent layers, is calculated as the product of the contributory height and the average factored horizontal stress acting upon that contributory height. The average stress can be calculated based upon the tributary trapezoidal area (i.e. average of the stress at top and at the bottom of the contributory height) or at the midpoint of the contributory height, as illustrated in Figure 4-14.

Alternatively, for discrete reinforcements (metal strips, bar mats, geogrids, etc.) T_{MAX} (force per unit width) may be calculated at each level as $P_{TMAX-UWR}$ in terms of force per unit width of reinforcement, as:

$$P_{TMAX-UWR} = \frac{\sigma_H S_v}{R_c} \quad (4-32b)$$

where:

R_c = ratio of gross width of strip, sheet, or grid to the center-to-center horizontal spacing between the strips, sheets, or grids (see Eq. 3-9 and Figure 3-3); e.g., $R_c = 1$ for full coverage reinforcement.

For discrete reinforcements of known spacing and segmental precast concrete facing of known panel dimensions, T_{MAX} (force per unit width) can alternatively be calculated per discrete reinforcement, P_{TMAX-D} , per panel width, defined as.

$$P_{TMAX-D} = \frac{\sigma_H S_v W_p}{N_p} \quad (4-32c)$$

where:

P_{TMAX-D} = maximum factored load in discrete reinforcement element

W_p = width of panel

N_p = number of discrete reinforcements per panel width (e.g., 2, 3, etc.)

4.4.7.f Calculate Soil Reinforcement Resistance

The procedure and discussion on definition of nominal long-term reinforcement design strength (T_{al}), for both steel and geosynthetic reinforcements, are presented in Section 3.5 of this manual. The factored soil resistance is the product of the nominal long-term strength, coverage ratio, and applicable resistance factor, ϕ . The resistance factors for tensile rupture

of MSE wall soil reinforcements are summarized in Table 4-7. The factored tensile resistance, T_r , is equal to:

$$T_r = \phi T_{al} \quad (4-33)$$

T_{al} (as noted in Section 3.5) and T_r may be expressed in terms of strength per unit width of wall, per reinforcement element, or per unit reinforcement width.

Table 4-7. Resistance Factors, ϕ , for Tensile and Pullout Resistance for MSE Walls
(after Table 11.5.6-1, AASHTO {2007}).

Reinforcement Type and Loading Condition			Resistance Factor
Metallic reinforcement and connectors	Strip reinforcements ^(A)		
		Static loading	0.75
		Combined static/earthquake loading	1.00
		Combined static/traffic barrier impact ^(B)	1.00
	Grid reinforcements ^(A, C)		
		Static loading	0.65
		Combined static/earthquake loading	0.85
		Combined static/traffic barrier impact ^(B)	0.85
Geosynthetic reinforcement and connectors		Static loading	0.90
		Combined static/earthquake loading	1.20
		Combined static/traffic barrier impact ^(B)	1.20
Pullout resistance of tensile reinforcement (metallic and geosynthetic)		Static loading	0.90
		Combined static/earthquake loading	1.20
		Combined static/traffic barrier impact ^(B)	1.00
Notes: A. Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with AASHTO (2007) Article 6.8.3 and apply to net section less sacrificial area. B. Combined static/traffic barrier impact resistance factors are not presented in AASHTO. C. Applies to grid reinforcements connected to rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.			

4.4.7.g Select Grade of and/or Number of Soil Reinforcement Elements at Each Level

The soil reinforcement vertical layout, the factored tensile force at each reinforcement level, and the factored soil reinforcement resistance were defined in the previous three steps. With this information, select suitable grades (strength) of reinforcement, or number of discrete (e.g., strip) reinforcements, for the defined vertical reinforcement layout. Then with this layout check pullout and, as applicable, extreme event loadings. Adjust layout if/as necessary.

Stability with respect to breakage of the reinforcements requires that:

$$T_{MAX} \leq T_r \quad (4-34)$$

Where T_{MAX} is the maximum factored load in a reinforcement (Eqs. 4-32) and T_r is the factored reinforcement tensile resistance (Eq. 4-33).

4.4.7.h Internal Stability with Respect to Pullout Failure

Stability with respect to pullout of the reinforcements requires that the factored effective pullout length is greater than or equal to the factored tensile load in the reinforcement, T_{MAX} . Each layer of reinforcement should be checked, as pullout resistance and/or tensile loads may vary with reinforcement layer. Therefore, the following criteria should be satisfied:

$$\phi L_e \geq \frac{T_{MAX}}{F^* \alpha \sigma_v C R_c} \quad (4-35)$$

where:

- L_e = The length of embedment in the resisting zone. Note that the boundary between the resisting and active zones may be modified by concentrated loadings.
- T_{MAX} = Maximum reinforcement tension
- ϕ = Resistance factor for soil reinforcement pullout. See Table 4-7.
- F^* = Pullout resistance factor (see Chapter 3) with variation in depth starting at the same elevation as that for K_r/K_a variation.
- α = Scale correction factor (see Chapter 3)
- σ_v = Nominal (i.e., unfactored) vertical stress at the reinforcement level in the resistant zone, including distributed dead load surcharges, neglecting traffic loads. See Figure 4-15 for computing σ_v for sloping backfills.
- C = 2 for strip, grid, and sheet type reinforcement
- R_c = Coverage ratio

Therefore, the required embedment length in the resistance zone (i.e., beyond the potential failure surface) can be determined from:

$$L_e \geq \frac{T_{MAX}}{\phi F^* \alpha \sigma_v C R_e} \geq 3 \text{ ft (1 m)} \quad (4-36)$$

If a traffic or other live load is present, it is recommended that T_{MAX} be computed with the live loads and that the pullout resistance be computed excluding the live loads. This addresses the possibility of the live loads being present near the front of the wall but not above the reinforcement embedment length. The pullout resistance and the T_{MAX} can be calculated with the live load excluded (AASHTO {2009 Interims} specifications) if it can be shown that the live load will be on the active and resistant zones at the same time or on the resistant zone alone. An agency should note their pullout calculation requirement, if it varies from AASHTO, in their specifications.

Commentary C11.10.6.2.1 (AASHTO, 2009 Interims) notes that traffic loads and other live loads are not included for pullout calculations. Therefore, if T_{MAX} calculation for checking the reinforcement and connection strengths included a live load surcharge the value must be recomputed, without the surcharge load, for Equation 4-35 or 4-36.

If the criterion is not satisfied for all reinforcement layers, the reinforcement length has to be increased and/or reinforcement with a greater pullout resistance per unit width must be used, or the reinforcement vertical spacing may be reduced which would reduce T_{MAX} .

The total length of reinforcement, L , required for internal stability is then determined from:

$$L = L_a + L_e \quad (4-37)$$

where L_a is obtained from Figure 4-9 for simple structures not supporting concentrated external loads such as bridge abutments. Based on this figure the following relationships can be obtained for L_a :

For MSE walls with **extensible** reinforcement, vertical face and horizontal backfill:

$$L_a = (H - Z) \tan (45 - \phi'/2) \quad (4-38)$$

where Z is the depth to the reinforcement level.

For walls with **inextensible** reinforcement, vertical face and horizontal backfill, from the base up to $H/2$:

$$L_a = 0.6 (H-Z) \quad (4-39)$$

For the upper half of a wall with inextensible reinforcements, vertical face, and horizontal backfill:

$$L_a = 0.3H \quad (4-40)$$

For construction ease, a final uniform length is commonly chosen, based on the maximum length required. However, if internal stability controls the length, it could be varied from the base, increasing with the height of the wall to the maximum length requirement based on a combination of internal and maximum external stability requirements. See Chapter 6, section 6.3 for additional guidance.

4.4.7.i Check Connection Strength

The connection of the reinforcements with the facing, should be designed for T_{MAX} for all limit states. The resistance factors (ϕ) for the connectors are the same as for the reinforcement strength, and are listed in Table 4-7 (Article 11.10.6.2.2 (AASHTO, 2007)).

Connections to Concrete Panels

The metallic reinforcements for MSE systems constructed with segmental precast panels are structurally connected to the facing by either bolting the reinforcement to a tie strip cast in the panel or connected with a bar connector to suitable anchorage devices in the panels. The capacity of the embedded connector as an anchorage must be checked by tests as required by Article 5.11.3 AASHTO (2007) for each geometry used. Connections between metallic reinforcements and facing units should be designed in accordance with AASHTO Article 6.13.3, and consider corrosion losses in accordance with AASHTO Article 11.10.6.4.2a. The design load at the connection is equal to the maximum load on the reinforcement.

Polyethylene geogrid reinforcements may be structurally connected to segmental precast panels by casting a tab of the geogrid into the panel and connecting to the full length of geogrid with a *bodkin* joint, as illustrated in Figure 4-16. The capacity of the embedded connector as an anchorage must be checked by tests as required by Article 5.11.3 AASHTO (2007) for each geometry used. A slat of polyethylene is used for the bodkin. Care should be exercised during construction to eliminate slack from this connection.

Polyester geogrids and geotextiles should not be cast into concrete for connections, due to potential chemical degradation. Other types of geotextiles also are not cast into concrete for connections due to fabrication and field connection requirements.

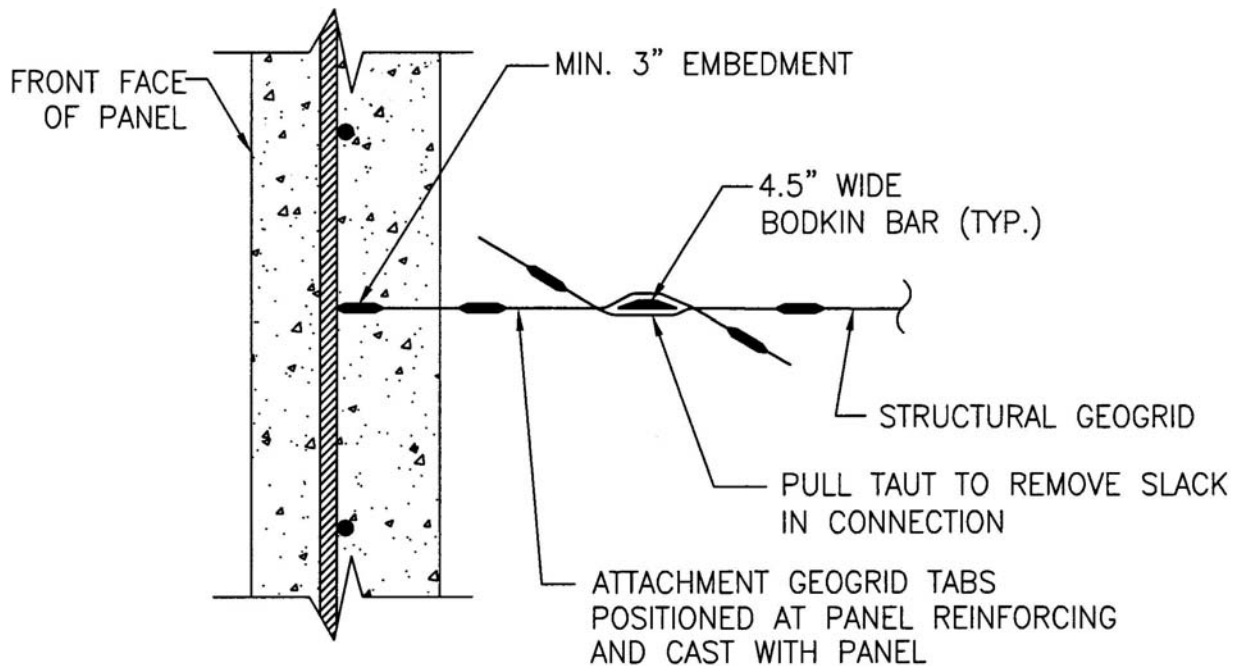


Figure 4-16. Bodkin connection detail (looking at cross section of segmental panel face).

Connections to MBW Units

MSE walls constructed with MBW units are connected either by (i) a structural connection subject to verification under AASHTO Article 5.11.3, (ii) friction between the units and the reinforcement, including the friction developed from the aggregate contained within the core of the units, or, (iii) a combination of friction and shear from connection devices. This strength will vary with each unit depending on its geometry, unit batter, normal pressure, depth of unit, and unit infill gravel (if applicable). The connection strength is therefore specific to each unit/reinforcement combination and must be developed uniquely by test for each combination.

The nominal long-term connection strength, T_{alc} developed by frictional and/or structural means is determined as follows:

$$T_{alc} = \frac{T_{ult} \times CR_{cr}}{RF_D} \quad (4-41)$$

where:

- T_{alc} = nominal long-term reinforcement/facing connection strength per unit reinforcement width at a specified confining pressure
- T_{ult} = ultimate tensile strength of the geosynthetic soil reinforcement, defined as the minimum average roll value (MARV)
- RF_D = reduction factor to account for chemical and biological degradation
- CR_{cr} = long-term connection strength reduction factor to account for reduced ultimate strength resulting from the connection

CR_{cr} may be obtained from long-term or short-term tests, as described below.

CR_{cr} Defined with Long-Term Testing

A series of connection creep tests are performed over extended periods of time to evaluate creep rupture at the connection. The long-term connection creep rupture data is extrapolated to the specified design life (e.g., 75 years, 100 years) to define the creep reduced connection strength, T_{cre} , at the specified design life. Details for long-term testing and interpretation of results are presented in Appendix B. With this long-term testing, CR_{cr} is defined as follows:

$$CR_{cr} = \frac{T_{cre}}{T_{lot}} \quad (4-42)$$

T_{lot} is the ultimate wide width tensile strength of the reinforcement material roll/lot used for the connection strength testing. The T_{lot} strength, for example, might be 103% to 115% of the minimum average roll value (MARV) ultimate strength, T_{ult} (or noted $T_{ult-MARV}$).

CR_{cr} Defined with Short-Term Testing

Short-term (i.e., quick) ultimate strength tests, per ASTM D6638, are used to define an ultimate connection strength, $T_{ultconn}$, at a specified confining pressure. Tests should be performed in accordance with ASTM D6638, *Determining Connection*

Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks). With short-term testing, CR_{cr} , is defined as follows:

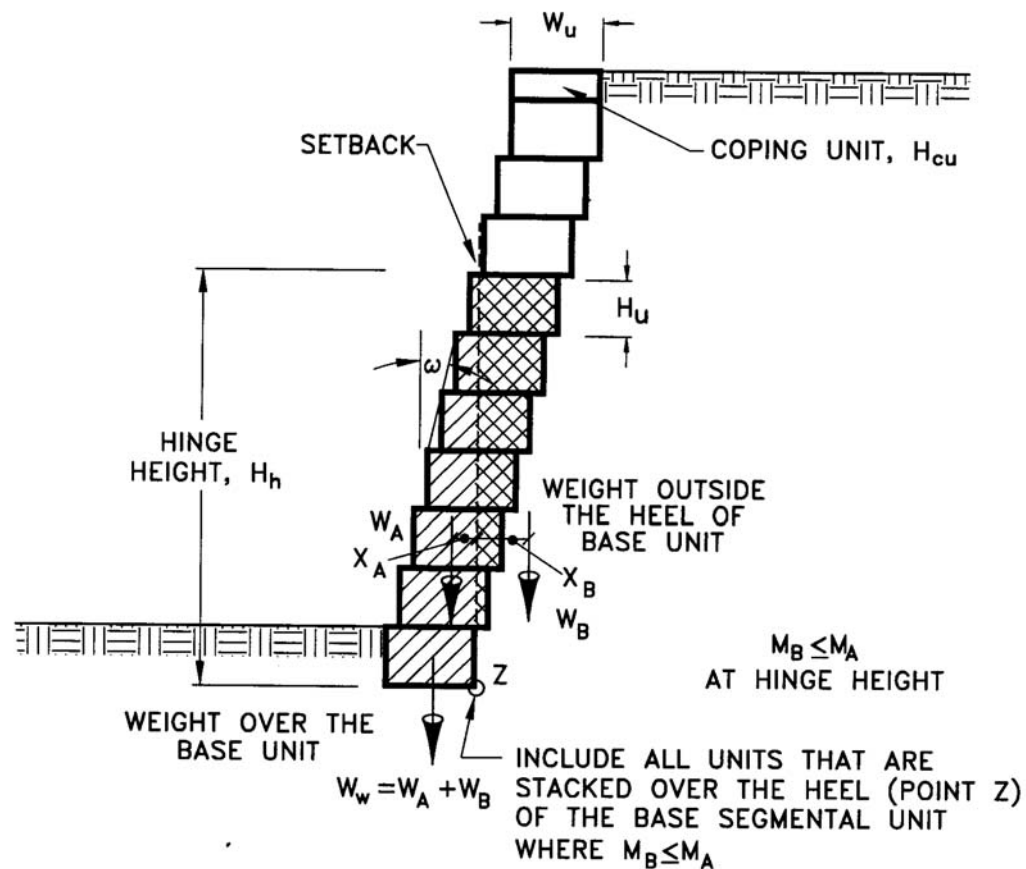
$$CR_{cr} = \frac{T_{ultconn}}{RF_{cr} T_{lot}} \quad (4-43)$$

RF_{cr} is the geosynthetic creep reduction factor (see Chapter 3), and T_{lot} is the ultimate wide width tensile strength of the reinforcement material roll/lot used for the connection strength testing.

Raw data from short-term connection strength laboratory testing should not be used for design. The wall designer should evaluate the data and define the nominal long-term connection strength, T_{alc} . Steps for this data reduction are summarized and discussed in Appendix B. An example of reduction of short-term connection strength data is presented in Appendix B.

Note that the environment between and directly behind the modular blocks at the connection may not be the same as the environment within the reinforced soil zone. Therefore, the long-term environmental aging factor (RF_D) may be significantly different than that used in computing the nominal long-term reinforcement strength T_{al} .

The connection strength as developed above is a function of normal pressure, which is developed by the weight of the units. Thus, it will vary from a minimum in the upper portion of the structure to a maximum near the bottom of the structure for walls with no batter. Further, since many MBW walls are constructed with a front batter, the column weight above the base of the wall or above any other interface may not correspond to the weight of the facing units above the reference elevation. The concept is shown in Figure 4-17, and is termed a hinge height (Simac et al., 1993). Hence, for walls with a nominal batter of more than 8 degrees, the normal stress is limited to the lesser of the hinge height or the height of the wall above the interface. This vertical pressure range should be used in developing CR_{cr} . This recommendation is based on research findings that indicated that the hinge height concept is overly conservative for walls with small batters (Bathurst et al., 2000).



HINGE HEIGHT, H_h . THE FULL WEIGHT OF ALL SRW UNITS WITHIN H_h WILL BE CONSIDERED TO ACT AT THE BASE OF THE LOWERMOST SRW UNIT.

$$M_A = W_A * X_A$$

$$M_B = W_B * X_B$$

$$H_h = 2(W_u - G_u) / \tan \omega$$

where:

- H_u = SRW unit height (ft)
- W_u = SRW unit width, front to back (ft)
- G_u = distance to the center of gravity of a horizontal SRW unit including aggregate fill, measured from the front of the unit (ft)
- ω = wall batter due to setback per course (deg)
- H = total height of wall (ft)
- H_h = hinge height (ft)

Figure 4-17. Determination of hinge height for modular concrete block faced MSE walls (NMCA, 1997).

4.4.7.j Lateral Movements

The evaluation of lateral wall movements in LRFD is the same as in ASD as the deformations are evaluated at the Service I limit state. In general, most internal lateral deformations of an MSE wall face usually occur during construction. Post construction movements, however, may take place due to post construction surcharge loads, settlement of wall fill, or long-term settlement of the foundation soils.

The magnitude of lateral displacement depends on fill placement techniques, compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-facing connection details, and details of the wall facing. The rough estimate of probable lateral displacements of simple MSE walls that may occur during construction can be estimated based on empirical correlations (see Figure 2-15). In general, increasing the length-to-height ratio of reinforcement, from its theoretical lower limit of $0.5H$ to the AASHTO specified $0.7H$, decreases the deformation by about 50 percent. For critical structures requiring precise tolerances, such as bridge abutments, more accurate calculations using numerical modeling may be warranted.

A deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to horizontal (and vertical) displacement. Horizontal deformation analyses are the most difficult and least certain of the performed analyses. In many cases, they are done only approximately. The results may impact the choice of facing, facing connections, or backfilling sequences.

4.4.7.k Vertical Movement and Bearing Pads

Bearing pads are placed in horizontal joints of segmental precast concrete panels in order to allow the panel and the reinforcement to move down with the reinforced fill as it is placed and settles, mitigate downdrag stress, and provide flexibility for differential foundation settlements. Internal settlement within the reinforced fill is practically immediate with some minor movement occurring after construction due to elastic compression in granular materials. The amount of total movement is the combination of the internal movement and external differential movement. The bearing/compression pad thickness and compressibility could be adjusted according to the anticipated movement. Otherwise concrete panel cracking and/or downdrag on connections resulting in bending of connections and/or out of plane panel movement can occur. Calculation of the external settlement was reviewed in Section 4.4.6.d. Normally the internal movement is negligible for well graded, granular fill and external movement will usually control the compression pad requirements as listed in Table 2-3. However, when using sand type fill and/or marginal fill containing an appreciable amount of fines, the internal movement can be significant and should be calculated to evaluate additional thickness requirements of the bearing pad. Immediate settlement of

granular fill can be calculated using the Schmertmann method, as described in the FHWA NHI-06-088 and NHI-06-089, *Soils and Foundation Reference* manuals (Samtani and Nowatzki, 2006).

The stiffness (axial and lateral), size, and number of bearing pads should be sized such that the final joint opening will be at least $\frac{3}{4} \pm 1/8$ -inch, unless otherwise shown on the plans. As noted in Chapter 2, a minimum initial joint width of $\frac{3}{4}$ -inch is recommended. The stiffness (axial and lateral), size, and number of bearing pads are should be checked assuming a vertical loading at a given joint equal to 2 to 3 times the weight of facing panels directly above that level. Laboratory tests in the form of vertical load-vertical strain and vertical load-lateral strain curves of the bearing pads are required for this check.

4.4.8 Step 8 – Design of Facing Elements

4.4.8.a Design of Concrete, Steel and Timber Facings

Facing elements are designed to resist the horizontal forces developed in Section 4.3. Reinforcement is provided to resist the maximum loading conditions at each depth in accordance with structural design requirements in Section 5, 6 and 8 of AASHTO (2007) for concrete, steel and timber facings, respectively. The embedment of the soil reinforcement to panel connector must be developed by test, to ensure that it can resist the T_{MAX} loads.

As a minimum, temperature and shrinkage steel must be provided for segmental precast panel facing. Epoxy protection of panel reinforcement or a minimum of 3 in. (75 mm) of concrete cover is recommended where salt spray is anticipated.

For modular concrete facing blocks (MBW), sufficient inter-unit shear capacity must be available, and the maximum spacing between reinforcement layers should be limited to twice the front to back width, W_u , as defined in Figure 4-17, of the modular concrete facing unit or 2.7 ft (32 in., 800 mm) whichever is less. The maximum depth of facing below the bottom reinforcement layer should typically be limited to the width, W_u (see Figure 4-17), of the modular concrete facing unit used. The top row of reinforcement should be limited to 1.5 the block depth (e.g. one unit plus a cap unit) (AASHTO 11.10.2.3.1 {2007}).

The factored inter-unit shear capacity as obtained by testing (ASTM D6916) at the appropriate normal load should exceed the factored horizontal earth pressure at the facing.

For seismic performance Zones 3 or 4, facing connections in modular block faced walls (MBW) should use shear resisting devices between the MBW units and soil reinforcement, and should not be fully dependent on frictional resistance between the soil reinforcement and

facing blocks. Shear resisting devices between the facing blocks and soil reinforcement such as shear keys, pins, etc. should be used. For connections partially or fully dependent on friction between the facing blocks and the soil reinforcement, the nominal long-term connection strength T_{ac} , should be reduced to 80 percent of its static value. Further, the blocks above the uppermost layer of soil reinforcement must be secured against toppling under all seismic events.

4.4.8.b Design of Flexible Wall Facings

Welded wire or similar facing panels should be designed in a manner which prevents the occurrence of excessive bulging as backfill behind the facing elements compresses due to compaction stresses, self weight of the backfill or lack of section modulus. Bulging at the face between soil reinforcement elements in both the horizontal and vertical direction generally should be limited to 1 to 2 in. (25 to 50 mm) as measured from the theoretical wall line. Specification requirements and design detailing to help achieve this tolerance might include limiting the face panel height to 18 in. (460 mm) or less, the placement of a nominal 2 ft (0.6 m) wide zone of rockfill or cobbles directly behind the facing, decreasing the vertical and horizontal spacing between reinforcements, increasing the section modulus of the facing material, and/or by providing sufficient overlap between adjacent facing panels. Furthermore, the top of the flexible facing panel at the top of the wall should be attached to a soil reinforcement layer to provide stability to the top of the facing panel.

Geosynthetic facing elements generally should not be left exposed to sunlight (specifically ultraviolet radiation) for permanent walls. If geosynthetic facing elements must be left exposed permanently to sunlight, the geosynthetic should be stabilized to be resistant to ultraviolet radiation. Furthermore, product specific test data should be provided which can be extrapolated to the intended design life and which proves that the product will be capable of performing as intended in an exposed environment. Alternately a protective facing should be constructed in addition (e.g., concrete, shotcrete, etc.).

4.4.9 Step 9 – Assess Overall/Global Stability

This design step is performed to check the overall, or global, stability of the wall. Overall stability is determined using rotational or wedge analyses, as appropriate, to examine potential failure planes passing behind and under the reinforced zone. Analyses can be performed using a classical slope stability analysis method with standard slope stability computer programs. In this step, the reinforced soil wall is considered analogous to a rigid body and only failure surfaces completely outside a reinforced zone (e.g., global failure planes) are considered. Computer programs that directly incorporate reinforcement elements

(e.g., ReSSA) can be used for analyses that investigate both global and *compound* failure planes. See Section 4.4.10 for failure planes that pass partially through the reinforced zone.

Per Article 11.6.2.3 AASHTO (2007), the evaluation of overall stability of MSE walls should be investigated at the Service I Load Combination, and using an appropriate resistance factor. Commonly used slope stability programs can be used to conduct this evaluation. The load factor at Service I limit state is 1.0 for permanent loads. In lieu of better information, the soil shear resistance factor (ϕ) is defined in Article 11.6.2.3 (AASHTO, 2007) as:

$\phi = 0.75$; where the geotechnical parameters are well defined, and the slope does not support or contain a structural element; and

$\phi = 0.65$; where the geotechnical parameters are based on limited information, or the slope contains or supports a structural element

The intent of the term “structural element” is that a resistance factor of 0.65 should be used for slope stability analysis if the slope/wall supports a bridge foundation, a building, or similar structure foundation that cannot tolerate significant movement or if the consequences of the failure of the supported structure are severe. A resistance factor of 0.75 may be more appropriate for slopes/walls that support structures such as a sign foundation where movements may not be detrimental or the consequences of the failure are not significant. The Agency/Owner should define whether the MSE wall structure itself is classified as a significant “structural element” (i.e., consequences of failure are severe) and a resistance factor of 0.65 is applicable, or if it is a minor structure and a resistance factor of 0.75 is applicable. (Also note that a slope supporting a structural element should have well defined geotechnical parameters.)

The codification of LRFD load and resistance factors by probabilistic calibrations for the design of slopes are currently being research and developed. Commercial slope stability analysis programs fully compatible with AASHTO LRFD procedures are not readily available. Therefore, designs today might be performed by traditional (non-LRFD) methods and with existing slope stability programs, and a comparison of computed safety factor to target resistance factor.

The AASHTO (2007) stated resistance factors of 0.75 and 0.65 are (generally) approximately equivalent to the safety factors of 1.3 and 1.5, respectively, that is:

$$\phi = 0.75 \Rightarrow 1/0.75 \approx 1.3 = \text{FS} \quad \text{and} \quad \phi = 0.65 \Rightarrow 1/0.65 \approx 1.5 = \text{FS}$$

Note that AASHTO resistance factors are stated to the nearest 0.05, so as to not overstate the level of accuracy of a resistance value. Therefore, if assessing global stability with limit equilibrium slope stability methods, the target safety factors are:

FS = 1.30 where the geotechnical parameters are well defined;

FS = 1.50 where the geotechnical parameters are based on limited information; and

FS = 1.50 where the wall/slope contains or supports a structural element.

This is consistent with past practice, per FHWA NHI-00-043 (Elias et al., 2001).

The evaluation of overall stability should be performed with reasonable estimates of short- and long-term water pressures (a geotechnical parameter) acting on the wall. If the evaluation of overall stability does not indicate a satisfactory result then the reinforcement length may have to be increased or the foundation soil may have to be improved. The design must be revised according to these changes.

Most agencies typically perform global stability assessments for MSE walls. Global stability generally is assessed by the agency during feasibility design, which might result in ground improvement or other wall options, and again after the wall is designed. The MSE wall vendors/suppliers typically exclude overall stability check and responsibility in their package unless contract documents require such an evaluation by the wall vendor/supplier.

4.4.10 Step 10 – Assess Compound Stability

Additional slope stability analyses should be performed for MSE walls to investigate potential *compound* failure surfaces, i.e., failure planes that pass behind or under and through a portion of the reinforced soil zone as illustrated in Figure 4-18. For simple structures with rectangular geometry, relatively uniform reinforcement spacing, and a near vertical face, compound failures passing both through the unreinforced and reinforced zones will not generally be critical. **However, if complex conditions exist such as changes in reinforced soil types or reinforcement lengths, high surcharge loads, seismic loading, sloping faced structures, significant slopes at the toe or above the wall, or stacked (tiered) structures, compound failures must be considered.**

This design step is performed to check potential compound failure planes passing through the reinforced soil zone. Compound stability is determined using rotational or wedge analyses, as appropriate, performed with computer programs that directly incorporate reinforcement elements (e.g., ReSSA) in the analyses. The reinforced soil wall is not considered a rigid body and is modeled with appropriate soil properties and the soil reinforcement layers as discrete elements. The long-term strength of each reinforcement layer intersected by the

failure surface should be considered as resisting forces in the limit equilibrium slope stability analysis. The facing system should be modeled with separate, but appropriate strength properties.

Proposed Procedure: Current AASHTO (2007) states that compound stability should be investigated. However, procedures (i.e., load and resistance factors) are not specifically defined. The recommended procedure is to follow global stability procedures and include reinforcement strength.

If assessing compound stability with limit equilibrium slope stability methods (e.g., modified Bishop, Spencer, etc.) a load factor of 1.0 should be used. Compound analyses should use the same AASHTO (2007) stated global stability resistance factors (ϕ) of 0.75 and 0.65. These resistance factors are approximately equivalent to safety factors of 1.3 and 1.5, respectively, as previously noted.

Therefore, if assessing compound stability with limit equilibrium slope stability methods, the target safety factors with limit equilibrium analysis are:

- FS = 1.30 where the geotechnical parameters are well defined;
- FS = 1.50 where the geotechnical parameters are based on limited information; and
- FS = 1.50 where the wall/slope contains or supports a structural element

This is consistent with past practice, per FHWA NHI-00-043 (Elias et al., 2001).

Note, however, that the method of incorporating the soil reinforcement strength into the stability calculations does affect the magnitude of factor of safety computed. See Section 9.3 for recommendations on how reinforcement strength should be incorporated.

The evaluation of compound stability should be performed with reasonable estimates of short- and long-term water pressures. If the evaluation of compound stability does not indicate a satisfactory result then the reinforcement length, reinforcement strength, reinforcement vertical spacing, and/or depth of wall may have to be increased, or the foundation soil may have to be improved. The design must be revised according to these changes, and compound stability rechecked as appropriate.

Compound stability analyses require detailed information on both the subsurface conditions (typically defined by the agency) and the soil reinforcement layout (typically vendor defined). Unlike global stability analyses, the responsibility for this analysis is not clearly defined. Agencies should perform an initial assessment of a proposed MSE wall structure with an assumed reinforcement layout to determine if compound stability is a concern and

must be addressed in final design. Typical geometries where compound stability is of concern are illustrated in Figure 4-18. Generally, MSE wall vendors/suppliers exclude compound stability check and responsibility in their package, unless specifically required by the Owner.

Compound stability can be addressed by selecting one of the following three options for specifying and bidding the MSE wall (Schwanz et al., 1997):

1. Agency Design. Agency prepares complete design for the MSE wall including external, internal, global, and compound stability analyses. This requires material specifications for all wall components.
2. Vendor Design. Agency prepares line and grade plans, and allows approved vendors to supply the complete design and wall components. Agency is responsible for and must provide detailed subsurface profile(s), soil shear strength, soil unit weight, and groundwater information for the vendor to use in external, global, and compound stability analyses. Agency should perform a feasibility analysis to ensure global stability can be achieved with the line and grade provided to the vendors.
3. Combined Design. Agency prepares line and grade plans, assesses global and compound stability requirements, and specifies/detail reinforcement requirements for adequate stability resistance. For example, the agency might specify two layers of reinforcement within a range of elevations (at bottom of wall) with minimum strength and minimum lengths required. Wall vendor completes wall design with incorporation of reinforcement required for adequate compound stability resistance.

Applications of the above three options by the St. Paul District of the U.S. Army Corps of Engineers are summarized by Schwanz et al. (1997). Advantages and disadvantages with each option are discussed in the cited reference.

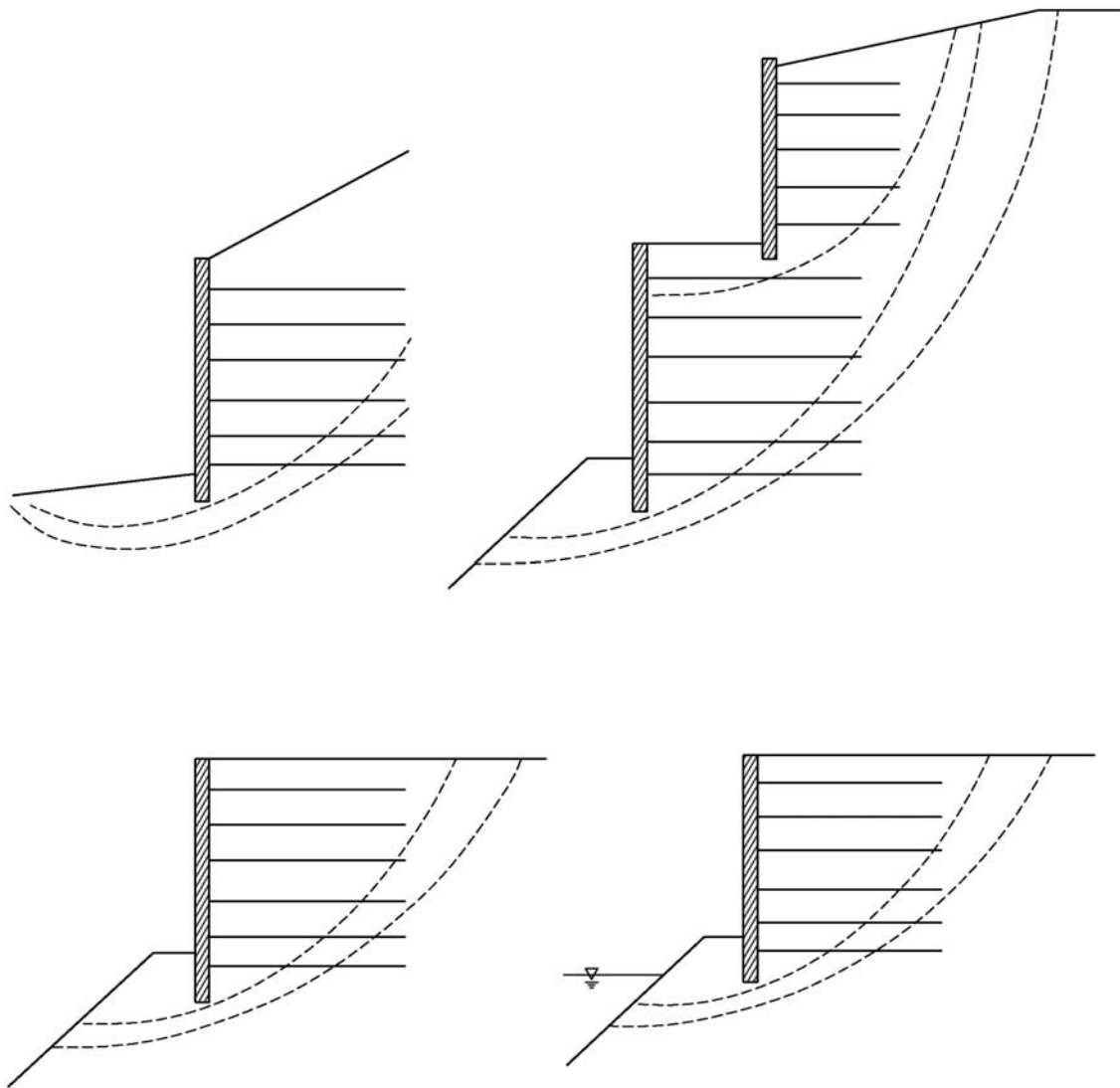


Figure 4-18. Typical geometries where MSE wall compound stability is of concern: steep and tall backslope on top of the wall; tiered walls; slope at the toe of the wall; and water at toe of the slope.

4.4.11 Step 11 – Wall Drainage Systems

Drainage is a very important aspect in the design and specifying of MSE walls. The Agency should detail and specify drainage requirements for vendor designed walls. Furthermore, the Agency should coordinate the drainage design and detailing (e.g., outlets) within its own designers and with the vendor. The Agency is also responsible for long-term maintenance of drainage features, as discussed in Section 5.3.4.

4.4.11.a Subsurface Drainage

Subsurface drainage must be addressed in design. The primary component of an MSE wall is soil. Water has a profound effect on this primary component of soil, as it can both decrease the soil shear strength (i.e., resistance) and increase destabilizing forces (i.e., load). Thus, FHWA recommends drainage features be required in all walls unless the engineer determines such feature is, or features are, not required for a specific project or structure.

Drainage design and detailing are addressed in Section 5.3. Note that MSE walls using free draining reinforced fill do not typically need a full drainage system, but do need a method for discharging water collected within the reinforced wall fill. Also note that MSE walls can be designed for water loads, if needed. Basic soil mechanics principles should be used to determine the effect of phreatic surface on wall loads. See discussion in Chapter 7 for design of MSE walls for flood and scour events.

4.4.11.b Surface Water Runoff

Surface drainage is an important aspect of ensuring wall performance and must be addressed during design and during construction. Appropriate drainage measures to prevent surface water from infiltrating into the wall fill should be included in the design of a MSE wall structure. Surface drainage design and detailing are addressed in Section 5.3.

4.4.11.c Scour

There are additional detailing considerations for walls that are exposed to potential scour. The wall embedment depth must be below the Agency predicted scour depth. Wall initiation and termination detailing should consider and be design to protect from scour. Riprap may be used to protect the base and ends of a wall. A coarse stone wall fill may desired to drain rapidly. The reinforced wall fill at the bottom of the structure may be wrapped with a geotextile filter to minimize loss of fill should scour exceed design predictions. These items are discussed in detail in Chapter 5.

4.5 TEMPORARY WALLS

Temporary walls are normally considered wall structures with a 36 month or less service life (Article 11.5.1 {AASHTO, 2007}). The design method remains the same as for permanent walls, except for the calculation of the soil reinforcement long-term nominal strength, T_{al} . Metallic soil reinforcements are not normally galvanized for temporary walls. An exception might be when aggressive wall fill materials are being used and galvanization is specified to provide corrosion resistance.

The long-term nominal strength for black steel (i.e., non-galvanized) in non-aggressive reinforced fill soil may be calculated with the whole steel cross-section for temporary walls. The long-term nominal strength for black steel (i.e., non-galvanized) and non-aggressive wall fill soil may be calculated with a corrosion rate of 1.1 mils/yr (28 $\mu\text{m}/\text{yr}$) (FHWA NHI-09-087 {Elias et al., 2009}). Higher corrosion rates need to be considered for reinforced fills that are moderately aggressive or corrosive, and a corrosion specialist should be consulted to assess the sacrificial steel requirements or other possible corrosion protection measures. Steel reinforcement should be galvanized if a service life greater than 36 months is required for a temporary structure.

For geosynthetic soil reinforcements, the long-term nominal strength may be calculated with a minimum durability reduction factor of 1.0 in lieu of 1.1 minimum used for permanent walls. This is for temporary walls and for geosynthetics that meet the minimum requirements listed in Table 3-12.

4.6 DESIGN CHECKLIST

Agencies should have an established, or should establish a, protocol for checking designs. This is particularly important for vendor supplied designs, but should also be used with in-house designs. The protocol should assign responsibilities for the review and list items that should be checked. Thus, the protocol can be in the form of a checklist.

Based upon work by the Arizona Department of Transportation (ADOT), an example design checklist follows. This example may be used by agencies to develop their own checklist with their defined responsibilities and references to the agency's standard specifications, standard provisions, etc. Some of the items on the following checklist are project specific, and others are project and wall structure specific.

MSE WALLS - DESIGN REVIEW EXAMPLE CHECKLIST

TO BE FILLED BY RESIDENT ENGINEER

Project (Name, Contract No., etc.)	
Resident Engineer (RE)	
Date MSE submittal received	
Is this a re-submittal? If yes, attach previous checklist	
Name of Engineer of Record (ER)	
Date submittal transmitted to ER	
Date comments due back to RE	

<u>TO BE FILLED BY ER</u>	REVIEWED BY				
	Materials Group Due Date**	Date Received	Date Reviewed	Name	Organization
Professional Engineer of Record*(ER)					
Date completed checklist sent to RE					
* Contact designated agency Design Engineer immediately upon receipt of the submittal(s) from RE. ** Due date for submittal to Design Engineer.					

This checklist has been completed under the supervision of the Professional Engineer of Record whose seal and signature appears hereon.	

LEGEND FOR ABBREVIATIONS / ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
DE	Agency Design Engineer from assigned to the project.
APL	Approved Products List (For latest APL visit _____).
ER	Engineer of Record (Registered Professional Engineer in the state)
FHWA	Federal Highway Administration
MBW	Modular Block Wall
MSE	Mechanically Stabilized Earth
MSEW 3.0	Version 3.0 of proprietary software, MSEW, by ADAMA Engineering (visit www.geoprograms.com)
NA	Not Applicable
NHI	National Highway Institute
PE	Professional Engineer of Record (shall be a Registered Professional Engineer)
PGR	Project Geotechnical Report
Project Drawings	Complete final plan set for the project
RE	Resident Engineer
Section #, Figure # or Table #	This refers to an appropriate section, figure or table in the following manual by FHWA/NHI: “Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,” Publication No. FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, (Authors: Ryan R. Berg, Barry R. Christopher and Naresh C. Samtani)
Spec	Project specification including standard specification and special provisions
_____	_____ Department of Transportation
Vendor Drawings	Working drawings provided by MSE wall vendor

All symbols used within the questions are consistent with those used in the documents in the “Reference” column

NOTES FOR CHECKLIST

1. The following information/material should be collected before starting the checklist:
 - a. Contractor submittals (transmittal letter, design drawings, design calculations)
 - b. Project documents (final plan set, standard specifications, special provisions, Project Geotechnical Report)
 - c. FHWA/NHI manual (“Mechanically Stabilized Earth Walls and Reinforced Soil Slopes,” Publication Nos. FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, December 2009; Authors: Ryan R. Berg, Barry R. Christopher and Naresh C. Samtani)
 - d. Latest version of AASHTO LRFD Bridge Design Specifications, including interims
 - e. All due dates for checklist
 - f. Name of the structural engineer
 - g. Name of the roadway engineer
 - h. Name of “prime” designer
2. Each question must have a “Yes”, “No” or “NA” box checked. Any comment or action required should be entered in the “Comments/Action Required” column. If the “No” or “NA” box is checked then an appropriate comment or action required must be entered. Use separate sheets if comments cannot be fitted in the space within the checklist.
3. The documents listed under the “Reference” column in the checklist are not intended to be a complete list of documents. Rather, the most common documents are listed where guidance/information related to the question in the checklist may be found. More stringent criteria may exist in other project documents (e.g., drainage, signage, utilities, etc.) that may be relevant to a given question. In such an event, the governing document should be noted in the “Comments/Action Required” column of the checklist.
4. Add any pertinent project specific questions to the checklist as necessary. Two empty rows are provided at end of each section for this purpose. Use additional sheets as necessary if more space is required.
5. This checklist is intended to be completed, signed, and sealed by the Engineer of Record who is Registered Professional Engineer in the state.
6. The Engineer of Record should contact the project structural or roadway engineer in case of discrepancies between the contractor submittals and reference documents.
7. Wall details that were reviewed and approved as part of the “Approved Products List” will be available on a website; contact the _____ for further information.
8. After completing the checklist the ER should include an attachment that identifies specific questions that the MSE wall vendor (or in-house designer) has to address.

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
I.	GENERAL INFORMATION					
1.	Is the wall vendor pre-approved? (visit _____ for a list of pre-approved wall systems)	APL	☐	☐	☐	
2.	Is the wall within the limitations of the pre-approved product? (e.g., wall height, external loading, environmental constraints, seismic loading and other project specific constraints; visit _____ for limitations)	APL	☐	☐	☐	
3.	Has the Contractor used the correct design survey data (e.g., existing ground elevations and horizontal offsets) for wall design?	Project/vendor drawings	☐	☐	☐	
4.	Has the Contractor correctly reflected the location of utilities in the area of the wall(s)?	Project/vendor Drawings	☐	☐	☐	
5.	Is the wall profile (top and bottom elevations) including start and end stations correct?	Project/vendor Drawings	☐	☐	☐	
6.	Is the wall design life specified?	Spec/ Section 2.8	☐	☐	☐	
7.	Have the following items been specified by the vendor and are they in conformance with the project requirements?					
	a. Material requirements					
	i. Soil Properties (strength, gradation, PI, soundness, electrochemical)	Spec	☐	☐	☐	
	ii. Soil Reinforcement (ultimate and yield tensile strengths, reduction factors for geosynthetics)	Spec	☐	☐	☐	
	iii. Concrete (strength and other properties)	Spec/Project Drawings	☐	☐	☐	
	iv. Concrete reinforcement (type, number and strength)	Spec/Project Drawings	☐	☐	☐	
	v. Leveling Pad (strength)	Spec/Project Drawings	☐	☐	☐	
	vi. Steel facing elements for wire mesh systems (ultimate and yield tensile strengths)	Spec	☐	☐	☐	
	b. Construction procedures including sequence	APL	☐	☐	☐	
	c. Soil compaction procedures and restrictions for reinforced fill, retained fill and foundation preparation	APL/spec/ PGR	☐	☐	☐	
	d. Facing alignment tolerances	Spec	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
	e. Acceptance/rejection criteria (tolerances, facing finish, etc.)	Spec	☐	☐	☐	
	f. Corrosion protection systems for soil reinforcement	Spec	☐	☐	☐	
	g. Handling and storage of reinforcements	Spec/APL/ PGR	☐	☐	☐	
8.	Is the initial wall batter during construction specified?	APL	☐	☐	☐	
9.	Are the structural (select) backfill dimensions shown?	Spec	☐	☐	☐	
10.	Are the wall quantities (area of wall, volume of structural fill, etc.) listed in accordance with the pay quantity schedule in the project specifications?	Spec	☐	☐	☐	
11.	Wall installation guide		☐	☐	☐	
	a. Has the proprietary vendor submitted a wall installation guide?	APL	☐	☐	☐	
	b. Does the submitted wall installation guide address site-specific conditions?	PGR/Spec	☐	☐	☐	
12.	Is the Contractor's transmittal letter acceptable? (e.g., does it contain acceptable statements consistent with the submittal?)		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
II.	TOP OF WALL					
1.	Do the top of wall elevations match the roadway design elevations?	Project drawings	☐	☐	☐	
2.	Are top of wall elevations such that they can allow for proper interfacing with barriers, copings, surface ditches, bridge abutments, etc. as shown on the plans?	Project drawings	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
III.	LEVELING PAD (Note: Only lean concrete leveling pads are allowed)					
1.	Are the leveling pad dimensions shown?	Spec	☐	☐	☐	
2.	Does the leveling pad profile satisfy the minimum depth of embedment criteria?	Section 2.8/ PGR/ Project Drawings	☐	☐	☐	
3.	Are the leveling pad elevations such that they allow for transverse and longitudinal	Project drawings	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
	drainage structures shown on the plans?					
4.	Are leveling pad steps such that they can accommodate the bottom row facing unit type and size without cutting and/or splicing of the facing units?	APL/vendor drawings	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
IV.	FACING UNITS AND JOINTS					
1.	Are the facing units from the pre-approved list?	APL	☐	☐	☐	
2.	Do facing units meet the project aesthetic criteria?	Spec/Project Drawings	☐	☐	☐	
3.	Have the material properties of the facing units been specified? (Examples: density, strength, freeze-thaw, etc.)	Section 4.4.8/Spec	☐	☐	☐	
4.	Are the materials properties of the facing units in conformance with the project criteria? (Examples: density, strength, freeze-thaw, etc.)	Section 4.4.8/Spec				
5.	Are the facing units structurally adequate as per the project facing unit structural criteria and/or per AASHTO? (deformation of facing elements including local bending should be within allowable limits)	Section 4.4.8/Spec	☐	☐	☐	
6.	Is the horizontal joint width between facing units in conformance with project criteria?	Section 2.8, Table 2-1	☐	☐	☐	
7.	Does the joint bearing pad material conform to project specifications?	Spec	☐	☐	☐	
8.	Is the joint bearing pad material of proper compressive strength such that facing unit to facing unit crushing and /or high stress concentrations on any facing units are prevented?	Spec/APL	☐	☐	☐	
9.	For Modular Block Wall (MBW) units with geosynthetic soil reinforcement has the hinge height concept been used for establishing connection details?	Section 4.4.7	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
V.	DRAINAGE					
1.	Are all vertical and horizontal joints covered with geotextile fabric on the backside of the wall facing units?	Spec	☐	☐	☐	
2.	Is the geotextile fabric covering the joints of sufficient width and continuous across the joints?	Spec	☐	☐	☐	
3.	Do the geotextile fabric properties (survivability, filtration and permittivity) covering the joints meet project specifications?	Spec	☐	☐	☐	
4.	Has drainage along the backcut been included as per project criteria?	PGR/Spec	☐	☐	☐	
5.	If geocomposite is used for drainage, then is it pre-approved and do its properties (flow capacity, filtration and permeability) meet project requirements?	PGR/APL/ Spec	☐	☐	☐	
6.	Is the water from subsurface drainage adequately led out of the wall system? e.g., collector and drain system with weepholes, grades towards wall ends, etc.	PGR/Spec	☐	☐	☐	
7.	Is surface drainage in accordance with project criteria?	Project Drawings	☐	☐	☐	
8.	If Modular Block Wall (MBW) units are used for facing then has adequate drain fill been provided?	Section 5.35/Figure 5.6/Spec	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
VI.	SPECIAL WALL DETAILS					
1.	Are wall interface details with other walls that will be constructed before, during, or after this contract shown?	Spec/ Section 5.5	☐	☐	☐	
2.	Are following special wall details shown and are they adequate?:					
	a. special facing element if interfacing with other wall systems	Spec/APL/ Section 5.5	☐	☐	☐	
	b. slip joint(s) (e.g., at wing walls, differential settlement concerns, etc.)	Spec/APL	☐	☐	☐	
	c. wall end(s)	Spec/APL	☐	☐	☐	
	d. connection to appurtenances (e.g., box inlets and large obstructions)	Spec/APL/ Section 5.5	☐	☐	☐	
	e. acute angles	Spec/APL/ PGR	☐	☐	☐	
	f. coping	Spec/APL/ Section 4.5	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
	g. railing, guard rails or traffic barriers	Spec/APL/ Section 5.1	☐	☐	☐	
	h. miscellaneous obstructions (e.g., utilities) below ground elevation	Spec/APL/ Section 4.55.4	☐	☐	☐	
	i. measures to prevent migration of de-icing salts in the reinforced fill	Spec/APL/ Section 5.3	☐	☐	☐	
	j. measures to protect against rapid drawdown conditions and hydrostatic pressures	Spec/APL/ Section 4.55.3	☐	☐	☐	
3.	Are structural frames (“yokes”) provided to navigate the bar mat soil reinforcements around vertical obstructions within the MSE backfill? (examples of vertical obstructions include piles, shafts, inlet structures, etc.)	Spec/APL/ Section 4.55.4	☐	☐	☐	
4.	Are the structural frames designed properly so that moments and torques are not introduced in the bar mat soil reinforcements and/or the reinforcement/facing unit connection?	APL/ Bridge Group	☐	☐	☐	
5.	Is the splay of strip reinforcements limited to less than 15 degrees?	Spec	☐	☐	☐	
6.	If strip reinforcements are splayed, then is the length increased to compensate for reduction in effective length?	PGR/Spec	☐	☐	☐	
7.	Is the maximum vertical bend (maximum 15 degrees) in metallic soil reinforcements within acceptable limits?	Spec/ Section 5.4	☐	☐	☐	
8.	Are geosynthetic reinforcement details around vertical obstructions acceptable?	APL	☐	☐	☐	
9.	Are overlapping reinforcements separated vertically by at least 3-in. of soil?	Spec	☐	☐	☐	
10.	If walls are tiered, then are they in accordance with project criteria?, e.g., bench widths, aesthetics within benches, etc.	Spec/ Section 6.2	☐	☐	☐	
11.	If instrumentation is required per project specs, then is it provided? (List the instrumentation in the comments column)	PGR/ Spec	☐	☐	☐	
12.	Are corrosion/durability protection details acceptable?	Spec/ Section 3.5	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
VII.	SOIL REINFORCEMENT					
1.	Is the soil reinforcement type (extensible or inextensible) and configuration (strip, grid or sheet) in conformance with pre-approved list?	APL	☐	☐	☐	
2.	Are the following soil reinforcement dimensions in conformance with those approved by the Agency during the pre-approval process?	APL				
	a. strip thickness or bar diameter	APL	☐	☐	☐	
	b. strip width or bar mat width	APL	☐	☐	☐	
	c. center to center spacing of the longitudinal bars in bar mats	APL	☐	☐	☐	
	d. center to center spacing of the transverse bars in bar mats	APL	☐	☐	☐	
	e. Geosynthetic grid (uniaxial/biaxial) openings and junction sizes	APL	☐	☐	☐	
3.	Is the connection of the soil reinforcement to the facing units as per the pre-approved connection detail?	APL	☐	☐	☐	
4.	Is the soil reinforcement specified to have the correct type and thickness of the corrosion protection as per the project specifications?	Spec/ Section 3.5	☐	☐	☐	
5.	Is all soil reinforcement, except at acute angle corners, perpendicular to the face of the wall facing units? If no, please comment.	Spec/ PGR	☐	☐	☐	
6.	Is all soil reinforcement connected to facing units?	Spec	☐	☐	☐	
7.	If metallic soil reinforcements are cut and/or spliced then have the corrosion protection measures at cuts/connections been provided and are they acceptable? (Note: cutting transverse bars of bar mats is not allowed)	Spec/ APL	☐	☐	☐	
8.	Are means and methods for splicing of geosynthetic reinforcement (overlap, mechanical connections, edge seams, etc.) in accordance with that approved by the Agency during the pre-approval process?	APL	☐	☐	☐	
9.	Are placement procedures for reinforcement acceptable?	APL	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
VIII.	EXTERNAL STABILITY					
1.	Have all assumed soil parameters (Cohesion, Angle of Internal Friction, Soil Unit Weight, and Sliding Friction Coefficient) for retained, reinforced and foundation soils been listed?	PGR/Spec/ Section 3.3, 4.4.6	☐	☐	☐	
2.	Are soil parameters consistent with those recommended in the geotechnical report / project specifications?	PGR/Spec	☐	☐	☐	
3.	Have the maximum bearing pressures been listed along the length of the wall?	Vendor drawings	☐	☐	☐	
4.	Have all the loads been incorporated into the wall analysis and design? (e.g., traffic loads, seismic loads, sloping surcharge, broken-back surcharges, etc.)	PGR/ Section 4.4.4	☐	☐	☐	
5.	Have all the critical sections along all walls been analyzed? (e.g., highest wall sections, sections where slopes above and below the walls are steepest, etc.)	Project Drawings/ PGR	☐	☐	☐	
6.	Are the static and seismic analyses adequate (as per performance requirements) for the following failure modes?	Spec/Section 4.4.6, 7.1.1	☐	☐	☐	
	a. Sliding	Spec/Section 4.4.6.a	☐	☐	☐	
	b. Eccentricity (overturning)	Spec/Section 4.4.6.c	☐	☐	☐	
	c. Bearing	Spec/Section 4.4.6.c				
	i. General bearing capacity	Spec/Section 4.4.6.c	☐	☐	☐	
	ii. Local bearing capacity / lateral squeeze	Spec/Section 4.4.6.c	☐	☐	☐	
	iii. Is the bearing resistance greater than the maximum bearing pressure at all locations along the wall?	PGR	☐	☐	☐	
7.	Is the wall embedment equal to or greater than the project requirements?	PGR	☐	☐	☐	
8.	Has total settlement analysis been performed?	PGR	☐	☐	☐	
9.	Has differential settlement analysis been performed?	PGR	☐	☐	☐	
10.	Have slip joints been provided to prevent stresses due to large anticipated differential settlements?	PGR/APL/ Section 5.4.5	☐	☐	☐	
11.	Is an undercut needed due to soft or poor soils? If so, is the depth of treatment and the replacement material specified?	PGR/ Spec	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
12.	Will deep foundations be needed for very deep layers of soft/loose soils?	PGR/ Spec	☐	☐	☐	
13.	Will waiting period(s) and stage construction be needed if the design wall pressure exceeds the maximum allowable bearing pressure?	PGR/ Spec	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
IX. INTERNAL STABILITY						
1.	Have calculations for internal stability of the wall been performed?	PGR/ Spec	☐	☐	☐	
2.	Has the static and seismic internal stability evaluation been performed by the "Simplified Method"?	PGR/Spec/ Section 7.1	☐	☐	☐	
3.	Have all the critical sections along all walls been analyzed? (e.g., highest wall sections, sections where slopes above and below the walls are steepest, etc.)	Project Drawings/ PGR	☐	☐	☐	
4.	Is pullout resistance adequate at each level of the reinforcement?	PGR/Spec/ Section 4.7.h	☐	☐	☐	
5.	Is the correct value of nominal strength of steel used? (e.g., $0.55 F_y$ for strips and $0.48F_y$ for bar mats)	PGR/Spec/ Section 3.5	☐	☐	☐	
6.	Are corrosion loss rates in conformance with project criteria?	PGR/Spec/ Section 3.5	☐	☐	☐	
7.	Has the cross-sectional area for the soil reinforcement been corrected for corrosion losses over the design life of the structure?	PGR/Spec/ Section 3.5	☐	☐	☐	
8.	Is resistance against tensile failure adequate at each level of reinforcement?	PGR/Spec/ ection 4.4.7.f	☐	☐	☐	
9.	Are the connections designed for maximum tension in soil reinforcements?	Spec/Section 4.4.7.i	☐	☐	☐	
10.	Have the proper values of F^* (including C_u , F_q , α_β , $\tan\phi$ and variation with depth) been used?	Section 3.4, 4.4.7.h	☐	☐	☐	
11.	Is the correct value for the scale correction factor, α , been used?	Section 3.4, 4.4.7.h	☐	☐	☐	
12.	Is the correct value of unit perimeter, C , used?	Section 3.4, 4.4.7.h	☐	☐	☐	
13.	For geosynthetic reinforcement have the reduction factors for creep (RF_{CR}), durability (RF_D) and installation damage (RF_{ID}) been specified and are they acceptable?	Section 3.5/ Spec	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
14.	For geosynthetic reinforcement is the computation of long-term allowable strength acceptable?	Section 3.5/ Spec	☐	☐	☐	
15.	Have the correct stress ratio (K_r/K_a) and lateral pressure coefficient (K_a) been used for computing internal loads?	Section 4.4.7.c, Figure 4-10	☐	☐	☐	
16.	Has the correct internal failure surface been used for static and seismic cases?	Section 4.4.7.b	☐	☐	☐	
17.	Has the vertical stress been computed as per the requirements of the Simplified Method?	Section 4.4.7.e				
18.	Are the definitions of the reinforcement configuration (grid openings, ratios of the bar diameters to spacing of bars in bar mats, etc.) consistent with pre-approved product list?	APL/Section 3.4	☐	☐	☐	
19.	Have all the external loads been incorporated into the wall analysis and design? (e.g., traffic impact loads, seismic loads, sloping surcharge, broken-back surcharges, etc.)	Section 4.4.5, 7.1.1	☐	☐	☐	
20.	Have all the internal loads been incorporated into the wall analysis and design? (e.g., lateral loads from piles at abutments or overhead mast structures)	PGR/Spec/ Section 4.4.7, 6.1	☐	☐	☐	
21.	Has the internal stability evaluation accounted for complex geometries such as tiered structures, acute corners, back-to-back walls, and obstructions?	PGR/Spec/ Section 6.1 – 6.6	☐	☐	☐	
22.	Is the vendor's analysis acceptable to the Geotechnical Engineer of Record based on an independent verification using "Simplified Method" and MSEW 3.0 or hand calculations? Please attach a copy of the verification calculations using the Simplified Method.	GER/ PGR	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
X.	GLOBAL / COMPOUND STABILITY					
1.	Has the owner's geotechnical engineer of record checked global stability?	PGR	☐	☐	☐	
2.	Has the vendor checked compound stability?	PGR/Spec/ Section 4.4.10	☐	☐	☐	
3.	Has the vendor checked the global stability?	PGR/Spec	☐	☐	☐	

		Reference (See Note 3)	Yes	No	NA	Comments/ Action Required
4.	Is the safety factor against global stability failure adequate?	PGR/Spec	☐	☐	☐	
5.	Is the safety factor against compound stability failure adequate?	PGR/Spec	☐	☐	☐	
6.	Are the geotechnical parameters for global and compound stability analyses appropriate and consistent with those used for other failure modes?	PGR/Spec	☐	☐	☐	
7.	Is ground improvement needed based on global stability analysis?	PGR	☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
XI.	FILE INFORMATION					
1.	Has the Geotechnical Engineer of Record completed this checklist? If not, who?		☐	☐	☐	
2.	Has a representative from agency's _____ Group ensured that this checklist has been completed and outstanding issues identified?		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	
	{add as appropriate, may be agency or project specific}		☐	☐	☐	

LIST OF ATTACHMENTS BY ENGINEER OF RECORD

No.	Attachment	Comments/Action Required
1		
2		
3		
4		
5		
6		
7		
8		
9		
10		

As a minimum the ER should include an attachment that identifies the specific issues that need to be addressed by the MSE wall designer (vendor).

4.7 COMPUTER-AIDED DESIGN

The repetitive nature of the computations required at each level of reinforcement lends itself to computer-assisted design. The computer program MSEW (ADAMA, 2000) developed under FHWA sponsorship analyzes and/or designs MSE walls using any type of metallic or geosynthetic reinforcement in conjunction with any type of facing (precast concrete, MBW, etc.). Version 1.0 has been designated exclusively for use by U.S. State Highway Agencies and by U.S. Federal agencies and performs computations in compliance with the ASD design methods in FHWA (Elias et al., 2001) and AASHTO (2002). Version 3.0 is available for purchase through ADAMA Engineering (www.MSEW.com) and includes LRFD-based computations. Alternatively, spreadsheet based solutions can be developed. The example problems in Appendix E, provide comprehensive step-by-step solutions that can be easily programmed into a spreadsheet.

Other MSE wall analysis and design programs are available. Many wall vendors have their own programs that are tailored to their system, and may have additional features for estimating quantities and costs. Agency personnel should understand the features and finer points of the computer program and spreadsheets that they use to design or check vendor designs. Likewise, wall vendors and design consultants should understand the features and finer points of computer programs and spreadsheets they use. This is particularly important with the recent change to an LRFD design platform.

4.8 VENDOR DESIGNS

As previously discussed, it is recommended that Agencies use a pre-approved proprietary wall system list (an approved products list) for specifying MSE walls with a performance or end-result approach. Specific wall systems and respective vendors, along with any application restrictions (e.g., height limit), are provided on the list. Detailed evaluations are typically required for placement on an approved products list. The design program and spreadsheets used by the vendor should be reviewed by the Agency as part of this evaluation.

4.9 STANDARD MSEW DESIGNS

MSEW structures are customarily designed on a project-specific basis. Most agencies use a line-and-grade contracting approach, with the contractor selected MSEW vendor providing the detailed design after contract bid and award. This approach works well for segmental and full-height panel faced walls, and can be used for MBW unit faced walls. However, standard

designs can be developed and implemented by an agency for MSEW structures, somewhat similar to standard concrete cantilever wall designs used by many agencies.

Use of standard designs for MSEW structures could offer the following advantages over a line-and-grade approach:

- Agency is more responsible for design details and integrating wall design with other components.
- Pre evaluation and approval of materials and material combinations, as opposed to evaluating contractor submittal post bid.
- Economy of agency design versus vendor design/stamping of small walls.
- Agency makes design decisions versus vendors making design decisions.
- More equitable bid environment as agency is responsible for design details, and vendors are not making varying assumptions.
- Reduces the possibility of substandard work, systems and designs with associated approved product lists.

The Minnesota Department of Transportation (Mn/DOT), with support of the FHWA (via Demo 82 project) developed and implemented standardized MSEW designs (Berg, 2000) for MBW unit faced and geosynthetic reinforced MSEW structures. The use of these standard designs are limited by geometric, subsurface and economic constraints. Structures outside of these constraints should be designed on a project-specific basis. The general approach used in developing these standards could be followed by other agencies to develop their own, agency-specific standard designs.

Standardized designs require generic designs and generic materials. Generic designs require definition of wall geometry and surcharge loads, soil reinforcement strength, structure height limit, and MBW unit properties of width and batter. As an example, the Mn/DOT standard designs address four geometric and surcharge loading cases, and could be used for walls up to 23 ft (7 m) in height. Since original development the number of cases has been reduced to three and the maximum height has been reduced to 12 ft (3.6 m) due to MBW durability concerns (see Section 3.6.2 for discussion on MBW freeze-thaw durability).

Definition of generic material properties for the standard designs requires the development of approved product lists for MBW units, soil reinforcement and MBW unit-soil reinforcement combinations. The combinations require a separate approved product list as the connection strength is specific to each unique combination of MBW unit and reinforcement, and often controls the reinforcement design strength. An additional requirement for MBW units is an approved manufacturing quality control plan on file with the agency. This requirement is a

result of the stringent durability (to freeze thaw and deicing salt conditions) specifications for the units and the long duration testing used to demonstrate durability.

An example design cross section and reinforcement layout table from the Mn/DOT standard designs are presented in Figure 4-19 and Table 4-8. A list of approved combinations (Mn/DOT, 2009) of MBW units and soil reinforcements with classification as MBW-700, MBW-1050, or MBW-1400, is used in conjunction with the table and figure. Note that the Mn/DOT standard designs are not directly applicable to, nor should they be used by, other agencies.

Another example of an agency standard design is Washington DOT's geosynthetic walls. Standard designs for two stage construction of walls up to 35 ft (11 m) are provided. The geosynthetic wrap-around walls are constructed in the first stage. The walls can be faced with shotcrete or cast-in-place concrete in the second stage.

Table 4-8. Example MBW Faced MSE Wall Standard Design (Minnesota DOT, 2008).

MODULAR BLOCK WALL REINFORCEMENT LAYOUT — CASE 4 - 1:3 FILL SLOPE —														
MBW Reinforcement Class	Strength of Soil Reinf. (plf)		Minimum Reinforcement Length, L (ft)	Maximum Wall Height (ft)	Nominal Block Width (in.)	Wall Batter Range (degrees)		Maximum Unreinforced Wall Height (in.)	Zone 1		Zone 2		Zone 3	
	L. Term (Tal)	Design (Ta)				≥	<		H1 (ft)	S1 _{max} (in.)	H2 (ft)	S1 _{max} (in.)	H3 (ft)	S1 _{max} (in.)
MBW-700	1050	700	0.7 H	12.0	12	0	3	24	8.5	24	3.5	16		
						3	7	24	9.2	24	2.8	16		
						7	10	24	11.2	24	0.8	16		
						10	15	24	12.0	24				
					21	0	3	42	4.6	32	3.9	24	3.5	16
						3	7	42	5.2	32	3.9	24	2.9	16
						7	10	42	5.2	32	5.9	24	0.9	16
						10	15	42	5.9	32	6.1	24		
MBW-1050	1575	1050	0.7 H	12	12.0	0	3	24	12.0	24				
						3	7	24	12.0	24				
						7	10	24	12.0	24				
						10	15	24	12.0	24				
					21	0	3	42	5.6	42	3.3	32	3.1	24
						3	7	42	8.2	42	2.6	32	1.2	24
						7	10	42	8.5	42	3.5	32		
						10	15	42	9.8	42	2.2	32		
MBW-1400	2100	1400	0.7 H	12	12.0	0	3	24	12.0	24				
						3	7	24	12.0	24				
						7	10	24	12.0	24				
						10	15	24	12.0	24				
					21	0	3	42	8.9	42	3.1	32		
						3	7	42	10.8	42	1.2	32		
						7	10	42	12.0	42				
						10	15	42	12.0	42				

CHAPTER 5

MSE WALL DETAILS

Proper attention to details of various components is critical to the successful implementation of MSE wall projects. This Chapter discusses various details related to the following elements of MSE walls:

- Top of wall elements such as copings, traffic barriers and geomembrane caps
- Bottom of wall elements such as leveling pads
- Drainage features such as filters, drains and pipes
- Internal elements such as obstructions in reinforced soil mass and slip joints
- Wall initiations and terminations
- Aesthetics

The example details shown in this chapter have been used successfully in actual projects. However, these details may need modifications to fit the requirements of specific projects. Therefore, the user should treat the details in this chapter as initial guidance and modify them as appropriate before actual implementation on a given project and for a given product.

5.1 TOP OF WALL ELEMENTS

The top of wall is important from both aesthetic as well as technical aspects. Aesthetically, the top of a MSE wall should provide a smooth profile. Technically, the top of wall needs to integrate roadway elements such as pavements, traffic barriers, and drainage features.

5.1.1 Copings

The purpose of a coping is to create a smooth and aesthetically pleasing clean line at the top of an MSE wall. Copings can be cast-in-place or precast. Precast coping can generally be installed more rapidly than cast-in-place coping. Figures 5-1a to 5-1c provide common details for cast-in-place and precast copings for segmental precast concrete facing units. Figure 5-1d shows a common detail for a precast cap unit on top of modular block wall.

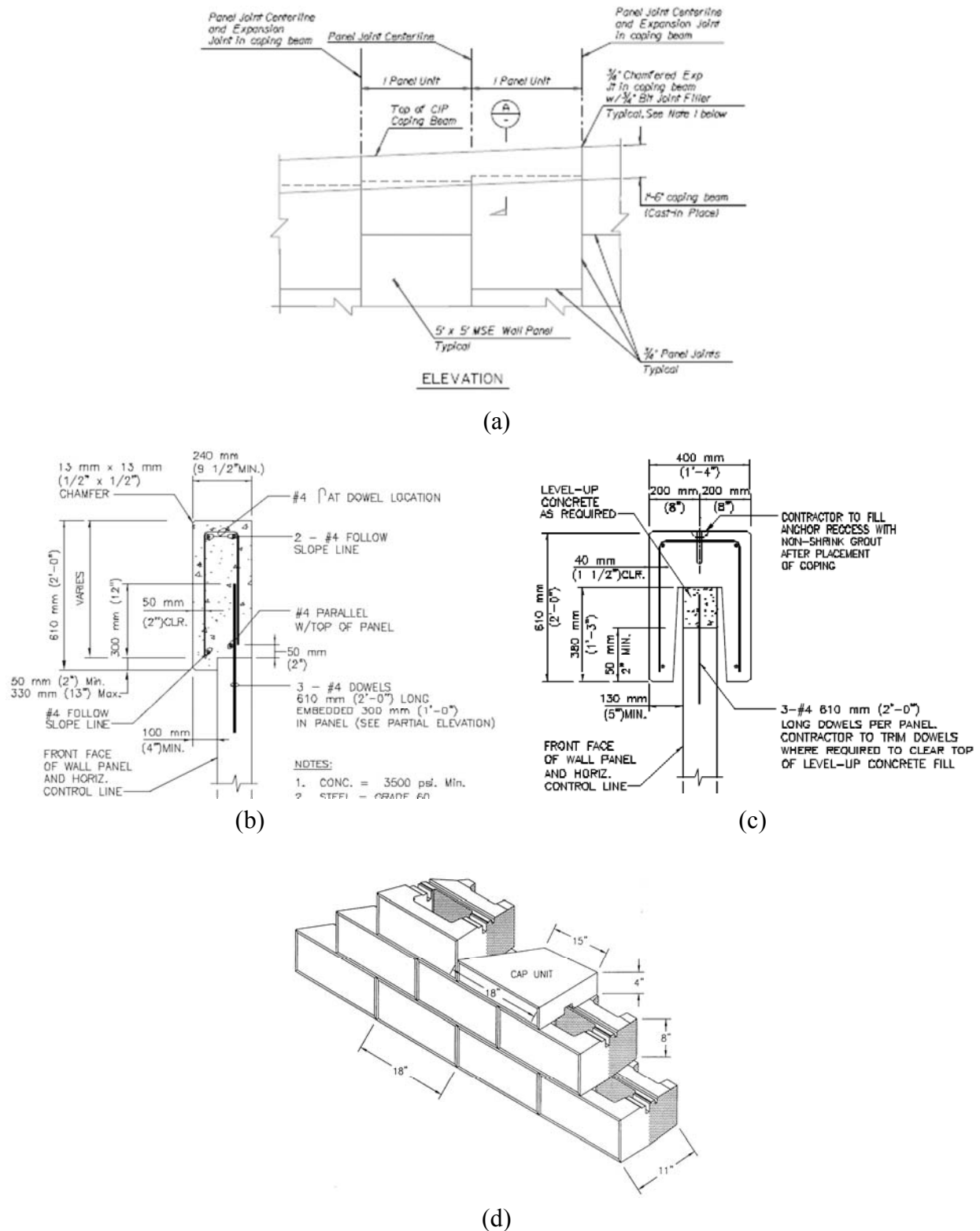


Figure 5-1. Example copings for MSE walls.

Regardless of whether the coping is cast-in-place or precast, the coping should have full-depth open joints, i.e., reinforcement should not be continuous across the joints. General guidance for copings is as follows:

- For segmental precast concrete facing units, the joints for a cast-in-place coping should align with the vertical joints in the MSE wall face at a frequency not exceeding 10 ft (3 m) on centers with a preferable frequency of every panel width. The spacing of the joints may be increased to 20 ft (6 m) if differential settlement is not a concern.
- For modular block facing units, the joints in a cast-in-place coping should line up with the vertical joints in the face at a frequency not exceeding 5 ft (1.5 m) or less as required to line up with block joints. The spacing of the joints may be increased to 10 ft (3 m) if differential settlement is not a concern. Precast cap units should have a width equal to the width of a modular block unit and should be attached to the top modular facing unit using a mortar and pin connection. Adhesives should not be used for permanent structures unless the agency is prepared to perform continuous maintenance to check and reattach the cap blocks as necessary. Warranties for separation and displacement of glued cap blocks are, at best, usually on the order of ten years and that assumes that blocks meet the required installation conditions of the adhesive manufacturer, e.g., the blocks are clean, dry and bonded at the required curing temperature.

The cast-in-place coping can provide a smooth finish and be adjusted to meet final top of wall elevations after settlements have occurred. Cast-in-place copings are also recommended in situations where a wall follows a horizontal or vertical curve with less than a 100 ft (30 m) curve. Since precast coping sections are cast with square ends, joints between coping sections as seen from the front of the wall may become too tight or too wide depending on whether the radius point is in front or behind the wall face, respectively. Custom fitted cast-in-place coping should be used at kink points and corners in the wall and at slip joints so that the in-plane movement on each side of the slip joint can be tolerated without compromising the purpose of the slip joint.

Before installing precast coping, the top of the wall must be smooth and free of steps or irregularities. To accomplish this, level-up concrete is cast on top of the facing units. The smooth finished grade of this concrete fill should follow a line approximately 9 in. (225 mm) below the top of coping elevation. Top facing units that are to receive precast coping may have protruding dowels that tie in to the level-up concrete. The dowels are field trimmed 1 to 2 in. (25 to 50 mm) below the top of level-up concrete before pouring the level-up concrete.

5.1.2 Traffic Barriers

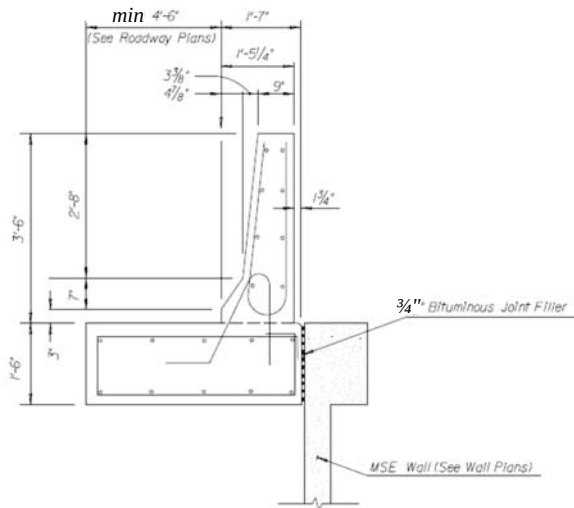
Figure 5-2 presents a variety of traffic barrier configurations. Typically, the base (or moment) slab length is a minimum of 20 ft (6 m) and jointed to adjacent slabs with shear dowels. The width typically varies from 4 ft (1.2m) to 6 ft (1.8m). The actual designs of traffic barriers should be in accordance with AASHTO (2007). In all cases, the base slab must be sized to prevent overturning and sliding of the barrier system during impact. When the base slab extends over the tops of the facing units to form a coping, a recess into which the facing units fit must be designed in the underside of the slab and a positive bond breaker must be provided to ensure isolation of the barrier from the facing units. Both vertical and horizontal bond breaks are required to avoid direct impact loads on the facing unit and to prevent prying loads on the top panels during traffic loading. If a precast coping or precast traffic barrier is used, the top of the wall must be smooth and free of steps or irregularities. To accomplish this, level-up concrete fill is cast on top of the facing units (similar to that for coping).

5.1.3 Parapets

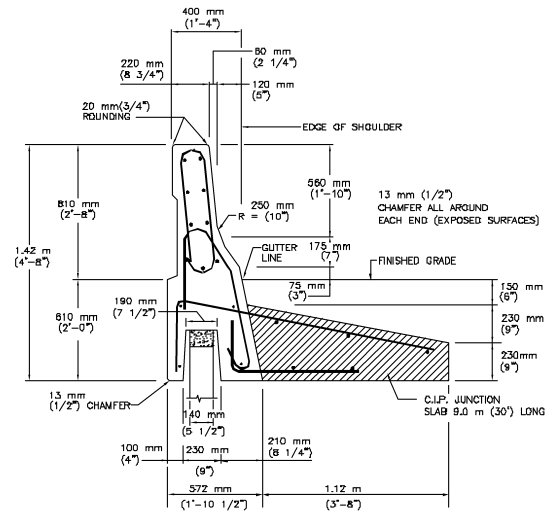
Where only pedestrian or bicycle loads are anticipated, the safety railing may be in the form of a concrete pedestrian parapet. A parapet is a cast-in-place or precast concrete rail located directly or nearly on top of the facing units. Although not designed for vehicular impact loading parapets do use a moment slab for stability. The moment slab may also serve as a sidewalk. The moment slab should be strong enough to resist the nominal (ultimate) strength of the pedestrian parapet. Where there is a possibility of vehicular load, the parapet should be protected by a non-mountable curb at the edge of the traveled roadway or the parapet be designed for impact load.

5.1.4 Post and Beam Barriers

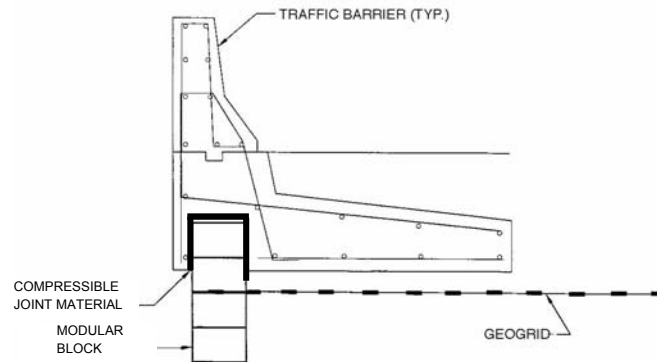
Where post and beam barriers such as guardrail systems are used, the posts are driven directly into the reinforced soil mass or installed in concrete-filled forms placed during backfill placement and compaction. The posts should be placed at a minimum distance of 3 ft (1 m) from the wall face, driven 5 ft (1.5 m) below grade, and reinforced spaced to miss the posts where possible. If the reinforcements cannot be missed, the wall should be designed to account for the presence of an obstruction as discussed later in this Chapter.



(a)



(b)



(c)

Figure 5-2. Example traffic barrier for MSE walls. (a) Barrier behind coping, (b) barrier on top of panels, (c) barrier on top of modular block units.

5.1.5 Drainage Related Top of Wall Elements

Whenever possible, the top surface of wall should be graded such that water drains away from the wall. A grassed swale or concrete ditch can be used behind the facing to collect and remove water. However, when this is not possible, depending on the configuration of the backslope with respect to the top of the wall and local hydrogeological considerations, several different details may need to be implemented from a drainage perspective. These elements are discussed in Section 5.3.

5.2 BOTTOM OF WALL ELEMENTS

The primary bottom of wall element is a leveling pad. Figure 5-3 shows common details of a leveling pad. Following are some considerations for the leveling pad:

- For structural walls, the leveling pad should be constructed from lean (e.g., 2,500 psi {17.2 MPa}) unreinforced concrete. The strength and thickness should be such that it allows cracking of the leveling pad during differential settlement as/if needed to relieve stress concentrations that can occur. Gravel pads may be allowed only for non-structural walls such as those for landscaping purposes.
- The common thickness of a leveling pad is 6 in. (150 mm). The width of the leveling pad should be such that it extends at least 3 in. (75 mm) beyond the thickness of the facing unit. Thus, for example, if the segmental precast concrete facing unit is 6 in. (150 mm) thick, then the width of the leveling pad shall be at least 12 in. (300 mm). At sharp curves the width of the leveling pad may be increased for segmental precast concrete facing units which are typically 5 ft (1.5 m) or 10 ft (3 m) wide so that the entire panel is resting on the leveling pad and at least 3 in. (75 mm) overhang of the leveling pad on each side of the facing unit.
- For ease of construction and to prevent misalignment of joints, the top of the leveling pad within any given step should be such that it does not vary by more than 1/8 in. (3 mm) over any 10 ft (3 m) run.
- Any openings between leveling pad steps should be completely filled after erection of the first row of panels. Where openings are more than 3 in. (75 mm) wide, filling with lean unreinforced cast-in-place concrete is preferred. For smaller openings, a geotextile filter with sufficient overlap of the panels and foundation soil could be used to fill openings.

When there is a roadway immediately adjacent to the bottom of the wall, it is advisable to provide a crash protection barrier to protect the wall against vehicular impact. The size and configuration of this barrier is a function of the vehicle sizes and speed limits on the roadway in front of the wall. An alternative to a crash barrier is a non-mountable, i.e., a high, curb at the edge of the traveled roadway adjacent to the wall.

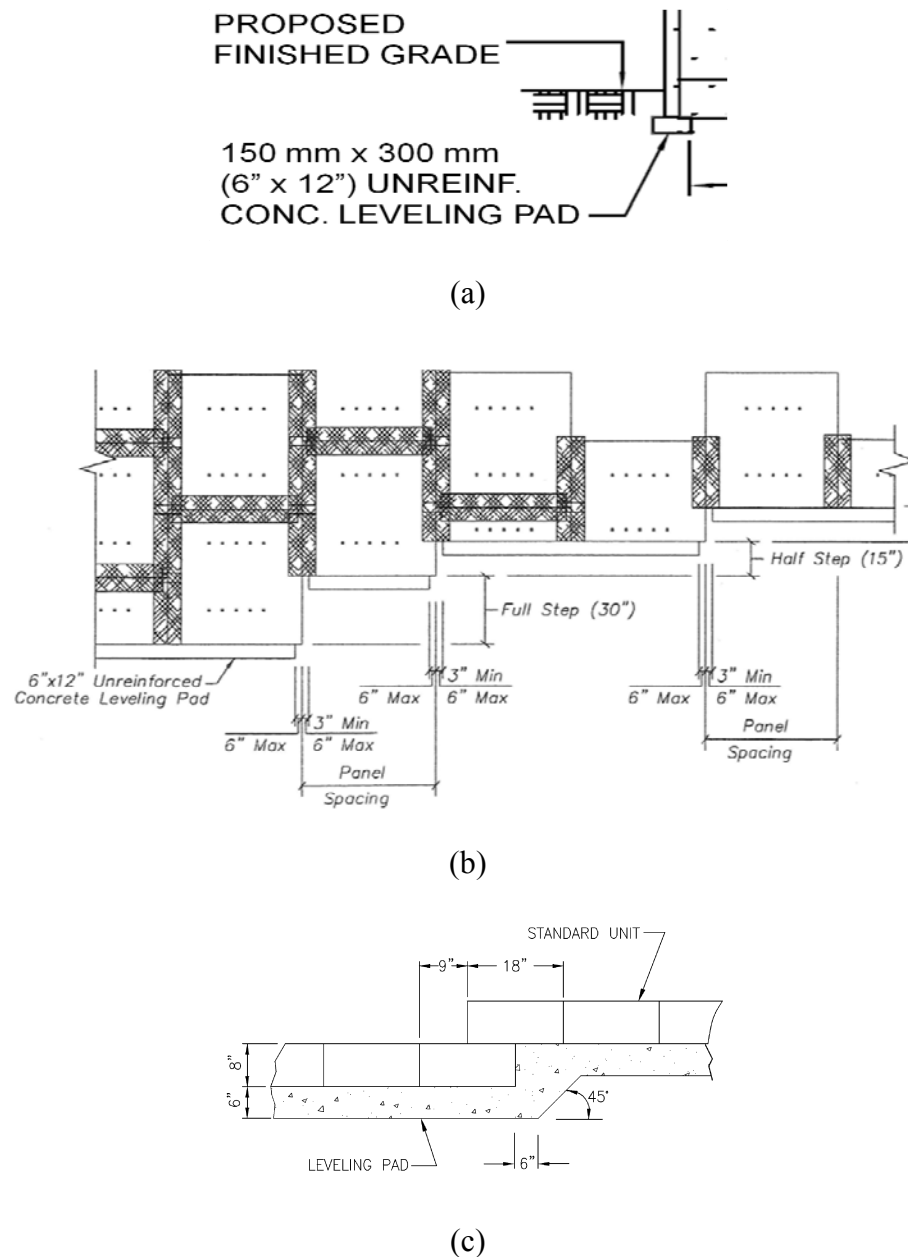


Figure 5-3. Leveling pads, (a) Common size, (b) Step detail for precast panel facing units, (c) Step detail for modular block facing units.

5.3 DRAINAGE

Good drainage is essential to the proper performance of an MSE wall. There are two types of drainage considerations for an MSE wall, internal and external. Internal drainage considerations are related to control of surface or subgrade water that may infiltrate the reinforced soil mass. The internal drainage of an MSE wall depends on the characteristics of the backfill used in the reinforced soil mass. External drainage considerations deal with water that may flow externally over and/or around the wall surface taxing the internal drainage and/or creating external erosion issues. The external drainage depends on the location of the MSE wall with respect to local hydrogeological factors and generally deals with diverting water flow away from the reinforced soil structure.

Regardless of the source of the water, i.e., internal or external, the cardinal rule in the design of MSE walls, as with any other wall type, is to allow unimpeded flow of water through the wall and/or collect and remove water before it enters the zone of influence of the wall to prevent the following:

- build-up of hydrostatic forces that increase lateral pressures,
- piping, i.e., erosion of one soil into another, which creates paths for additional water flow or clogging of drainage aggregate, and
- external soil erosion from the toe, around the edges or at the top of the wall.

It is recommended that adequate drainage features be required for all walls unless the engineer determines that such features are not needed for a specific project. During a determination of the need for drainage features, the engineer must include consideration for both subsurface (e.g., ground water, perched water, flooding and tidal action) and surface infiltration water (e.g., rain, runoff, and snow melt).

Effect of Fines on Drainage

Soil particles with sizes smaller than the U.S. No. 200 (0.075 mm) sieve are referred to as “fines.” The permeability of an overall soil mass is affected significantly by the amount of fines. In general, soils with less than 3 to 5% non-plastic fines by weight are considered to be free draining and water can readily flow through the soil mass even under low hydraulic gradients. In the case of MSE walls, a reinforced soil mass with less than 3 to 5% non-plastic fines will allow unimpeded flow provided the permeability of the reinforced fill is greater than the permeability of the retained fill and the wall is not exposed to significant water events such as flooding, tidal action or significant snow melt.

When the amount of the fines is more than 3 to 5%, the permeability is significantly reduced and drainage requirements must be carefully evaluated as groundwater and/or infiltration of surface water can result in build-up of seepage/hydrostatic forces within the reinforced soil mass. Surface water that infiltrates into the reinforced soil mass will tend to move toward the permeable face of an MSE wall and can have a destabilizing effect due to a potential increase in seepage forces (Terzaghi et al., 1996; Cedergren, 1989). Such a condition can occur during severe rainstorms, if the permeability of the fill is equal to or less than about 0.002 cm/sec (Terzaghi et al., 1996; Cedergren, 1989). Therefore, good drainage features should be incorporated into the design if low permeability reinforced fill is used, i.e., if the reinforced fill has more than 3 to 5% non plastic fines. Special precaution is also advised for hillside construction due to the potential for seepage to occur through retained soil and rock seams, faults and joints during rain events that may not be apparent during subsurface exploration and construction.

Internal and external drainage details, which represent good drainage, are presented in the following Sections 5.3.1 and 5.3.2, respectively. Good design of drainage features requires proper consideration of the filtration properties of various geomaterials within and external to the MSE wall as well as drains that are adequately sized to effectively remove any seepage water. The drainage components including filtration criteria and drain component requirements are presented in Section 5.3.3.

5.3.1 Internal Drainage Systems

There are two specific forms of internal drainage as shown in Figure 5-4, (a) drainage near wall face due to infiltration of surface water near the wall face, and (b) drainage behind and under reinforced soil mass from groundwater. Groundwater may be present at an elevation above the bottom of the wall and would flow to the MSE walls from an excavation backcut; or it may be present beneath the bottom of the MSE wall. A groundwater surface beneath a MSE wall may rise into the reinforced soil mass, depending on the hydrogeology of the site. Surface water may infiltrate into the reinforced soil mass from above or from the front face of the wall, for the case of flowing water in front of the structure.

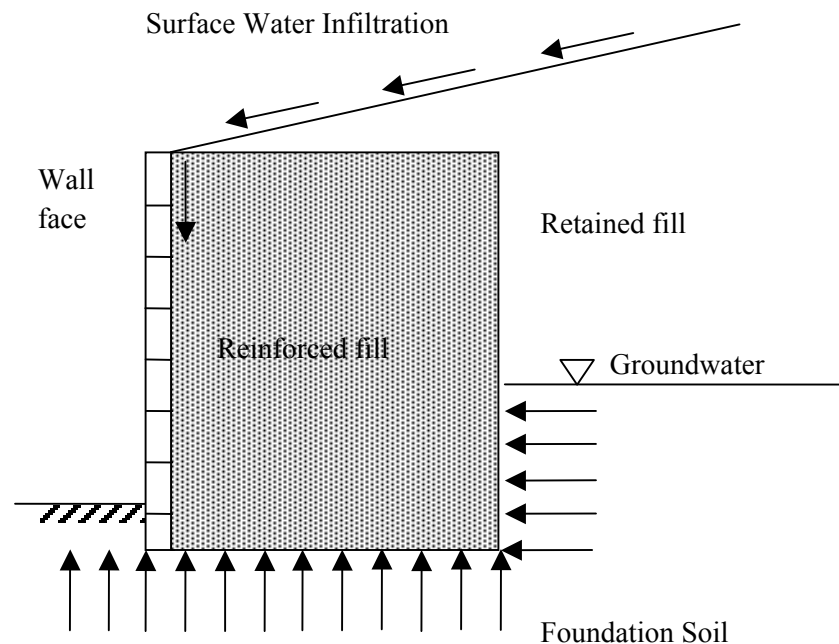
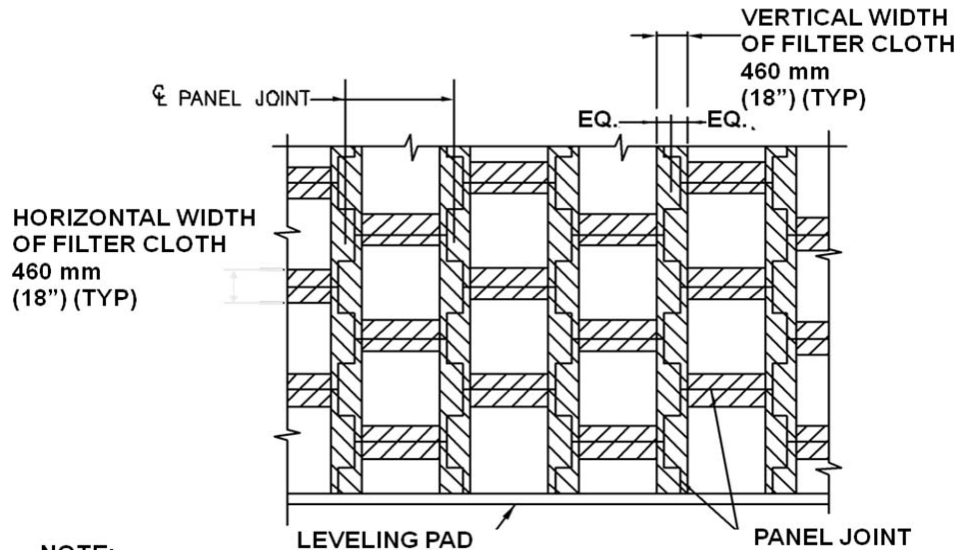


Figure 5-4. Potential sources and flowpaths of water.

Internal Drainage Near Wall Face

A filter is provided at all vertical and horizontal joints in the wall face to prevent the migration of fines from the reinforced soil mass through the joints. The location and configuration of the filter is a function of the type of wall facing units as follows:

- For segmental precast wall facing units, the filter is commonly in the form of geotextile fabric that is placed across all horizontal and vertical joints as shown in Figure 5-5. The geotextile should extend a minimum of 4 in. (100 mm) on either side of the joint and up into the coping to prevent soil from moving around the geotextile. The geotextile filter characteristics should be such that it is compatible with the backfill in the reinforced soil mass as discussed in Section 5.3.3.

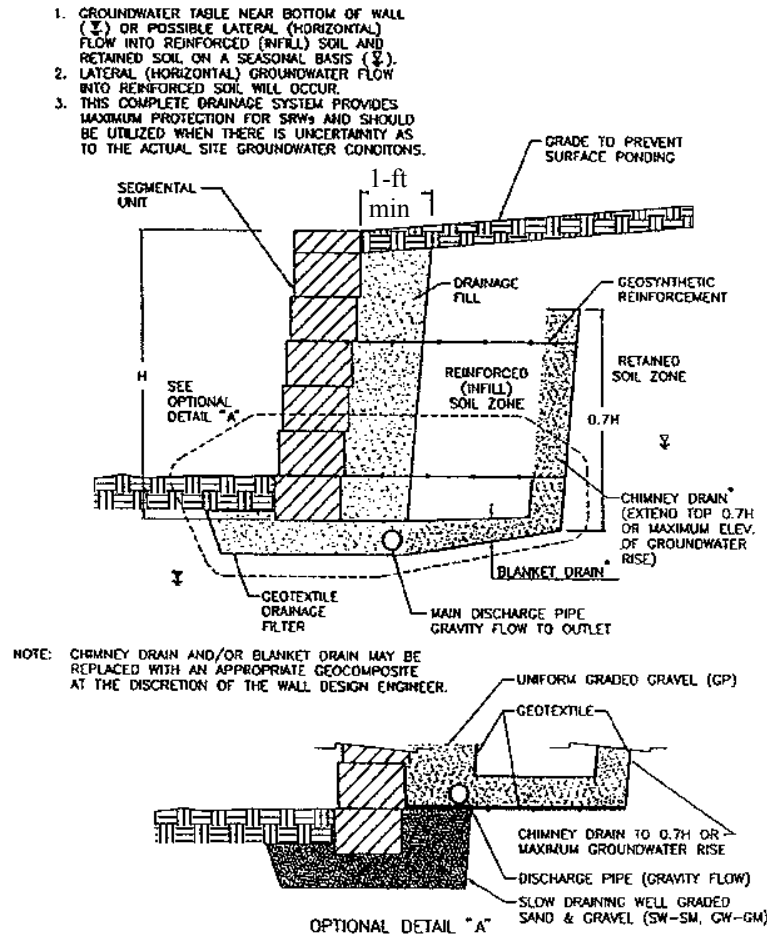


NOTE:
 STRIPS OF FILTER CLOTH SHALL BE PLACED ON BACK FACE OF PANEL, OVER PANEL JOINTS. FILTER CLOTH SHALL BE ADHERED TO BACK FACE OF PANELS USING AN ADHESIVE SUPPLIED BY THE MANUFACTURER.

Figure 5-5. Example layout of filter at joints between segmental precast facing units.

- Modular block wall (MBW) facing units are typically constructed with a zone of free drainage aggregate adjacent to the back face of the units. The minimum width of this aggregate zone is typically 1 ft (300 mm). In addition to serving as a back face drain, this aggregate is required for stiffness of the wall face and constructability, i.e., placement and compaction of wall fill may be difficult based on the configuration of the MBW units. This column of aggregate is often a high permeability well graded gravel as discussed in Section 5.3.3. The gradation of the aggregate should be used to determine the maximum allowable vertical joint opening between MBW units, using slot criterion given by Equation 5-8 in Section 5.3.3. The configuration of the gravel filter is a function of whether the modular block unit is solid or with a hollow-core. For solid modular block units, the well graded gravel should be at least 1 ft (300 mm) wide as shown in Figure 5-6a. For hollow-core modular block units, the well graded gravel should be at least 1 ft (300 mm) wide with a minimum volume of 1 ft³ per ft² (0.3 m³/m²) of wall face as illustrated in Figure 5-6b. The gradation of the gravel should be sized to be compatible with the reinforced wall fill gradation in the reinforced soil mass, i.e. meet soil filter criteria as discussed in Section 5.3.3. Alternatively, a geotextile may be used between the gravel and reinforced wall fill to meet filtration requirements, as illustrated in Figure 5-6b. Finally, the construction sequence should be specified to ensure a workable drain system.

(a)



(b)

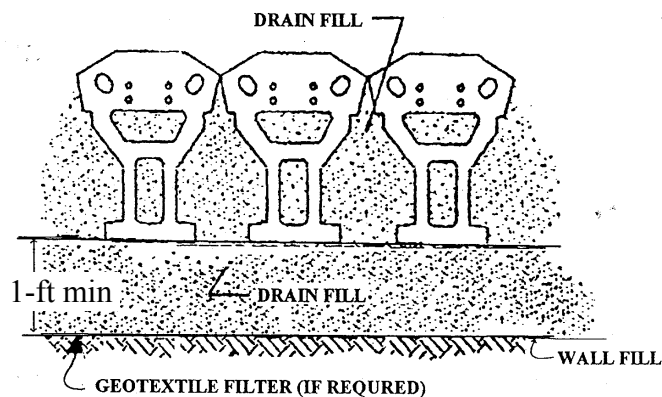


Figure 5-6. Layout of drainage fabric and drainage fill at the face for modular block units. (Collin et al., 2002).

- Figure 5-7 provides a common detail for face drainage in the case of wire-faced walls. In this case, the geotextile filter is placed between the facing stones and the reinforced soil mass.

Internal Drainage Under and Behind the MSE Wall

For walls in locations where groundwater can result in build-up of seepage/hydrostatic forces within the height of the reinforced soil mass and/or surface water infiltration is anticipated, a base drain that provides drainage beneath the MSE wall and a back or chimney drain that provides drainage behind the reinforced soil mass is strongly recommended to ensure proper long-term functionality of the MSE wall. This is because, as noted earlier, a reinforced fill with more than 3 to 5% non plastic fines is not “free draining.”

The base drain and back drain should be designed to collect and remove groundwater before it enters the reinforced mass and allows infiltration water to preferentially flow downward and toward the back of the wall, away from the face. An example of such a drainage system is illustrated in Figure 5-8 for segmental precast facing unit structure. Figure 5-6a shows a common detail for modular block unit faced structures. Figures 5-9 and 5-10 show alternative drainage systems that include geocomposite drains and blanket drains in lieu of open graded gravel drains with a geotextile or well-graded soil filter. Information on the various drains to relieve hydrostatic pressures is provided below. Design of the base drain and backdrain and the drainage system components is covered in Section 5.3.3.

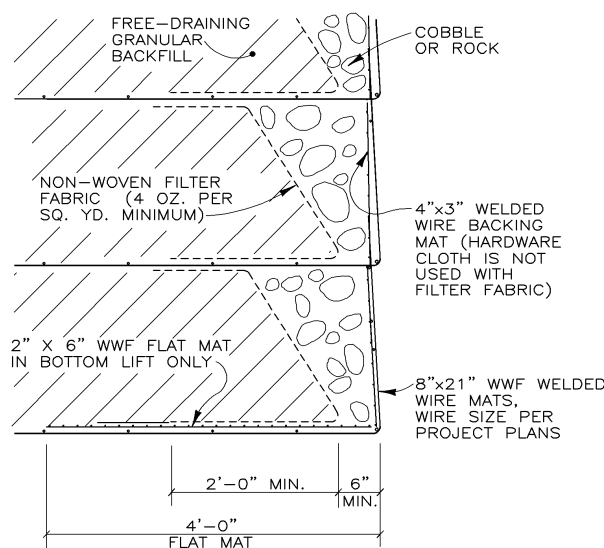


Figure 5-7. Example layout of geotextile filter near the face for welded wire facing units.

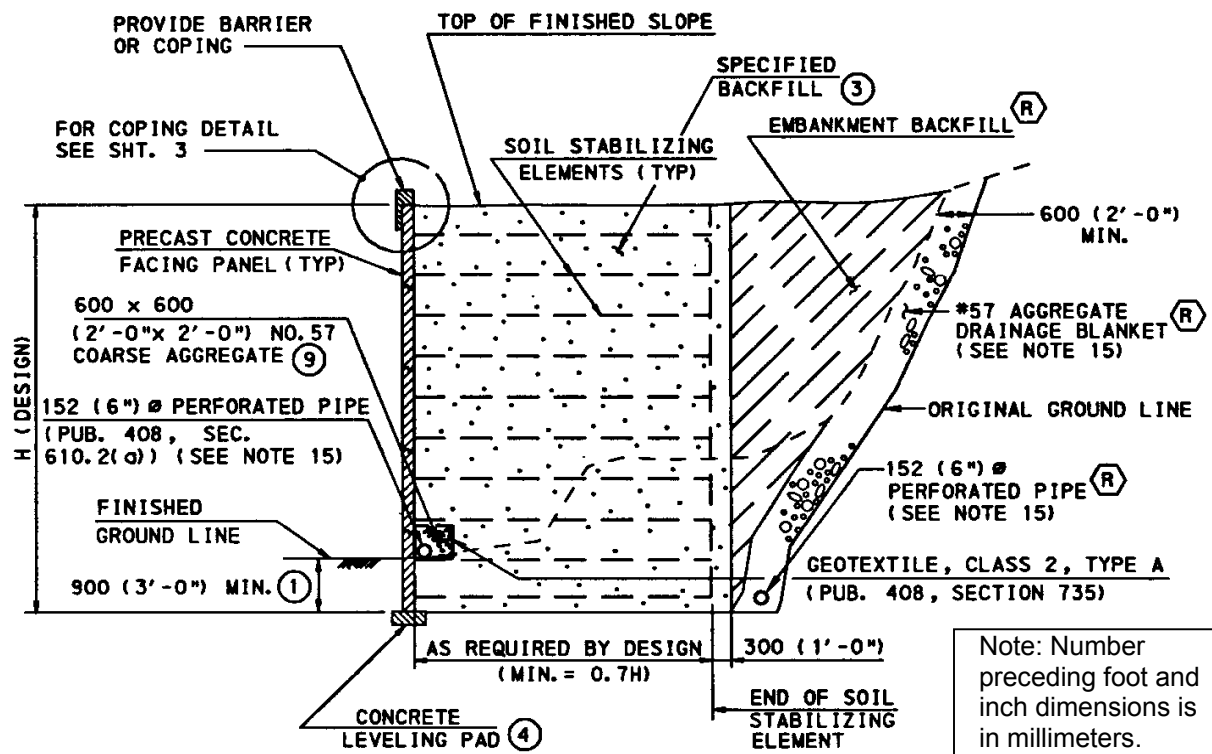


Figure 5-8. Example drainage blanket detail behind the retained backfill.

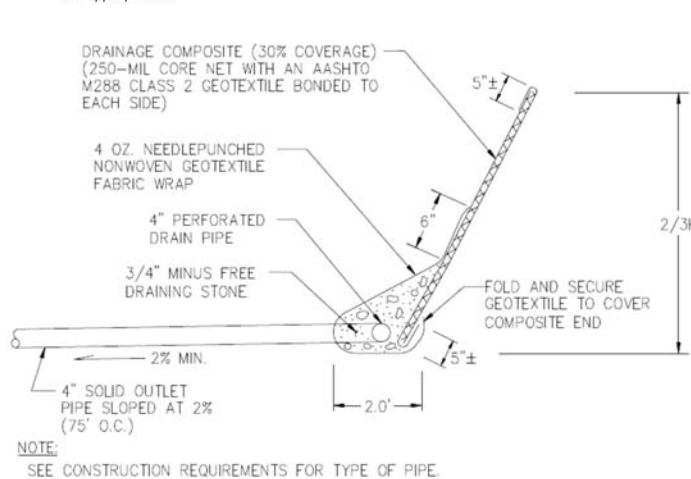
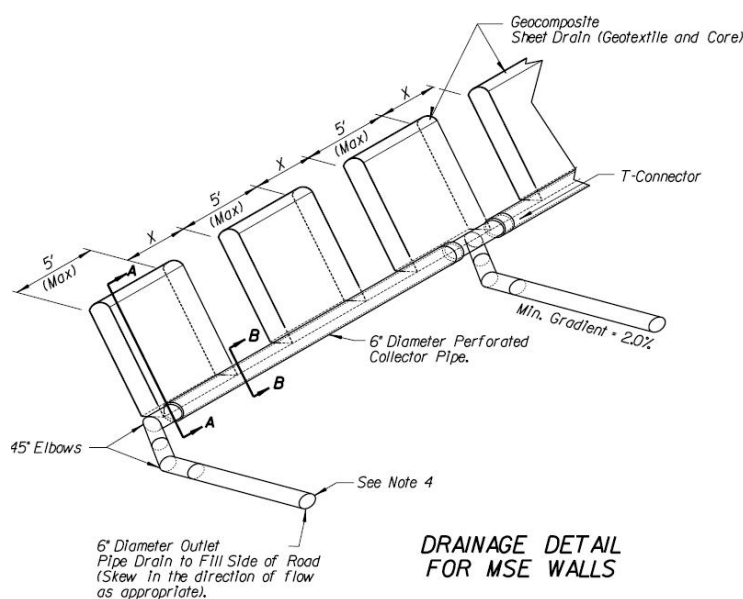
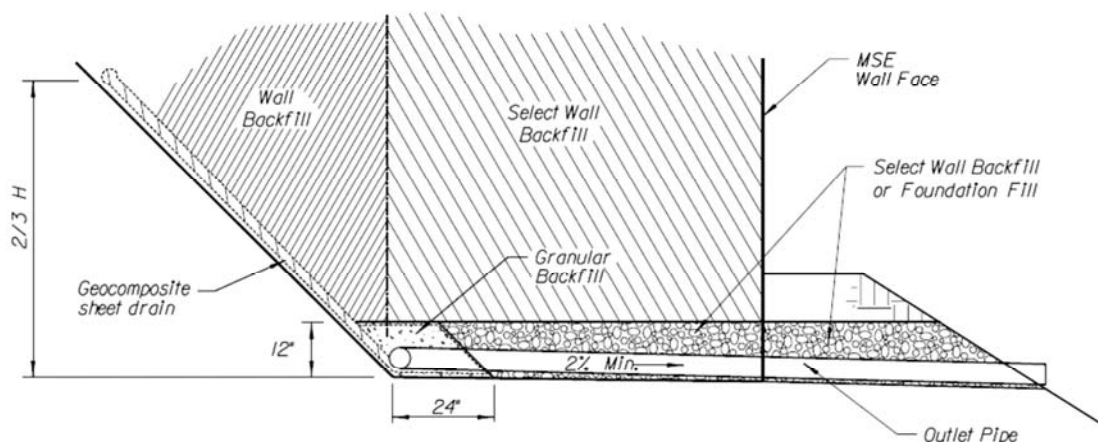


Figure 5-9. Example drainage detail using a geocomposite sheet drain.

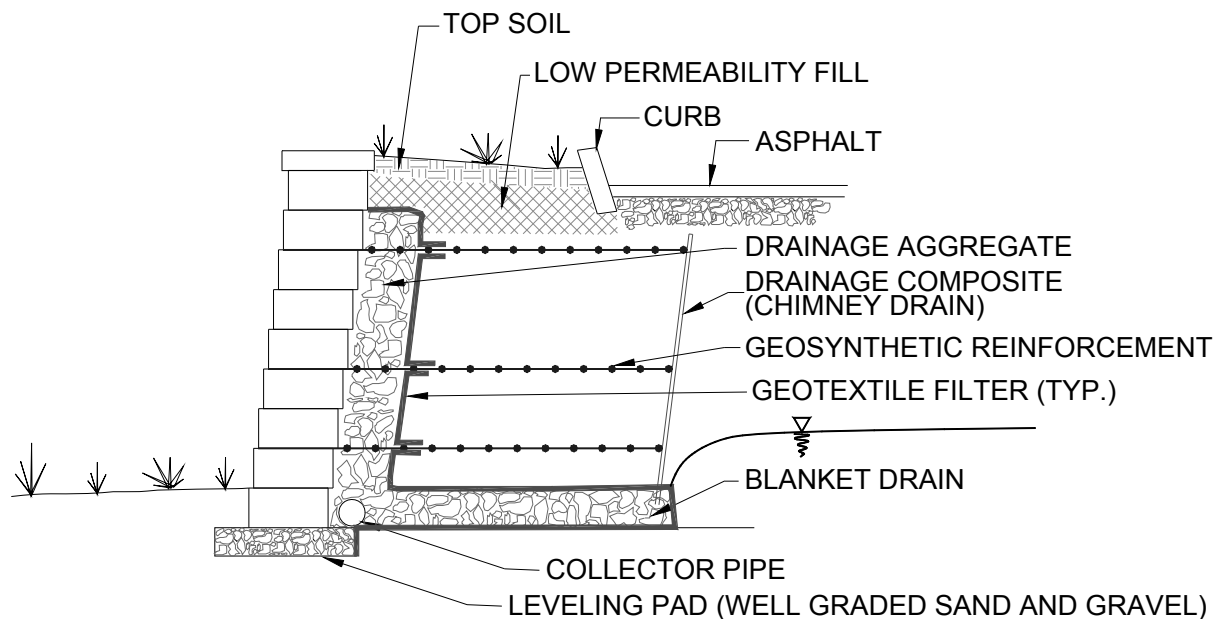


Figure 5-10. Example drainage detail using a blanket drain with chimney drain (Collin et al., 2002).

Walls with Possibility of Inundation

For walls potentially subject to inundation, such as those located adjacent to rivers, canals, detention basins or retention basins, a minimum hydrostatic pressure equal to 3 ft (1 m) should be applied at the high-water level for the design flood event. Effective unit weights should be used in the calculations for internal and external stability beginning at levels just below the equivalent surface of the pressure head line. Where the wall is influenced by water fluctuations, the wall should be designed for rapid drawdown conditions which could result in differential hydrostatic pressure greater than 3 ft (1 m). As an alternative to designing for rapid drawdown conditions, No. 57 coarse aggregate, as specified in AASHTO M 43, could be provided as reinforced backfill for the full reinforced zone of the wall and to the maximum height of submergence of the wall. A geotextile filter should be provided at the interface of the No. 57 coarse aggregate and reinforced backfill above it, at the interface of the retained backfill behind it, and at the interface of the coarse gravel and subgrade beneath it, unless the coarse aggregate meets the soil filtration criteria for the adjacent soils (see Section 5.3.3). The geotextile should meet the filtration and survivability criteria in Section 5.3.3. Adjoining sections of geotextile filter/separator shall be overlapped by a minimum of 1 ft (0.3 m). An example detail is shown in Figure 5-11.

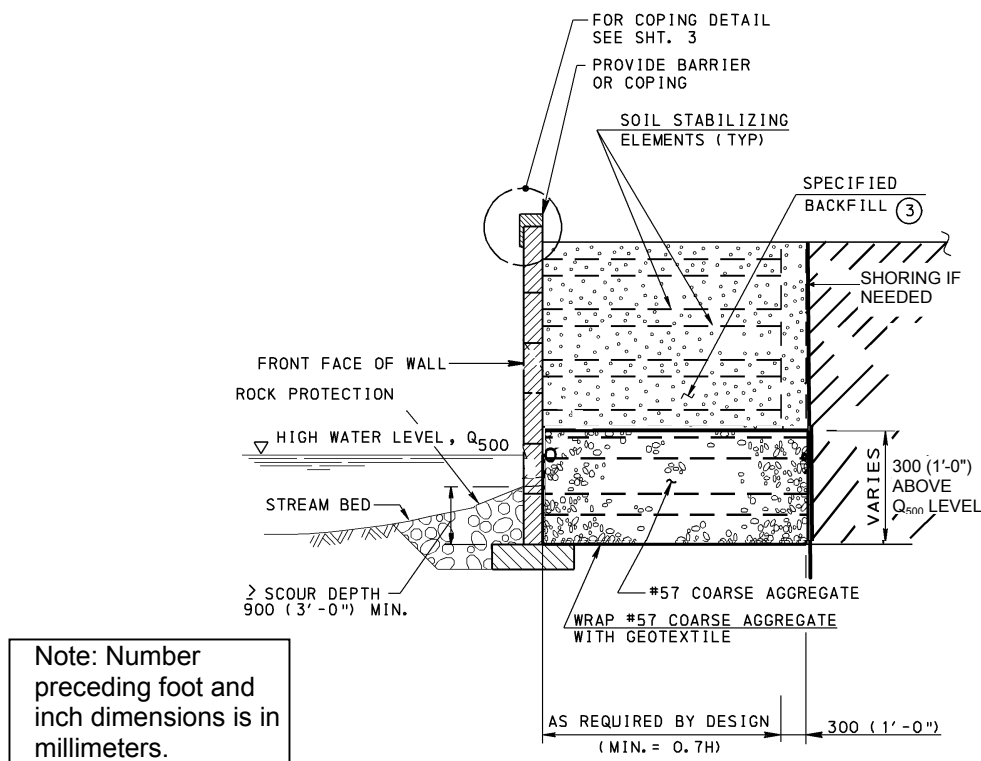


Figure 5-11. Example detail for wall that may experience inundation.

5.3.2 External Drainage

Surface drainage is an important aspect of ensuring MSE wall performance and must be addressed during design. Appropriate measures to prevent surface water from infiltrating into the wall backfill should be included in the design of all MSE walls. This typically requires coordination with designers of other project elements.

During construction of an MSE wall, the Contractor should grade the wall fill surface away from the wall face at the end of each day of construction to prevent water from ponding behind the wall and saturating the soil. In addition to softening the subgrade, surface water running onto a partially completed wall fill can carry fine-grained soils into the backfill work area and locally contaminate a free-draining granular backfill with fines. If finer grained backfill is being utilized for the reinforced wall fill, saturation can cause movements of the partially constructed wall.

When possible, finished grading at the top of a wall structure should provide positive drainage away from the wall to prevent or minimize infiltration of surface water into the reinforced wall fill. If the area above the wall is paved, a curb and gutter is typically used to direct the flow away from the wall. Drainage swales lined with concrete or asphalt can be used to collect and discharge surface water. Vegetation lined swales may be used where a vegetated finished grade slopes to the wall. Water runoff over the top of a wall where the backfill slopes towards it can lead to erosion behind the top of the wall and undermining of the wall. Such runoff can also cause staining of the wall face as soil is carried with the water. Construction of a collection swale close to the wall will help to prevent runoff from going over the top of the wall. Runoff flow will concentrate at grading low points behind the face and cause ponding which leads to undesirable infiltration of water into the backfill and increased compressibility due to softening of the backfill.

Collection and conveyance swales should prevent overtopping of the wall for the design storm event. Extreme events such as heavy rainfalls of short duration have been known to cause substantial damage to earth retaining structures due to erosion and undermining, flooding, and/or increased hydrostatic pressures both during and after construction. This is particularly true for sites where surface drainage flows toward the wall structure and where finer-grained backfills are used.

If the surface grading is such that there is likelihood of surface water flowing towards an MSE structure, then the water should be collected in a gutter or other collection feature that is part of the site drainage features. Such site drainage features are designed for an assumed or prescribed design storm event. For MSE walls, the design storm event should be based on a minimum 100 year event. However, extreme events can occur that result in short duration flows, e.g., 1 to 3 hours, which significantly exceed the design capacity of the stormwater management system. When such events occur, site flooding can cause overtopping of the wall, erosion and undermining, and an increase in hydrostatic forces within and behind the reinforced soil mass. Therefore, the site layout and wall structure should include features for handling flows greater than the design event as is typically done in the design of an overflow spillway for a dam. The project civil engineer should address potential excess flows and coordinate work with the wall designer. Consideration should be given to incorporating details of overflow features, such as a spillway, into the wall design for sites where surface water flows towards the wall structure. An example of an overflow feature is shown in Figure 5-12. Maintenance issues included in Section 5.3.4 should be addressed to ensure that all site drainage features are performing adequately.



Figure 5-12. Example MSE wall overflow sill at top of wall.

Drainage Swale at Top of Wall

A drainage swale is a man-made depression in the ground surface used to intercept surface water and direct it in a controlled manner to an outlet. Drainage swale can also be used to reduce the potential for surface water from overtopping the wall. Figure 5-13 shows typical drainage swale details for segmental precast concrete facing and modular block wall facing units. When a drainage swale is used, the project civil engineer and the wall designer should address and detail the outlet(s) for the swale. For example, the swale can be detailed to discharge water at the end of the wall structure or to low overflow points along the wall length. Overflow points should be detailed on the construction drawings. The designer should anticipate and address in design and detailing the possibility of water runoff from extreme events which will overtop the drainage swale and run down the wall face, unless the swales are specifically sized for such events. For sloping backfills, the wall designer should also address collection and diversion of water at the top of the slope. Site water runoff from above the backslope should not be directed toward the MSE wall backslope.

Vegetated swales as shown in Figure 5-13b can provide an aesthetically pleasing appearance. However, the effectiveness of the low permeability soil in preventing water from migrating into the reinforced soil mass and drainage aggregate should be evaluated. Shrinkage cracks in the low permeability soil during periods of extended dry weather may increase the permeability of the layer to the extent that it is no longer an effective barrier layer. Therefore, a geomembrane should be used beneath any vegetated swale.

Geomembrane Barriers

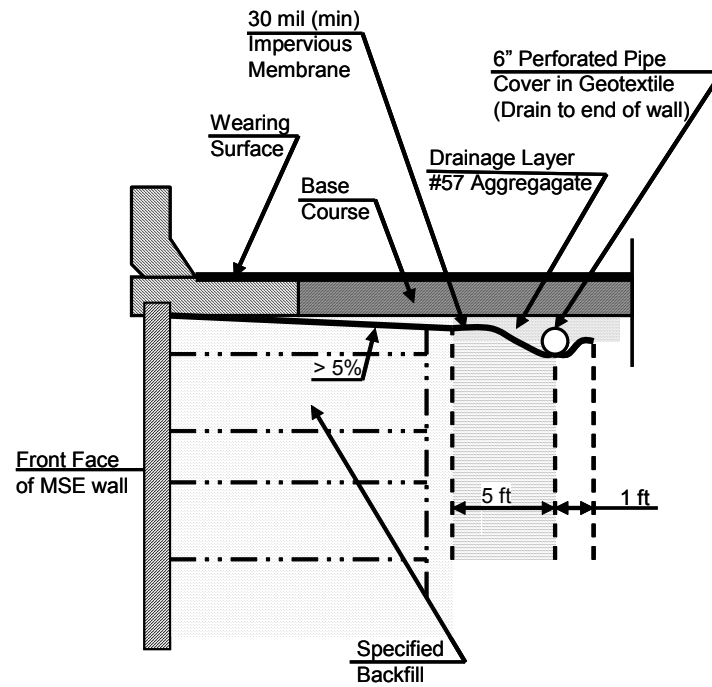
A geomembrane barrier can be used to prevent surface water infiltration and associated seepage forces that can occur when using poorly draining reinforced fill. In addition steel soil reinforcements in the upper portion of MSE walls exposed to runoff containing deicing salts are affected by higher corrosion rates than defined by current corrosion rate models. Therefore, a geomembrane barrier should be placed below the road base and just above the first layer of soil reinforcement. The geomembrane should be tied into a drainage system to collect and discharge the runoff. As per Article 11.10.8 of AASHTO (2007), a roughened surface PVC, HDPE or LLDPE geomembrane with a minimum thickness of 30 mils (0.75 mm) should be used. All seams in the membrane should be glued or welded to prevent leakage.

An example detail for use of geomembrane barrier to prevent infiltration of runoff into the reinforced soil mass is illustrated in Figure 5-14a. As shown in Figure 5-14a, the geomembrane should be sloped to drain away from the facing to an intercepting longitudinal drain outletted beyond the reinforced mass. Installation of a geomembrane infiltration barrier is shown in Figure 5-14b and 5-14c. Design requirements for the geomembrane are covered in Section 5.3.3.

Pavement Permeability and Runoff

Pavements are porous structures. Surface water flows through asphalt pavement cracks and concrete joints and cracks into the pavement base material(s). The flow into the base aggregates can be significant, with up to 50% of the water falling on the pavement finding its way to the base course, and much more if there are cracks in the pavement, e.g., upwards of 97% will flow through a 1/8 in. (3 mm) crack according to AASHTO (1986). This water then saturates the subgrade because the relatively high permeability base aggregate ponds the water above the MSE wall. The situation is compounded if the site and pavement grades toward a low spot as shown in Figure 5-15. The MSE wall designer should interact with the project civil engineer to ensure that such a condition is mitigated and positive drainage measures are provided to capture the pavement drainage in the form of proper grading away from the wall and edge drains. Consideration should also be given to using the geomembrane detail shown in Figure 5-14, to intercept and discharge the water seeping through cracks in the pavement.

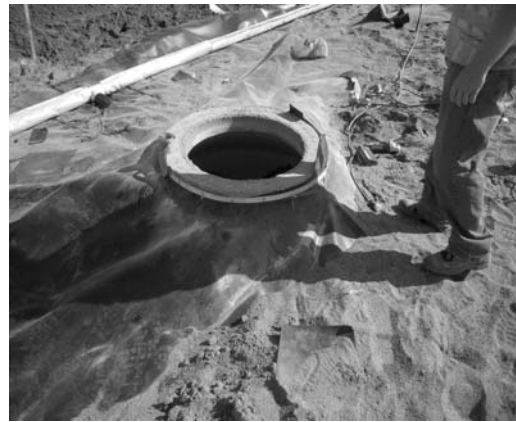
Surface runoff on the pavements that overtops the wall can cause undermining of the wall. Sloping the roadway towards a ditch is a common way to guard against wall overtopping. This is also sometimes referred to as roadway "in sloping."



(a)



(b)



(c)

Figure 5-14. (a) Example geomembrane barrier details, (b) Installation of geomembrane deicing salt runoff barrier, (c) Geomembrane installation around manhole penetration.

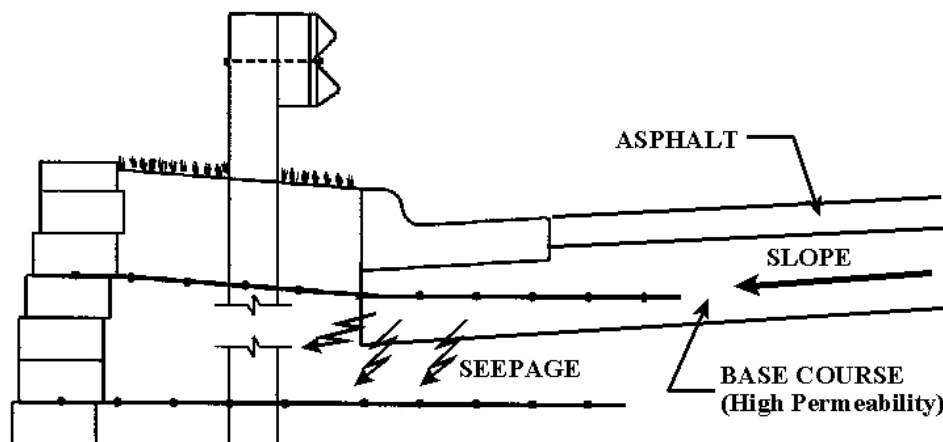


Figure 5-15. Example of undesirable water seepage through pavement due to deficient grades (Collin et al., 2002)

Grade at Toe and Ends of the Wall

The final grade at the toe and ends of the wall, both as designed and as constructed, is an important consideration for water flow conditions. Surface water flow along the toe of a MSE wall may occur around the ends or along the face of the structure and has the potential to erode the soil. An example of water damage is shown in Figure 5-16. Erosion of soil at the toe of a wall eventually may undermine the MSE wall facing units. Thus, design and construction details normally should direct flow away from the toe of wall structures. This can be accomplished with site grading and with a soil berm or slope at the toe of the wall.

Erosion control details are required where water will flow adjacent to the wall toe. Geotextile lined riprap stone or other means should be used to prevent scour. The designer also may elect to embed the wall deeper (i.e., lower foundation level) where the potential for erosion of the wall toe exists. Consideration should be given to turning the wall 90 degrees inward from the face.

The ends of the wall that terminate in or intercept embankment slopes should also be protected from erosion. Walls that terminate in slopes should be adequately keyed into the slope and a swale used to divert water away from the ends of the wall to mitigate erosion. Wing walls for approach fills should also be design such that water does not flow down the slope along the back of the wall face. Again a swale can be used to divert water and the surface of the slope should be graded to promote water flows away from the wall.



Figure 5-16. Example of surface flow erosion at the bottom of an MSE wall.

5.3.3 Filtration and Drainage System Component Requirements

Construction of an MSE wall may involve several types of soils. Groundwater flow from one soil type to another, and then to a drain and outlet feature, should be unimpeded. Soil filtration and permeability requirements must be met between adjacent zones of different soils to prevent impeded flow or piping. Adjacent soils of interest in an MSE wall system are as follows:

- the reinforced fill and any drainage layers,
- the reinforced fill, facing elements such as joints and/or face drainage aggregate and geotextile covering the joints,
- the reinforced fill and retained fill,
- the reinforced fill and foundation soil, and
- the reinforced fill and embankment fill above the wall and low permeable surface fill that may be used to reduce infiltration.

Filters may be in the form of a graded granular soil or a geotextile. Design of both soil filters and geotextile filters are discussed below. Design of geocomposite drains, drainage inflow and outflow, drain collection and outlet pipes and geomembrane barriers are also discussed.

Soil Filters

As water flows from one soil zone to another, the downstream soil must meet filter criteria to prevent piping of the upstream soil. Furthermore, the downstream soil must have adequate permeability relative to the adjacent, upstream soil. Therefore, the downstream soil must have the correct gradation range to function properly as a filter. The gradation requirements of the filter are also a function of the upstream soil gradation because the design flow capacity of the filter cannot be realized if the upstream soil pipes into the downstream soil. The pore sizes in the filter soil must be small enough to retain the larger size particles of the soil, which in turn retain the smaller sizes of the retained soil. The filter pore size is mathematically a function of its controlling particle size.

Design criteria for soil filters are summarized below and are based upon gradations of two adjacent soils. The particle sizes used in design are the D_{15} , D_{50} , and D_{85} sizes (subscript denotes the percentage of material, by weight, which has a smaller diameter). These criteria are applicable to adjacent soils with gradation curves that are approximately parallel. The equations are not applicable to gap-graded soils, soil-rock mixtures, non steady-state flow and soils with gradation curves that are not approximately parallel. When criteria are not applicable, filter design should be based upon laboratory filtration tests. The reader is referred to Cedergren (1989) for a comprehensive discussion on soil filtration.

The soil filtration criterion to prevent piping (i.e., retention) of the upstream soil into the filter is:

$$\frac{D_{15(\text{filter})}}{D_{85(\text{soil})}} < 5 \quad (5-1)$$

To ensure sufficient permeability of the filter material, the ratio of the filter D_{15} to the upstream soil D_{15} should be as shown in Equation 5-2.

$$4 < \frac{D_{15(\text{filter})}}{D_{15(\text{soil})}} < 20 \quad (5-2)$$

An additional criterion to prevent movement of soil particles into or through filters is presented in Equation 5-3. For CL and CH soils without sand or silt particles, the D_{15} size of the filter in Equation 5-2 may be as great as 0.016 in, and Equation 5-3 may be disregarded. However, if the upstream soil, i.e., retained fill or backcut soils, contains particles of uniform non-plastic fine sand and silt sizes, the filter must be designed to meet these criteria.

$$\frac{D_{50(\text{filter})}}{D_{50(\text{soil})}} < 25 \quad (5-3)$$

Geotextile Filters

A geotextile is often used as a filter between a finer-grained and a more permeable soil. The geotextile must retain the finer-grained soil, while allowing water to readily pass into the more permeable soil, and function throughout the life of the earth retaining structure. Thus, geotextile design must address retention, permeability, and clogging. The geotextile must also survive the installation process.

The following design steps are from the FHWA Geosynthetic Design and Construction Guidelines Manual (Holtz et al. 2008).

Step 1. Determine the gradation of the material to be separated/filtered. The filtered material is directly upstream or downstream of the geotextile filter for the drainage layer. Determine D_{85} , D_{15} , $C_u = D_{60}/D_{10}$ and the percent passing a No. 200 (0.075 mm) sieve. When the soil contains particles 1 in. (25 mm) and larger, use only the gradation of soil passing the No.4 (4.75 mm) sieve in selecting the geotextile (*i.e.*, scalp off the + No.4 (+4.75 mm) material).

Step 2. Determine the permeability of the upstream or downstream material to be filtered. These include the reinforced fill, foundation soil, retained fill and the natural soil in cut slope.

Step 3. Apply design criteria for retention, permeability and clogging resistance to determine apparent open size (AOS), permeability (k), and permittivity (ψ) requirements for the geotextile (after Holtz et al., 2008). AOS, k and ψ of the candidate geotextile are determined from standard ASTM tests and is typically the value published by the geotextile manufacturers/suppliers. Use only needlepunched nonwoven or monofilament woven geotextiles (*i.e.*, slit film woven geotextiles shall not be used).

A. Retention Criteria – Steady State Flow

Using the D_{85} and C_u values from Step 1, determine the largest allowable pore size as follows:

$$\text{AOS} \leq B D_{85} \quad (5-4)$$

where:

AOS = apparent opening size of the geotextile

- B = dimensionless coefficient
 D_{85} = soil particle size for which 85% are smaller

The AOS value of the candidate geotextile is determined from the results of the ASTM D4751 test method, and is typically the value published by the geotextile manufacturers/suppliers. The B coefficient ranges from 0.5 to 2 and is a function of the upstream finer-grained soil, type of geotextile, and/or the flow conditions. For sands, gravelly sands, silty sands and clayey sands (i.e., sands with less than 50% passing the No. 200 sieve), B is a function of the uniformity coefficient, C_u ($C_u = D_{60}/D_{10}$), of the upstream soil. Table 5-1 presents values of B for various values of C_u .

If the upstream soil contains any fines, only the portion passing the No. 200 sieve should be used for selecting the geotextile. For silts and clays (more than 50% passing the No. 200 sieve), B is a function of the type of geotextile as given in Table 5-2.

These retention criteria are for internally stable soils and steady-state seepage conditions. Laboratory performance tests should be conducted for unstable soils. For soils with a $C_u > 20$, unsteady seepage may occur. For dynamic and cyclic flow condition use $AOS < 0.5D_{85}$. See Holtz et al. (2008) for further information on dynamic flow conditions such as wave action.

**Table 5-1. Values of B for Various C_u Values
for Soils with Less than 50% Passing the No. 200 Sieve.**

C_u	B
$C_u \leq 2$	1
$2 \leq C_u \leq 4$	$0.5 C_u$
$4 < C_u < 8$	$8 / C_u$
$C_u \geq 8$	1

**Table 5-2. Values of B and AOS for Soils with
More than 50% Passing the No. 200 Sieve Based on Type of Geotextile.**

Type of Geotextile	B	AOS
Woven monofilament	$B = 1$	$AOS \leq D_{85}$
Nonwoven	$B = 1.8$	$AOS \leq 1.8D_{85}$
Both woven and nonwoven	-	$AOS \leq 0.012 \text{ in. (0.3 mm)}$

B. Permeability/Permittivity Criteria

For steady-state flow, low hydraulic gradient and well graded or uniform upstream soil, the permeability and permittivity criteria are:

- For permeability $k_{\text{geotextile}} \geq k_{\text{soil}}$ (Less Critical / Less Severe) (5-5a)
 $k_{\text{geotextile}} \geq 10 k_{\text{soil}}$ (Critical / Severe)

- For permittivity $\Psi \geq 0.5 \text{ sec}^{-1}$ for < 15% passing No. 200 sieve (5-5b)

- $\Psi \geq 0.2 \text{ sec}^{-1}$ for 15% to 50% passing No. 200 sieve (5-5c)

- $\Psi \geq 0.1 \text{ sec}^{-1}$ for > 50% passing No. 200 sieve (5-5d)

where:

k = coefficient of permeability (or hydraulic conductivity) and

Ψ = geotextile permittivity, which is equal to $k_{\text{geotextile}}/t_{\text{geotextile}}$.

Critical or severe applications are described in Holtz et al. (2008) and, as indicated in Equation 5-5a, a geotextile permeability of 10 times the soil permeability should be used. The geotextile permittivity is determined from the results of the ASTM D4491 test method.

C. Clogging Criteria

a. For steady state flow, low hydraulic gradient and well graded or uniform upstream soil, the clogging criterion is:

$$\text{AOS} \geq 3 D_{15(\text{upstream soil})} \quad (5-6)$$

This equation applies to soils with $C_u > 3$. For soils with $C_u \leq 3$, a geotextile with the maximum AOS value from the retention criteria should be used.

b. Other qualifiers

Nonwoven geotextiles: Porosity (geotextile) $\geq 50\%$

Woven geotextiles: Percent open area $\geq 4\%$

c. Alternative: Run filtration tests, especially for critical and severe applications

Step 4. In order to perform effectively, the geotextile must also survive the installation process. AASHTO M288 (2006) provides the criteria for geotextile strength required to survive construction of roads, as shown in Table 5-3. Use geotextile Class 2 where a moderate level of survivability is required (e.g., for geotextile filters at the wall face and on back drains). Class 1 geotextiles are recommended when heavy construction equipment is used and/or angular fill will be placed directly above or below the geotextile (e.g., geotextile filters for base drains). A minimum of 6 in. (150 mm) of base/subbase should be maintained between the wheel and geotextile at all times.

Table 5-3. Geotextile Survivability Requirements (AASHTO M 288, 2006).

Test	Test Method	Units	Geotextile Class			
			Class 1		Class 2	
			< 50%*	≥ 50%*	< 50%*	≥ 50%*
Grab Strength	ASTM D4632	N (lb)	1400 (315)	900 (200)	1100 (250)	700 (157)
Seam Strength	ASTM D4632	N (lb)	1200 (260)	810 (180)	990 (220)	630 (140)
Tear Strength	ASTM D4533	N (lb)	500 (110)	350 (80)	400 (90)	250 (56)
Puncture Strength	ASTM D6241	N (lb)	2750 (620)	1925 (433)	2200 (495)	1375 (309)
Ultraviolet Stability (Retained Strength)	ASTM D4355	%	At face joints - 70% after 500 hours of exposure Buried in wall - 50% after 500 hours of exposure			
*Note: Elongation measured in accordance with ASTM D4632 with < 50% typical of woven geotextiles and ≥ 50% typical of nonwoven geotextiles. (1 N = 0.22 lbs, 1 kPa = 0.145 psi)						

Step 5. Collect samples of geotextile, reinforced fill and retained fill at time of construction to confirm acceptance.

Step 6. Monitor installation during construction.

Step 7. Observe effectiveness of drainage system during and after storm events.

For a more thorough treatment of geotextile drains see Holtz et al. (2008).

Geocomposite Drain

A geocomposite, or prefabricated, drain consists of a geotextile filter and a water collection and conveyance core. The cores convey the water and are generally made of plastic waffles, three-dimensional meshes or mats, extruded and fluted plastic sheets, or nets. A wide variety of geocomposites are readily available. For MSE wall design, only geocomposites that allow two-sided flow (i.e., flow into the drains from both sides) should be used. However, the filtration and flow properties, detailing requirements, and installation recommendations vary and may be poorly defined for some products. The geotextile of the geocomposite should be designed to meet filter and permeability requirements discussed previously in this section. The flow capacity of geocomposite drains can be determined by using the procedures described in ASTM D4716. Long-term compressive stresses and eccentric loadings on the geocomposite core should be considered during design and selection.

MSE walls can place a significant stress on the geocomposite. Hence, the design pressure on a geocomposite core should be limited to either of the following:

- the maximum pressure sustained on the core in a test of 10,000 hr minimum duration; or
- the crushing pressure of a core, as defined with a quick loading test, divided by a safety factor of five.

Finally, as with in drain system, consideration should be given to system performance factors such as distance between drain outlets, hydraulic gradient of the drains, potential for blockage due to small animals, freezing, etc. Other design aspects of geocomposite drains are addressed in Holtz et al. (2008).

Installation details, such as joining adjacent sections of the geocomposite and connections to outlets, are usually product-specific. Product-specific variances should be considered and addressed in the design, specification, detailing and construction phases of a project. General construction specification requirements will be review in Chapter 10. Post installation examination of the drainage core/path with a camera scope should be considered for critical applications.

Drainage Inflow and Outflow Design Requirements

For proper design of the drains at the back or base of the reinforced soil mass, the flow into the system and the flow in the drain must be evaluated. These flow conditions are discussed below and apply to either gravel or geocomposite drains. Cedergren (1989) and Huntington (1957) present a more thorough treatment of pressures induced by the influence of ground water and seepage acting on retaining walls as well as drainage design.

Flow into the System. Anticipated flow into the drain system may be estimated using Darcy's Law. Flow is equal to:

$$q = k i A \quad (5-7)$$

where:

- q = infiltration rate
- k = effective permeability of retained backfill soil
- i = average hydraulic gradient in retained backfill soil
- A = area of soil normal to the direction of flow

Conventional flow net analysis can be used to calculate the hydraulic gradient.

Some drains consist of drainage aggregate surrounding a perforated pipe with a filter (usually a geotextile) surrounding the drainage aggregate. Flow into the drainage aggregate may be calculated with Equation 5-7. Flow from the drainage aggregate into the pipe is through the circular or slot perforations. Perforated, corrugated HDPE pipe is manufactured with minimum inlet openings of approximately 1 square inch per 1 foot length (20 cm² per meter length) for standard pipe (AASHTO M252, 2006). Standard pipe is generally adequate for most subsurface drainage applications. Hole diameter or slot width must be checked relative to the size of the surrounding drainage aggregate, to ensure soil retention. For slots, Equation 5-8 may be used to check retention.

$$\frac{D_{85(\text{drain fill})}}{\text{Slot Width}} > 1.2 \text{ to } 1.4 \quad (5-8)$$

For circular perforations, Equation 5-9 may be used to check retention.

$$\frac{D_{85(\text{drain fill})}}{\text{Hole Diameter}} > 1.0 \quad (5-9)$$

Flow Capacity of the Drain. Flow capacity within aggregate drains can be estimated with Equation 5-7, using k and i for the soil drain material. Flow capacity within geocomposite drains is expressed in term of unit width using the following form of Darcy's Law.

$$q = \lambda i B \quad (5-10)$$

where:

- q = flow rate
- λ = transmissivity of geocomposite drain
- i = hydraulic gradient in drain
- B = width of geocomposite drain

The geocomposite transmissivity should be evaluated with an appropriate laboratory model test. Product long-term transmissivity should be quantified at anticipated (or greater) design pressure and over time to evaluate potential decrease of flow capacity due to creep (i.e., creep of geotextile into flow channel).

Flow Capacity of the Drain Pipe. Flow capacity within drain pipes, flowing full, can be computed with the Manning's equation. Flow is equal to:

$$q = \frac{0.463}{n} d^{8/3} s^{1/2} \quad (5-11)$$

where:

- q = flow rate (cfs)
- n = roughness coefficient, or Manning's value
- d = diameter of pipe (feet)
- s = slope of energy grade line (ft per ft)

Drain Collection and Outlet Pipes

Collection and outlet pipes are often used with the drain directly behind the facing units and with the drain at the back of the reinforced soil mass. Examples of such drains are shown in Figures 5-8 and 5-10. Pipes are generally laid at required slopes, with a minimum of 2% for constructability and to ensure positive flow. Outlets are generally spaced based on the flow capacity of the pipes or alternatively at 20 ft (6 m) to 50 ft (15 m) maximum lateral spacing, and protected as noted in a later discussion on maintenance. The outlet pipes should be solid and gravity flow (e.g., 2% minimum grade) to daylight or the storm drain system.

Geomembrane Barriers

Design and specification of a geomembrane as a deicing salt barrier must address installation requirements. A geomembrane must be capable of withstanding the rigors of installation to ensure the integrity of the barrier. The subgrade material, subgrade preparation, geomembrane placement method, overlying soil fill type, and placement and compaction of overlying fill soil all affect the geosynthetic barrier's survivability. Recommended properties of geomembrane barriers (Koerner, 1998) are presented in Table 5-4. A minimum thickness of 30 mils (0.75 mm) is recommended for geomembranes above MSE walls.

Three areas of construction which are critical to a successful installation are:

- subgrade preparation;
- handling/installation including field seaming; and
- sealing around penetrations and adjacent structures.

The subgrade must provide support to the geosynthetic barrier and minimal point loadings. The subgrade must be well-compacted and devoid of large stones, sharp stones, grade stakes, etc., that could puncture the geosynthetic barrier. In general, no objects greater than ½ in. (12 mm) should be protruding above the prepared subgrade (Daniel and Koerner, 1993).

Handling and installation specifications for geomembrane and other geosynthetic barriers should, as a minimum, conform to the manufacturer's recommendations. All seams in the membrane should be glued or welded to prevent leakage. Special project requirements for geomembranes should be noted in the construction specifications and plans.

Table 5-4. Recommended Minimum Properties for General Geomembrane Installation Survivability (after Koerner, 1998).

Property and test method	Required degree of survivability	
	Medium ¹	Very high ²
Thickness, mils (mm) – ASTM D5199 or ASTM D5994 for Textured	30 (0.75)	40 (1.00)
Tear (Die C), lbf (N) - ASTM D1004	10 (45)	20 (90)
Puncture, lbf (N) - ASTM D4833	32 (140)	45 (200)
NOTES: 1. <i>Medium</i> refers to placement on machine-graded subgrade with medium loads. Soil fill should have a maximum size of ¾-inch. 2. <i>Very high</i> refers to placement on machine-graded subgrade of very poor texture. Soil fill with maximum size greater than ¾-inch.		

Geomembrane selection should also consider installation details of attachment to the MSE wall facing and details around penetrations. Construction details around penetrations and adjacent structures depend upon the chosen geosynthetic material and the project design. As such, they must be individually designed and detailed. For example, batten strips and mechanical fasteners were used with the 30 mil (0.75 mm) thick HDPE geomembrane shown in Figure 5-14c. Geosynthetic manufacturers and waste containment manuals can provide design guidance.

Another design consideration may be the frictional resistance of the geomembrane. As per Article 11.10.8 of AASHTO (2007), typically, a roughened surface PVC, HDPE or LLDPE geomembrane with a minimum thickness of 30 mils (0.75 mm) should be used. Such roughened geomembranes are readily available in the marketplace.

5.3.4 Maintenance of Drainage

Features that minimize water flow into an MSE wall and features that preserve MSE wall drainage should be maintained over the life of the structure. For example, cracks in pavement above MSE walls should be sealed. Differential settlements and pavement cracks around catch basins should be corrected to minimize potential inflow into the reinforced soil or retained soil mass. These maintenance items are for non-wall features and the wall designer may have little influence on these items. However, in interacting with designers of other project features, the need to maintain items that potentially could affect the wall should be discussed.

One of the maintenance items that the wall designer has control over is the drain outlet(s). Screens should be installed and maintained on drainpipe outlets. Screening is used to prevent small animals from nesting in and clogging the pipe. Outlet screens and cleanouts to provide access to clogged drainage should be detailed on the retaining wall construction drawings.

Additional items should be detailed when outlets are located in a soil embankment beneath the MSE walls. Drains are not effective unless the outlets are maintained, i.e., not clogged. Outlets in soil embankments should drain onto a concrete (usually precast) apron and should be marked with a permanent metal fence post. The apron and post minimize the chance of the outlet being run over and crushed by mowers or covered in subsequent construction activities. The apron and post should be detailed on the wall construction drawings.

5.4 INTERNAL DETAILS

There are a number of internal details that must be properly designed and implemented to ensure that the MSE wall system performs in an acceptable manner over its design life. This section presents common internal details

5.4.1 Contact between Dissimilar Metals

Often, several different types of metallic elements such as steel piles and drain pipes are placed in the reinforced soil mass. The metals used in such elements are different than those used in steel reinforcements. Corrosion can occur when dissimilar metals come in contact with each other due to galvanic action. Therefore, all steel soil reinforcements should be separated from other metallic elements by at least 3 in. (75 mm).

5.4.2 Vertical Obstructions in Reinforced Soil Mass

Vertical obstructions are structures that are embedded in or extend vertically through the reinforced soil mass. Examples of vertical obstructions are a catch basin, grate inlet, sign foundation, bridge foundation, light poles, guardrail post, or culvert. **Under no circumstances, should any reinforcement be left unconnected to the wall face or arbitrarily cut/bent in the field to avoid the obstruction.** A review of any modification to the design to avoid an obstruction must be made and approved by the wall designer of record. Additional consideration must be given to obstructions that exert a load on the wall (e.g., deep foundations, overturning of signs and light poles). Such applications may require additional reinforcement and facing support to resist the local increase in lateral stress. Likewise, the wall may exert lateral earth pressure or vertical downdrag stress on the obstruction due to movement of the wall, the consequences of which to the obstruction design and performance must also be evaluated.

The best design is to adjust the location of the obstruction and/or the soil reinforcement so that there is no interference. In some cases, where interference between the vertical obstruction and the soil reinforcement is unavoidable, the design of the wall near the obstruction may be modified using one of the following alternatives.

Alternative 1 - Fit the soil reinforcement around the obstruction without cutting the soil reinforcement as shown in Figure 5-17. In this alternative, the facing units near the obstruction are fitted with extra facing connections such that soil reinforcing can be connected at locations away from the vertical obstruction. For example, as shown in Figure 5-17a, a 5 ft (1.5 m) panel that needs a 4-wire bar mat may be fitted with 8 clevis loop

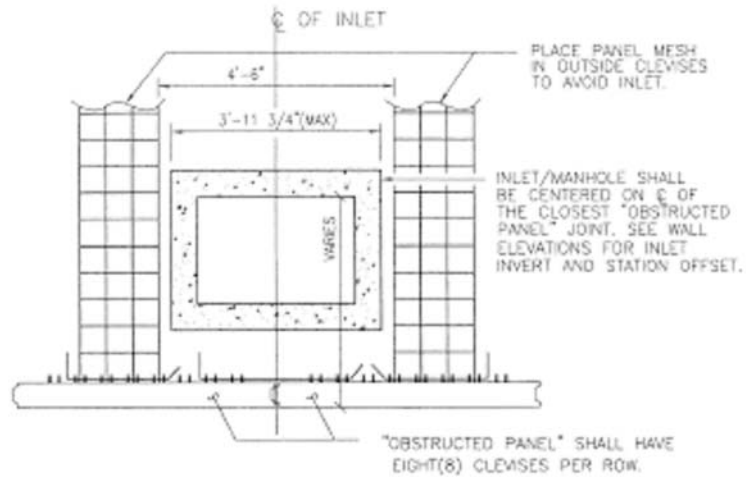
connections and in the field the bar mat can be attached to any 4 consecutive clevis loops depending on the location of the vertical obstruction. Similarly, as shown in Figure 5-17b, a 5 ft (1.5 m) panel with the obstruction blocking 2 strip reinforcements, a strong back consisting of a galvanized steel angle can bridge across two panels. The strong back then allows the two displaced reinforcements to be attached to each side of the obstruction. Where soil reinforcements are not centered on the panel, eccentric load of the facing panel must be evaluated with respect to the structural capacity of the face to resist increased bending moments and the potential for face rotation.

In case of strip reinforcements with a nut and bolt connection, it may be possible to splay the reinforcements around the obstruction as shown in Figure 5-17c. In such cases, the splay angle should be less than 15-degrees and the tensile capacity of the splayed reinforcement shall be reduced by the cosine of the splay angle. The splay angle is defined as the angle measured from a line perpendicular to the wall face in a horizontal plane. Due to the clevis loop or similar connection for bar mats splaying at more than 3- to 5-degrees is not possible without introducing moments at the connection and uneven loading of the clevis loop connectors. Under no circumstances should a bar mat be cut to force longitudinal wires around the vertical obstruction because it creates damaging moments on the cross bar welds as well as at the facing connection and cracking of galvanization. Bar mats should not be splayed if the connection mechanism does not accommodate such splay without cutting of cross bars.

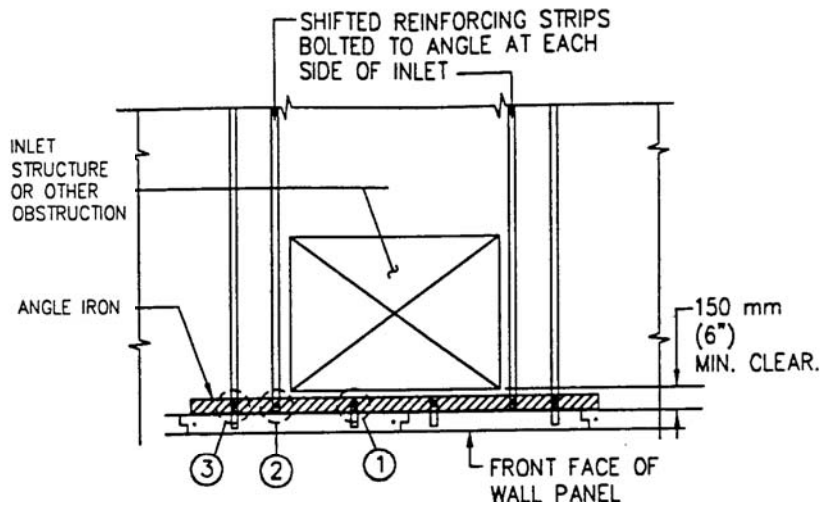
If the soil reinforcements are navigated around the vertical obstruction, then care must be taken to balance forces in the wall face to assure that the wall panels do not rotate outward. Facing units with a joint in front of the vertical obstruction may be structurally connected across the joint as shown in Figure 5-17, or a longer panel may be considered, e.g., 10 ft (3 m) instead of 5 ft (1.5 m) wide panel. The structural connection should not extend across more than one joint, i.e., 2 panels. If such a condition occurs, then Alternatives 2 and 3 should be implemented as appropriate.

Alternative 2 - Assuming that reconfiguration of soil reinforcements as per Alternative 1 is not possible and the reinforcement layers must be partially or fully severed in the location of the obstruction, the surrounding reinforcement layers should be designed to carry the additional load which would have been carried by the severed reinforcements.

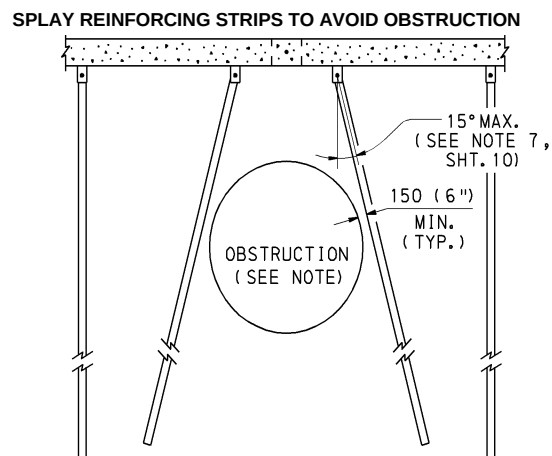
In this alternative, the portion of the wall facing in front of the obstruction should be made stable against a toppling (overturning) or sliding failure.



(a)



(b)

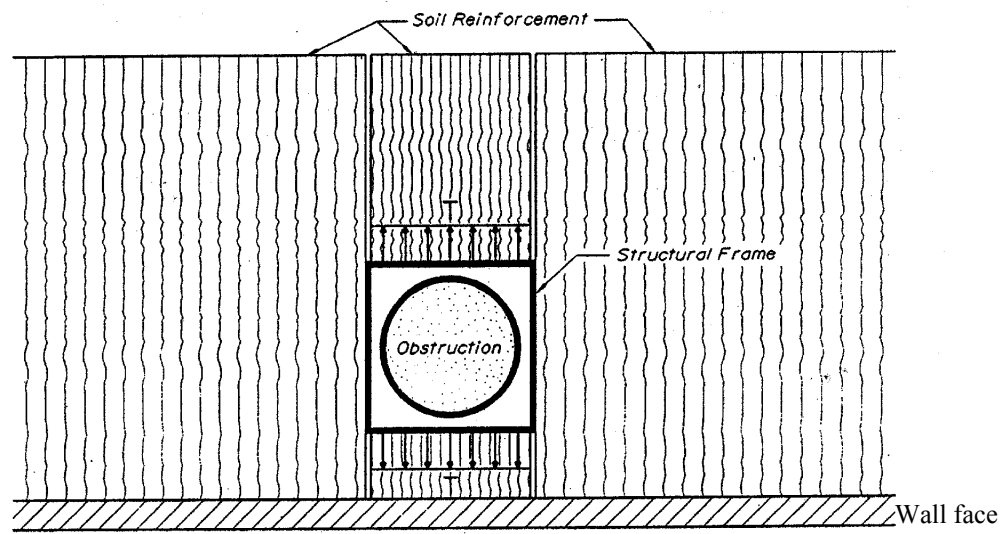


(c)

Figure 5-17. Examples of avoiding a vertical obstruction without severing soil reinforcements.

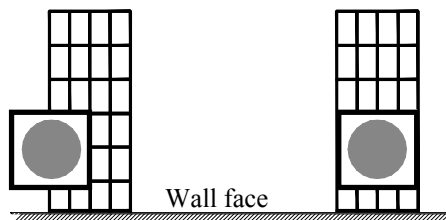
Alternative 3 - A structural frame can be placed around the obstruction which is capable of carrying the load from the facing in front of the obstruction to reinforcement connected to the structural frame behind the obstruction. This is illustrated in Figure 5-18a and 5-18b and in Figure 5-19. The structural frame and connections should be designed in accordance with Section 6 (“Steel Structures”) of AASHTO (2007) for the maximum tension at any level of reinforcements within the reinforced soil mass. The structural frame should be designed such that moments in the soil reinforcement or connection at the wall face are not generated.

Note that as shown in Figure 5-18c it may be feasible to connect the soil reinforcement directly to the obstruction depending on the reinforcement type and the nature of the obstruction. Figure 5-20 shows example details for MSE walls with modular block units with limited height vertical obstructions such as catch basin or fence post foundations near the wall face.

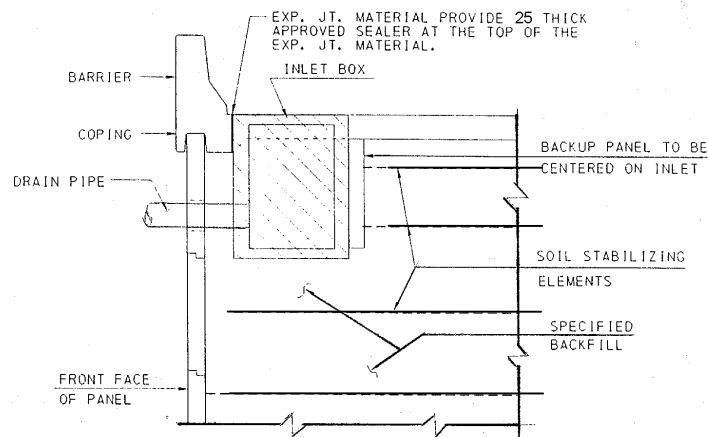


T = Total Load which Structural Frame must carry.

(a)



(b)



(c)

Figure 5-18. Vertical obstructions in reinforced soil mass with segmental precast facing units.



(a)

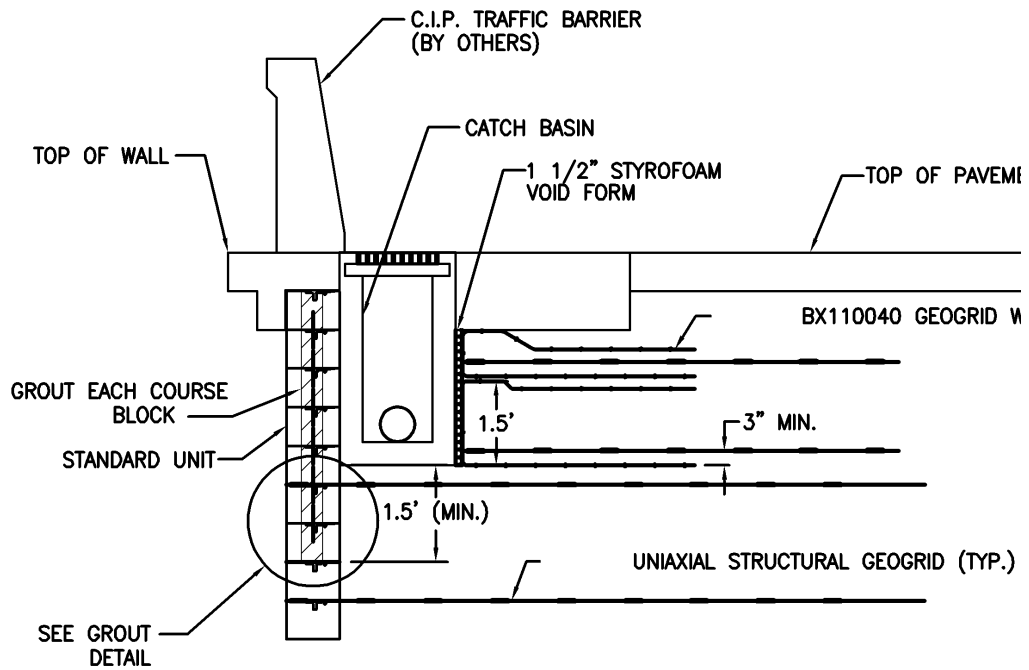


(b)

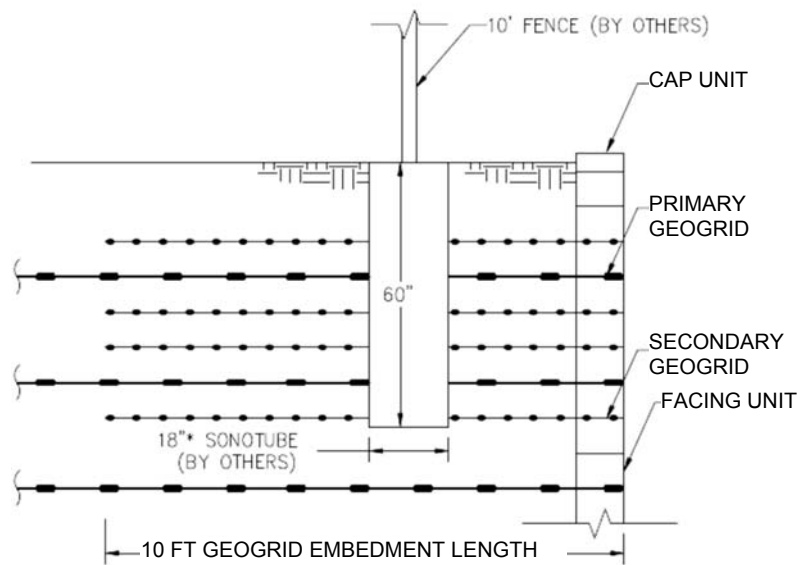


(c)

Figure 5-19. Example of a structural frame around vertical obstruction (a) with segmental precast facing - note that vertically adjacent layers of reinforcement to be separated by a minimum of 3-in. (75 mm) of wall fill, (b)-(c) with modular block face – note corner detail.



(a)



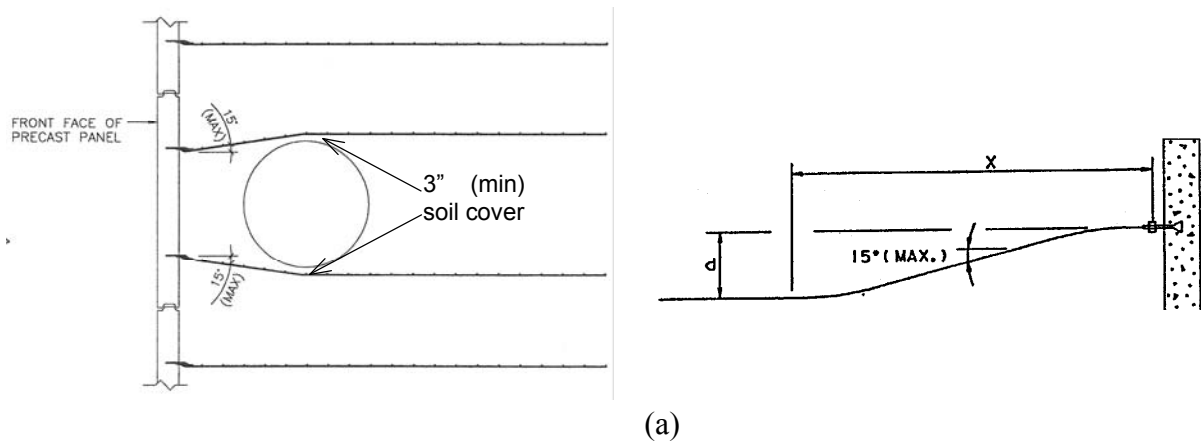
(b)

Figure 5-20. Example details of reinforcements around vertical obstructions in reinforced soil mass with modular block units.

5.4.3 Horizontal Obstructions in Reinforced Soil Mass

Horizontal obstructions are structures which are embedded in or extend horizontally through the reinforced soil mass for a substantial length along the wall. The horizontal obstructions are commonly due to utilities such as storm drain pipes. **Horizontal obstructions in reinforced soil mass should be avoided because not only do they create construction problems but obstructions such as utility pipes can be very expensive to repair and maintain as it may require dismantling the wall system.** If horizontal obstructions cannot be avoided, then some considerations for design are provided below:

- For inextensible reinforcements, the horizontal obstruction may be avoided if it is possible to deflect the reinforcement in a smooth manner up to 15 degrees of vertical skew as shown in Figure 5-21. Deflections greater than 15 degrees tend to break the galvanization and may reduce the tensile and pullout resistance of the inextensible soil reinforcements.
- Guidance for extensible reinforcements such as geogrids is presented in Figure 5-22.
- In cases where it is not possible to orient the reinforcements as shown in Figures 5-21 and 5-22, use of back-up panels may be considered as shown in Figure 5-23.
- It is not recommended to tie the reinforcements to pipes. Special details must be developed to accommodate the obstruction without attaching to it.
- Utility pipes in the reinforced mass are likely to settle differentially as the fill settles during construction. Downdrag stress should be anticipated where pipes intersect the wall face or a vertical structure such as a drop inlet. Significant leakage of water into MSE walls has been known to create wall problems including failures. Therefore, utilities should only be placed in double wall design systems such as locating utilities inside box culverts with inspection galleries or using double wall pipe with instrumentation to indicate leakage. Only leak proof joints should be used on drainage pipes. Where differential movement and downdrag stresses are anticipated, flexible connections should be used and designed to tolerate the estimated movement and stress.
- Pressurized water mains should not be constructed within an MSE structure.



Additional depth (d) required, in.	Required minimum distance (X) to achieve smooth bend, in.
3	27
6	39
9	48
12	60
15	72

Figure 5-21. Navigating horizontal obstruction in MSE walls with inextensible reinforcements.

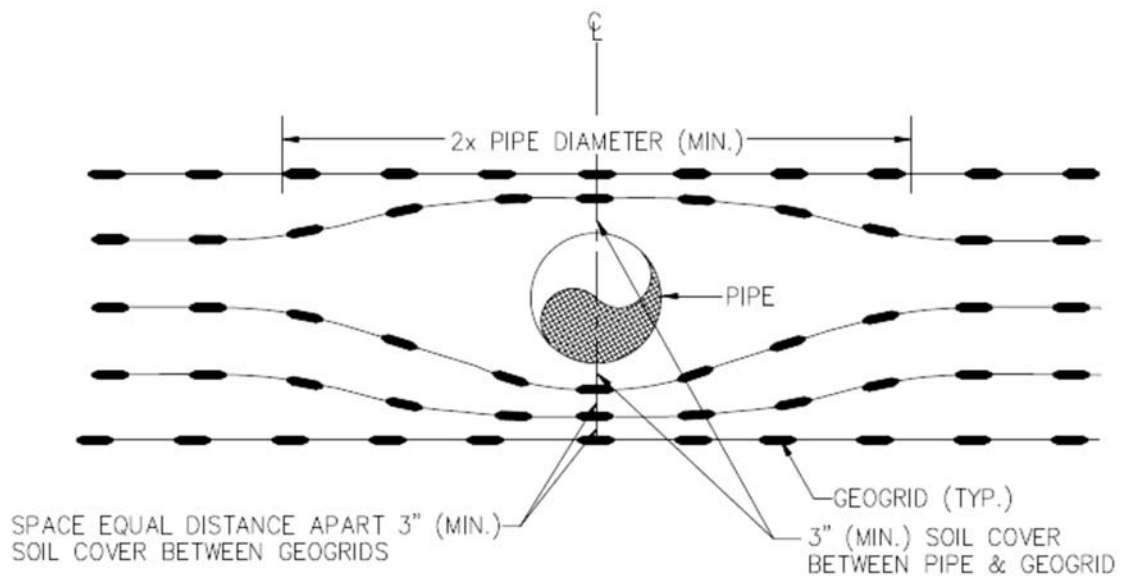


Figure 5-22. Navigating horizontal obstruction in MSE walls with extensible reinforcement.

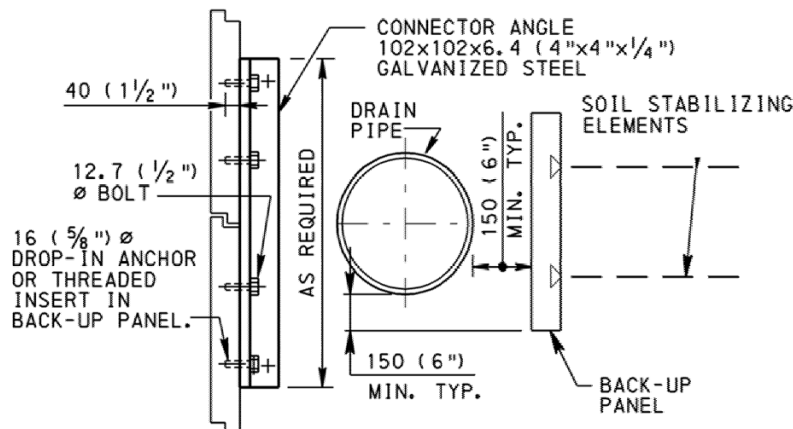


Figure 5-23. Example of backup panels for large horizontal obstructions.

5.4.4 Wall Face Penetrations

In some cases, pipes must penetrate the MSE wall or pass through the retained wall fill. Penetrations through the reinforced soil and/or wall facing units maybe at skew or perpendicular angles from the wall face.

If a pipe must penetrate through the face of the wall, the wall facing elements should be designed to fit around the pipe such that the facing elements are stable and the wall backfill an not spill through the wall face where it joins the obstruction. Differential movement between the facing and reinforced fill should be anticipated and associated downdrag stress must be consider in the design. Therefore, dry packing around the pipe should be done after the wall is substantially complete. Common details for penetrations through segmental precast concrete and modular block facing units are shown in Figures 5-24 and 5-25, respectively. Not noted on these details are the bedding and backfill for the pipe. Granular bedding may be significantly more permeable than the reinforced fill and/or the retained backfill. In these cases, the pipe bedding is a potential conduit for bringing water to the MSE wall structure. Therefore, a headwall is required to at the end of the pipe to prevent water from entering the bedding. Potential flow should be addressed in the wall details. A clear flow path, with filtration criteria addressed, from the pipe bedding and backfill to the drainage aggregate should be detailed. Weep holes through the concrete face collar may be needed to drain the pipe bedding and backfill.

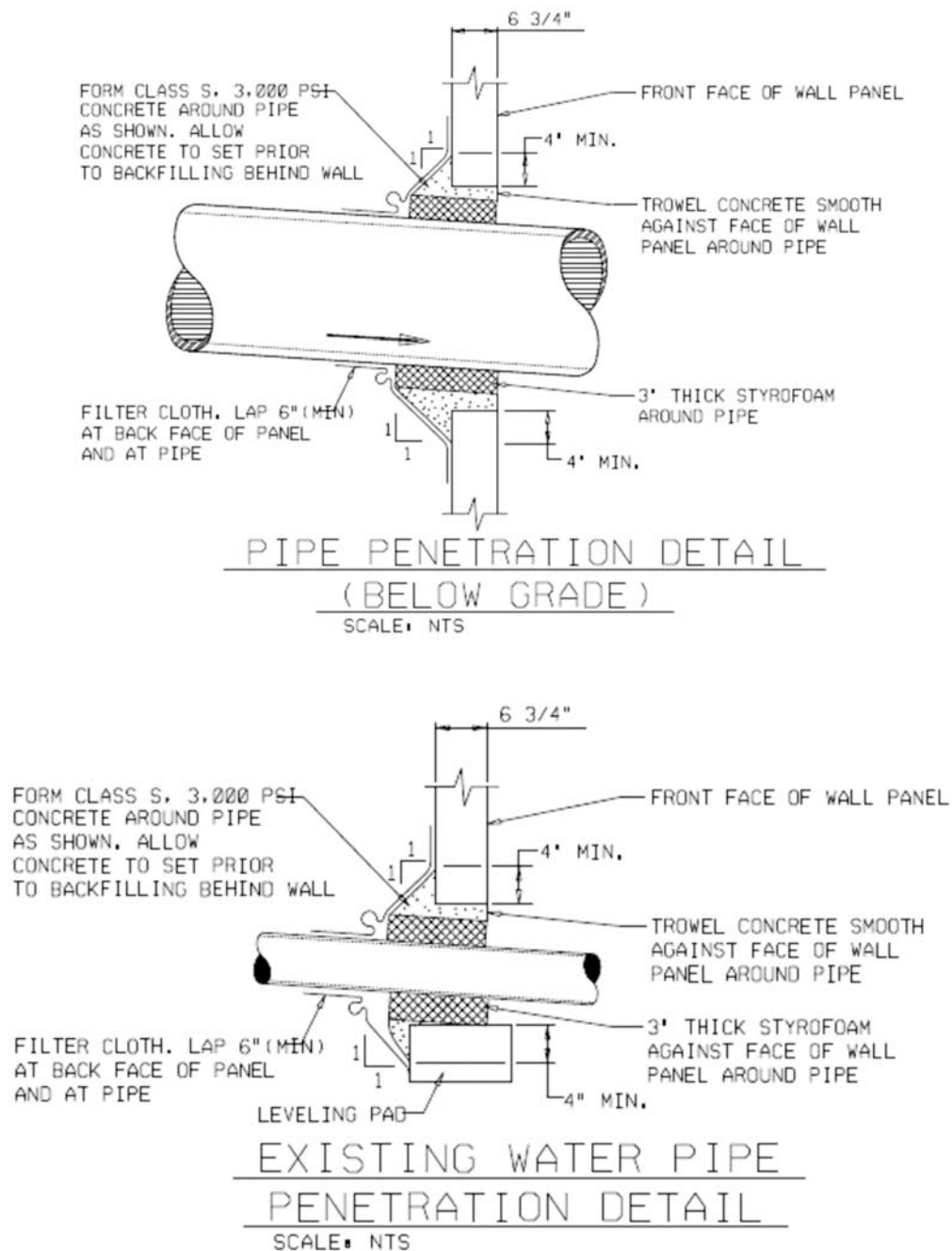


Figure 5-24. Example pipe penetrations through segmental precast panel facing units.

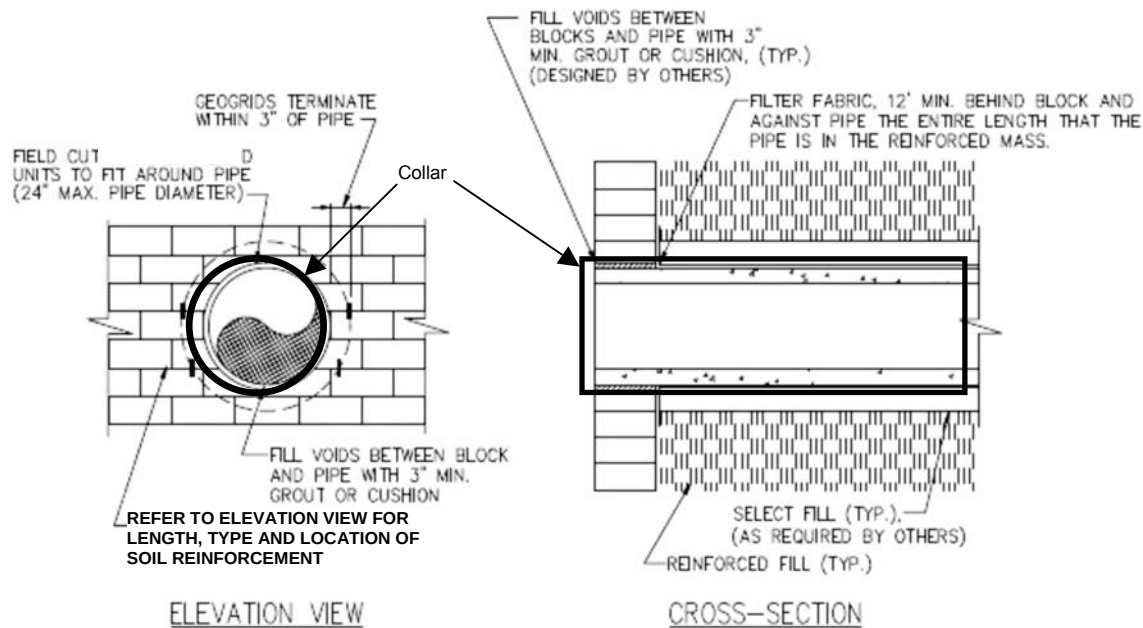


Figure 5-25. Example pipe penetration through modular block facing units.

Catch basins and manholes may penetrate vertically through the reinforced fill or retained backfill. The backfill around these manholes may be a granular soil. If the manhole backfill soils are more permeable than the wall fill soils, the manhole backfill is a potential conduit for water flow and collection. The wall designer should address this potential, as provided drainage if the surrounding wall fill soils are less permeable.

For critical wall structures, the wall designer may want to consider the possibility of leaking pipes saturating the surrounding soil. If this is a concern, a high permeability soil (relative to the wall fill) around the pipe leading to a drain or outlet may be used to provide a safety flow path.

5.4.5 Slip Joints

Where subsurface conditions and/or wall profile change abruptly, significant differential settlement may occur at the wall face with associated problems such as joint openings and facing unit to unit contact. In such cases, consideration may be given to use of slip joints which are continuous vertical joints. A slip joint is different than a regular vertical joint between panels in that there is a vertical separation between adjacent facing units that extends the full height of the wall. Due to this configuration of the joint, the wall on each side behaves independently.

Following conditions merit consideration of slip joints:

- Where abrupt differential settlement of more than 1% (or 0.01) is expected.
- Where there is an abrupt change in wall height of 5-ft or more.
- Where the wall is underlain by a relatively rigid feature such as an abutment footing or rock outcrop.
- Where a light weight rigid structure such as a box culvert intersects the face of a MSE wall.
- Where the wall terminates into a cast-in-place structure (see Section 5.5 for additional information)
- Where tight horizontal curves occur.

Figure 5-26 shows common slip joint details for segmental precast concrete facing units. As shown in the figure, the slip joint design uses either an exposed slip joint panel having its own soil reinforcement element or a hidden “backup” panel in the backfill behind the facing panel. In either case the normal connection between two panels is broken and independent movement on each side of the slip joint is possible. Figure 5-27 shows a slip joint detail for modular block facing walls.

5.4.6 Wall Curves

Curves in walls are approximated by chords that are equal to the nominal width of the facing units. Therefore, smaller wall facing units such as the modular block units are able to navigate sharp curves better than larger precast concrete facing units. Similarly, 5-ft (1.5 m) wide precast concrete facing units can navigate sharper curves than 10 ft (3 m) wide facing units. For precast concrete facing units, curves with radius as small as 50 ft (15 m) can be achieved for 5 ft (1.5 m) wide facing units with a $\frac{3}{4}$ -in. (19 mm) joint opening. For curved walls, regardless of the type of wall facing, it is critical to provide details for wall layout. The relationship of wall alignment to roadway alignment should be clearly provided. Clear dimensions need to be provided on project drawings for offsets from reference alignments and whether these offsets are relative to top of wall or bottom of wall, especially in the event of stepped foundations.

Figure 5-28 shows a typical detail for layout of geogrid reinforcement for walls with modular block facing units. Geogrid reinforcements typically require 100% area coverage whereas steel reinforcements are generally discrete and can be placed perpendicular to the wall face curves. In the case of geosynthetic reinforcements excessive overlap can result in reduced pullout resistance since contact between geosynthetics is smoother than contact between soil and geosynthetic. Therefore, a minimum soil layer of 3 in. (75 mm) between geosynthetics in the overlap zone is recommended as shown in Figure 5-28.

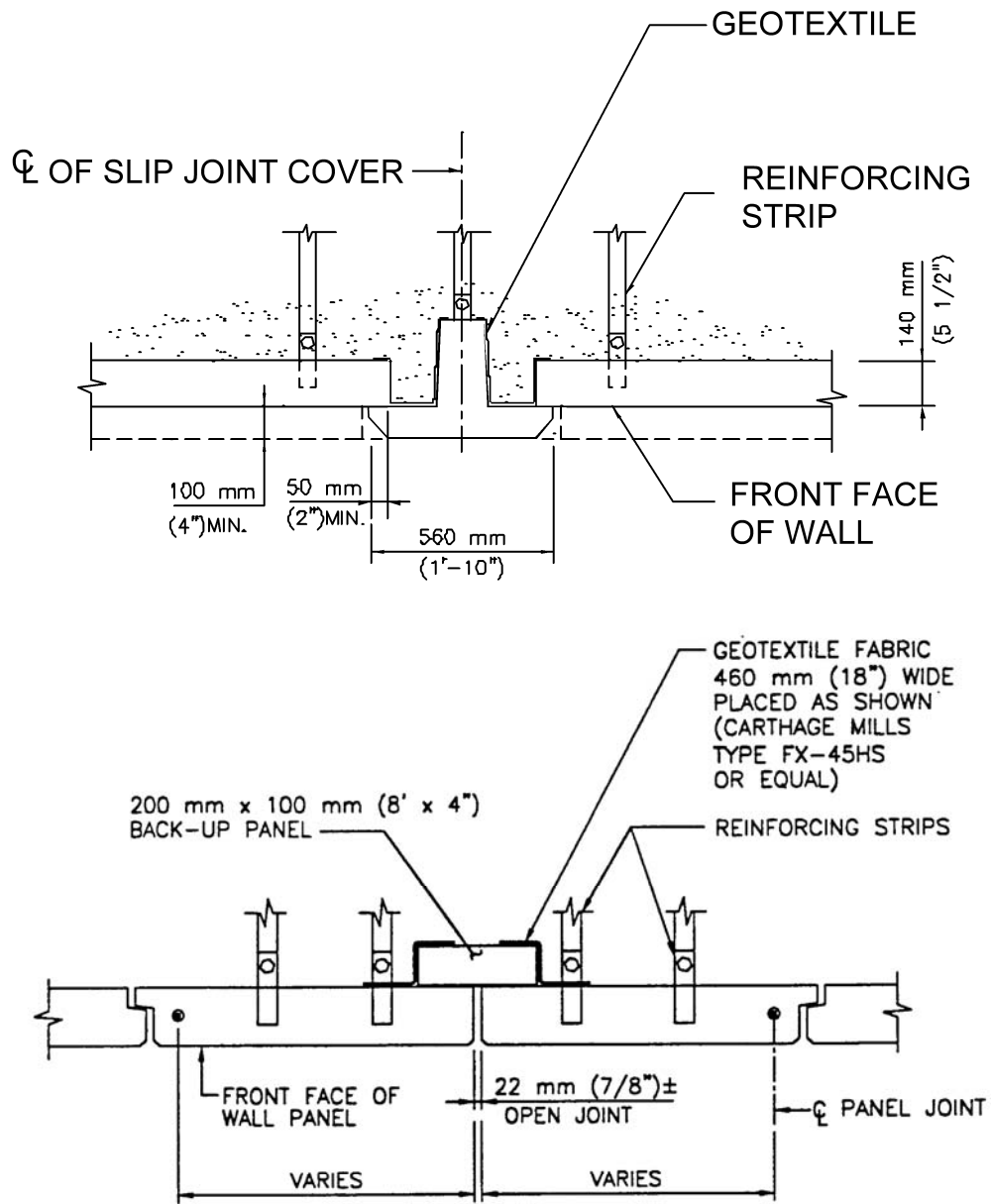


Figure 5-26. Example slip joints for segmental precast panel facings.

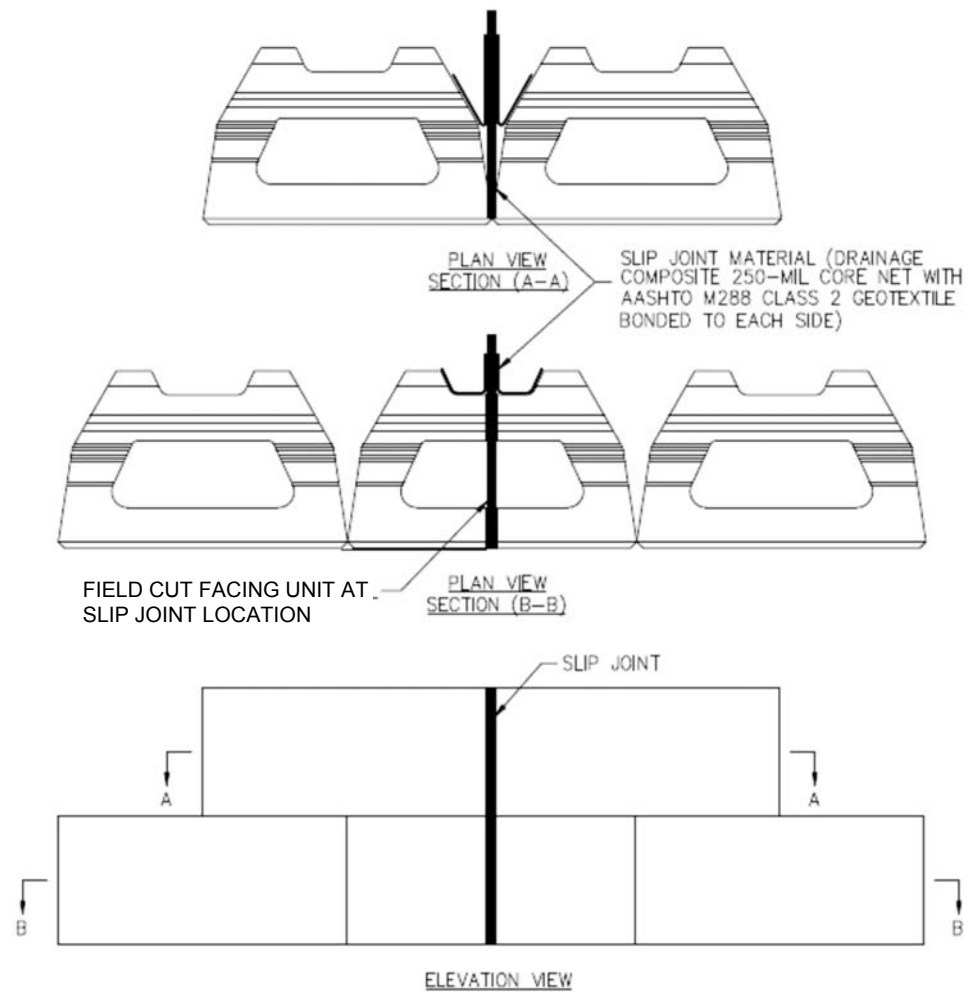


Figure 5-27. Example slip joint for modular block wall facings.

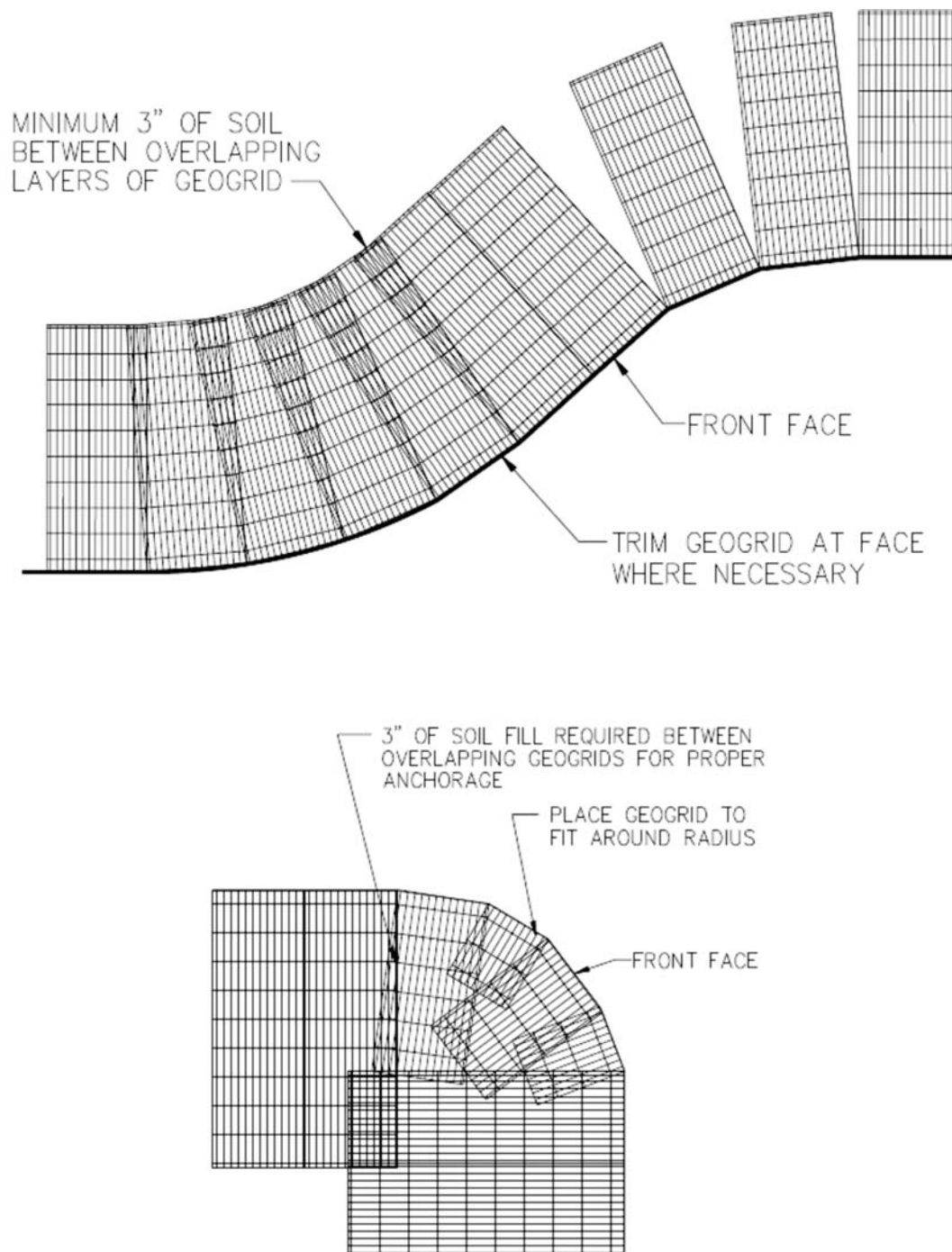


Figure 5-28. Example layout of geogrid reinforcement for walls with curves.

5.4.7 Wall Corners

When two MSE wall segments intersect to form an “external” (e.g., 90 degree) or an “internal” (e.g., 270 degree) corner, both wall segments will tend to move laterally such that corners tend to open up. Corner elements should be provided as shown in Figures 5-26a and 5-26b to accommodate differential movements, prevent fill from moving through the crack, and provide aesthetic treatment.

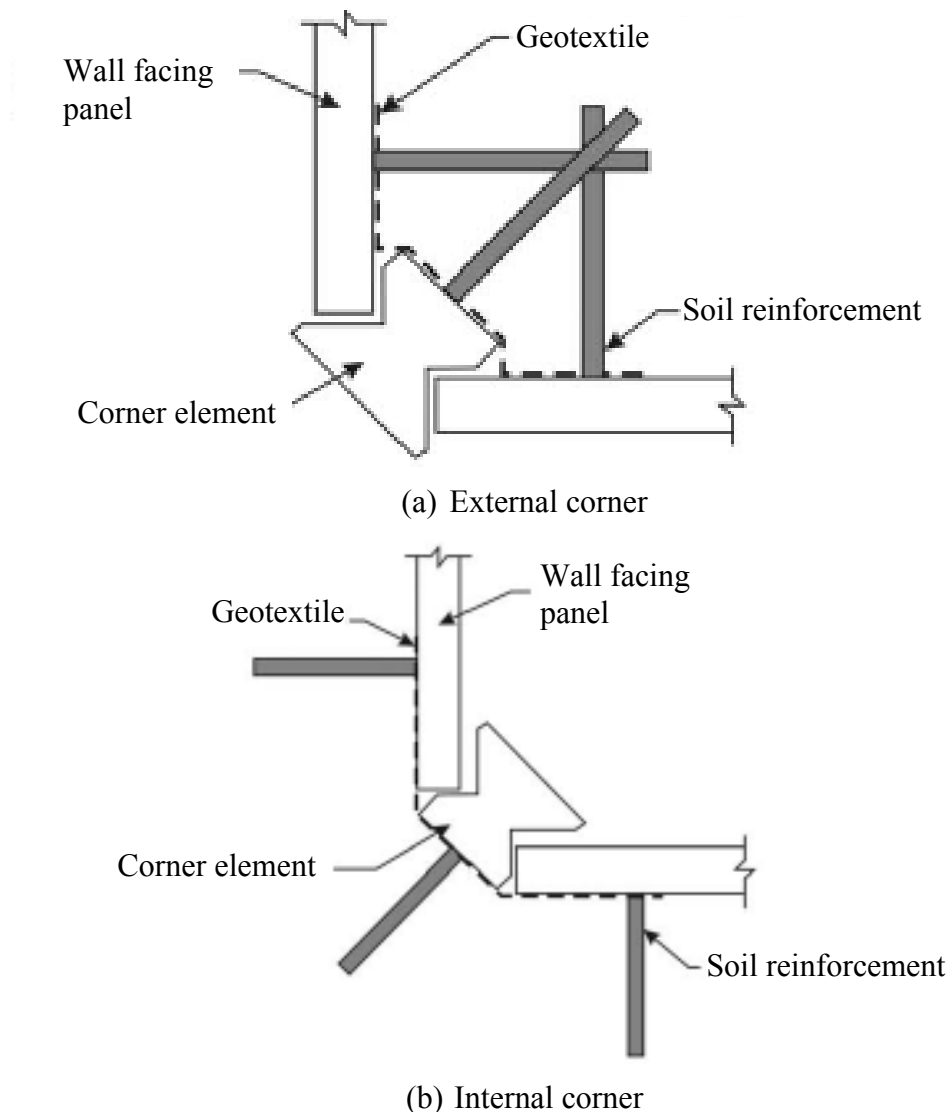


Figure 5-29. Example corner details.

Acute Angle Corners

External wall corners with an angle of less than 70 degrees, i.e., acute angle, should be avoided because of construction difficulties, e.g., compaction in corners and placement of reinforcements. However, if such a situation cannot be avoided, then the wall corner should be based on following considerations:

- The acute angle corner should be designed as a bin wall for the extent of the wall where the full length of the reinforcement cannot be installed without encountering the opposite wall face. In the bin wall section, the reinforcing elements are either structurally connected to both wall faces forming the acute angle corner or overlapped if there is adequate space to develop the required pullout resistance.
- Full-height vertical slip joints should be provided at the interface of acute corner and after the last column of panels where full length reinforcements can be placed.
- The soil reinforcement attached to the slip joints should be oriented perpendicular to the slip joint panels and shall be the full design length.
- Light weight concrete should be considered as an alternate to placing and compacting fill.
- Deformation compatibility between the bin wall section and the rest of the MSE structure should be carefully evaluated.

5.4.7 Two-Stage Facing

MSE walls with 2-stage facing systems can be used where significant (e.g., $> 1/100$) differential settlements are anticipated and use of slip joints, larger joint openings and/or ground improvement are not feasible to minimize the adverse effects of differential settlements. In an MSE wall with 2-stage facing, the primary MSE wall is constructed with a flexible face such as wire face or geosynthetic. After the primary flexible face wall has been constructed, it is left in place for a pre-determined amount of time to induce the settlements. Once the settlements are within acceptable limits, the facing units are installed in the second (final) stage. Figure 5-30 shows conceptual details of a 2-stage system that has been implemented in the industry; other similar details can be developed. Following are some general considerations for a 2-stage MSE wall system:

- The 2nd facing while usually consists of concrete panels should have a leveling/foundation pad with alignment/restraining mechanisms such as pins or dowels to receive and align the facing units as well as provide bearing resistance.
- In addition to the usual connections between facing units, e.g., tongue-and-groove joints, additional connection elements such as dowels may be needed based on the facing unit type.
- Turn-buckle type of connectors are used between the 1st stage wire mesh facing and the 2nd stage concrete facing units. The size and type of the turn-buckles and the number of connectors is a function of the facing panel size, distance between the two facing units, the type of infill used as well as the amount of relative settlement anticipated between the two facing systems after the 2nd stage facing is constructed. Detailed structural analysis and design of the connections should be performed.
- The sequence of construction should be clearly noted on the plans.

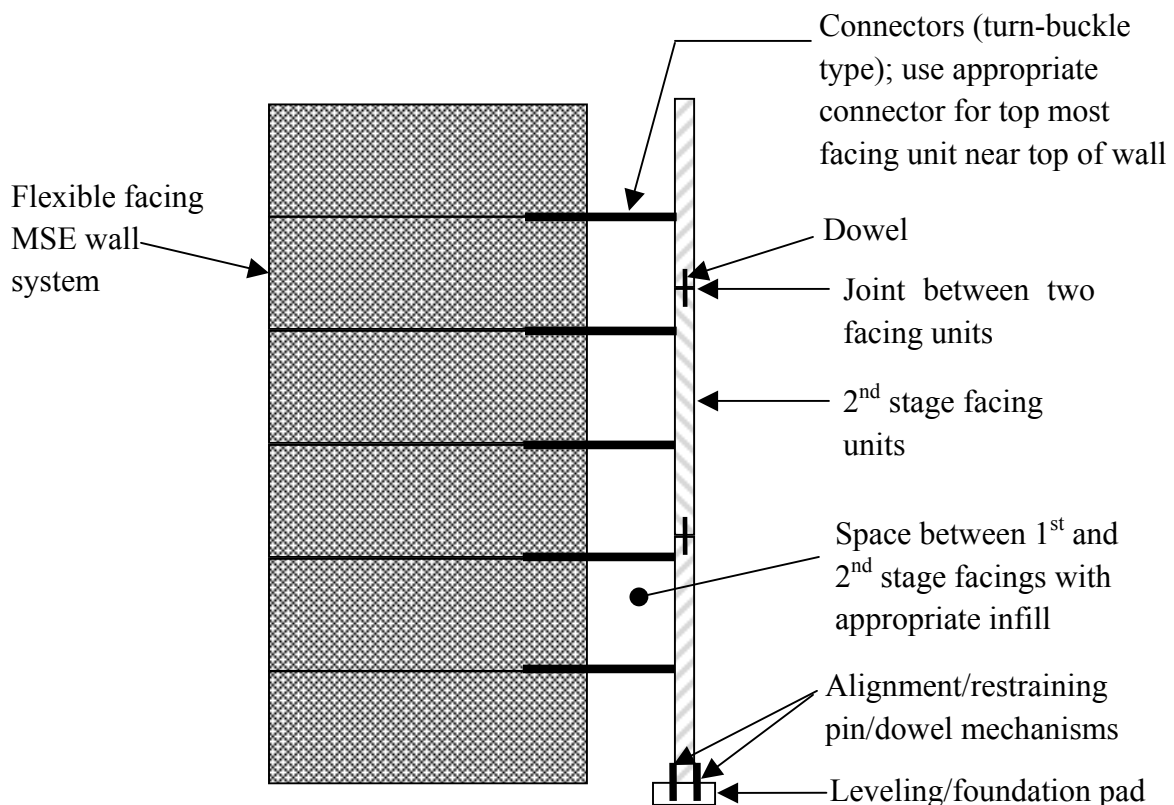


Figure 5-30. Conceptual connection details for a 2-stage facing system.

5.5 WALL INITIATIONS AND TERMINATIONS

The initiation and/or termination of an MSE wall may abut into another structure feature, slope or existing ground. The junctures of MSE walls with other structures are critical locations that are often observed to have distress such as misaligned facing units, leakage of backfill and erosion. Therefore, proper detailing is required at these locations. Following are some recommendations for wall initiations and terminations:

- The juncture of MSE walls and cast-in-place structures must be designed to prevent loss of fines and must allow for differential settlement between the two types of construction. Typical configurations for segmental precast panel facing units are shown in Figure 5-31. Either bituminous joint filler as shown in Figure 5-5a or a backer rod system and sealant as shown in Figure 5-31 is used. A common detail for a MBW facing unit is shown in Figure 5-32.
- A cast-in-place structure may have a lip to mask the joint as shown in Figure 5-31c. Sufficient distance between the facing and lip should be provided to allow for outward movement of the wall during construction. A geotextile should be used behind the joint to contain the soil. Joint filler such as expanded polystyrene may be used between the edge of the facing panel and the cast-in-place structure.
- Abrupt changes in wall heights should be avoided near wall initiation and termination points. This results in differential settlements and undesirable rotation of the facing units due to reduced confining pressures at such locations. Consideration may be given to not stepping the leveling pad within 10 ft (3 m) of the start or end of the wall.
- When starting or terminating into slopes and existing ground, the wall should be protected against erosion by vegetation and adequate embedment. In cases where the wall is adjacent to a steep slope or stream, riprap underlain by a filtration aggregate or geotextile may be needed. Swales should be used to divert water away from the end of the wall as discussed in section 5.3.3.

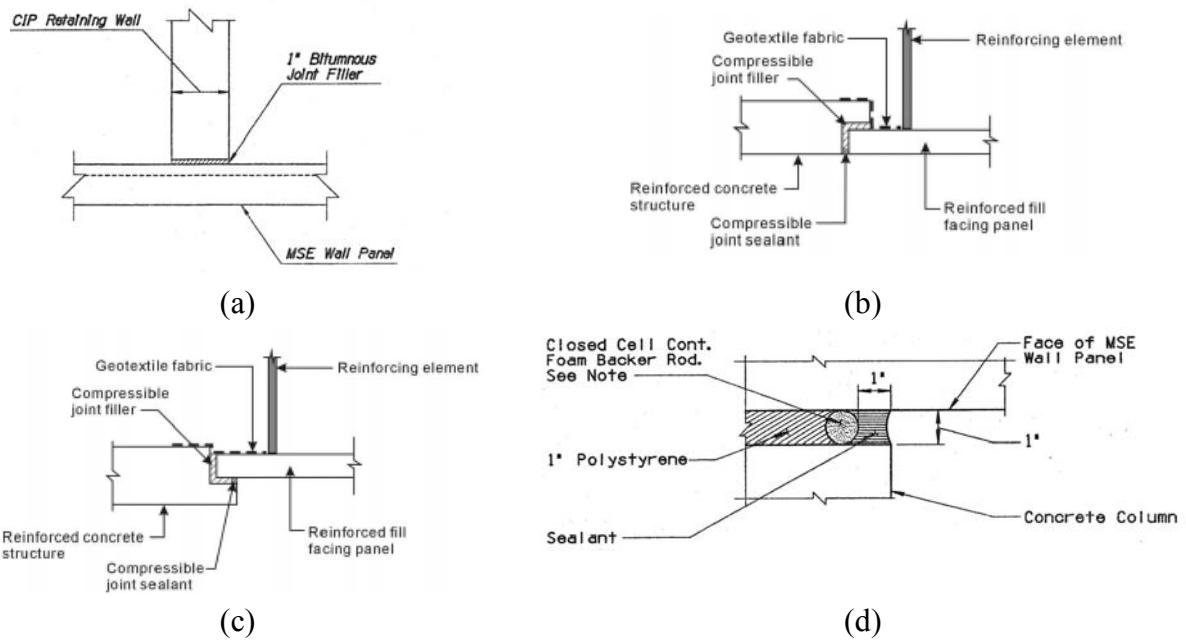


Figure 5-31. Common joint details between segmental precast panel facing units and CIP structures.

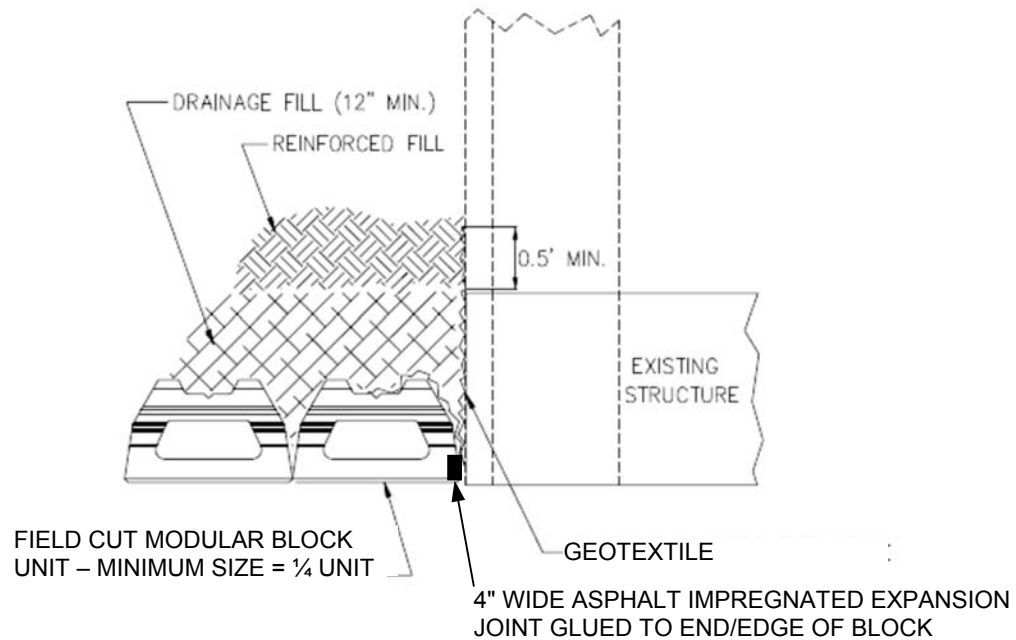


Figure 5-32. Common joint between modular block facing units and CIP structures.

5.6 AESTHETICS

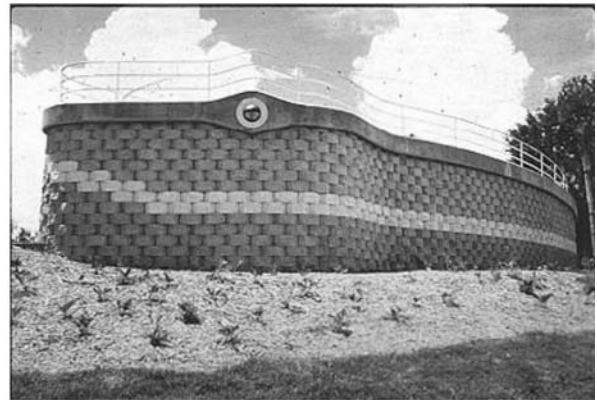
One of the attractive features of MSE walls is that aesthetics can be readily incorporated into the precast facing units. Several examples of wall aesthetics are shown in Figure 5-33. The choice of aesthetic treatments is virtually unlimited, however, it must be recognized that any aesthetic treatment should be compatible with the precasting processes and the construction tolerances. Following are some general guidelines that should be considered while developing aesthetic treatments:

- Cost of treatments that require special form-liners must be considered in the project cost estimate because form-liners require special fabrication and their number of uses is limited.
- Generally, the relief of the protruding artwork should be less than 1.5 in. (38 mm).
- Do not hang heavy aesthetic treatments from the facing units unless the facing units and the internal soil reinforcements are designed to withstand the forces from the artwork and environmental forces due to wind, snow, etc.
- Consider facing construction tolerances in formliner fabrication processes.
- Consider compatibility of the wall construction tolerances with the tolerances in details of the adjacent aesthetic treatments. Relief patterns are difficult to maintain between a cast-in-place structure and an adjacent MSE walls as illustrated in Figure 5-33(a), and is not recommended. Relief patterns between structures should be interrupted by a false column or other feature, as illustrated in Figure 5-33(b).
- Horizontal patterns parallel to the horizontal panel joints may not align after construction due to differential settlement.
- Consider using irregular patterns such as Ashlar stone that tend to hide inevitable imperfections in lines across joints between facing units.
- Consider the angle of sunlight expected at the location of the wall. At various times of the day, sunlight tends to accentuate the effect of the aesthetic features. The effect of imperfections resulting from regular construction tolerances on the artwork may be exaggerated leading to a false sense of problems and/or poor artwork.

- Consider use of colors that are compatible with the wall facing material and weathering of the color scheme.
- Aesthetic treatments may use obstructions, acute corners, and face penetrations for effect, which require careful design review along with increased inspection.



(a)



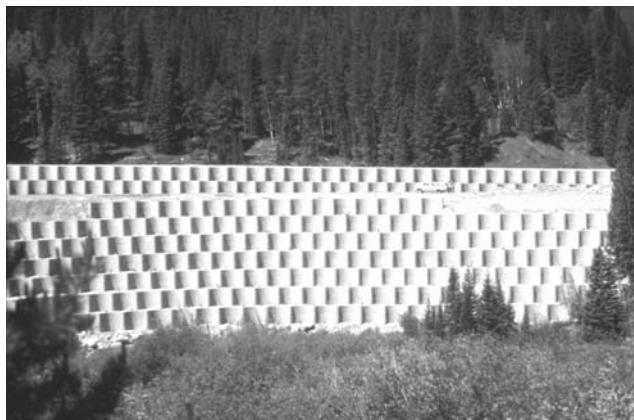
(b)



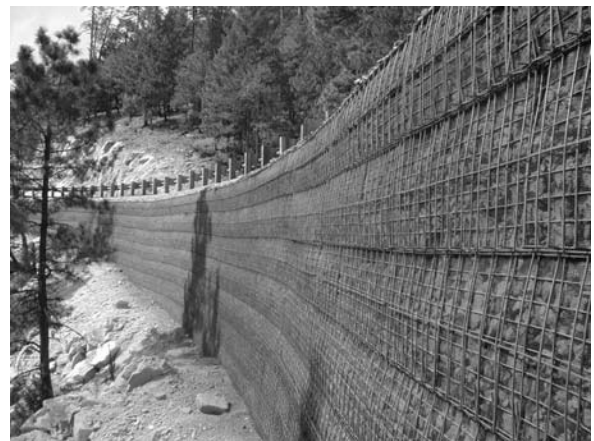
(c)



(d)



(e)



(f)

Figure 5-33. Example of MSE wall aesthetics.

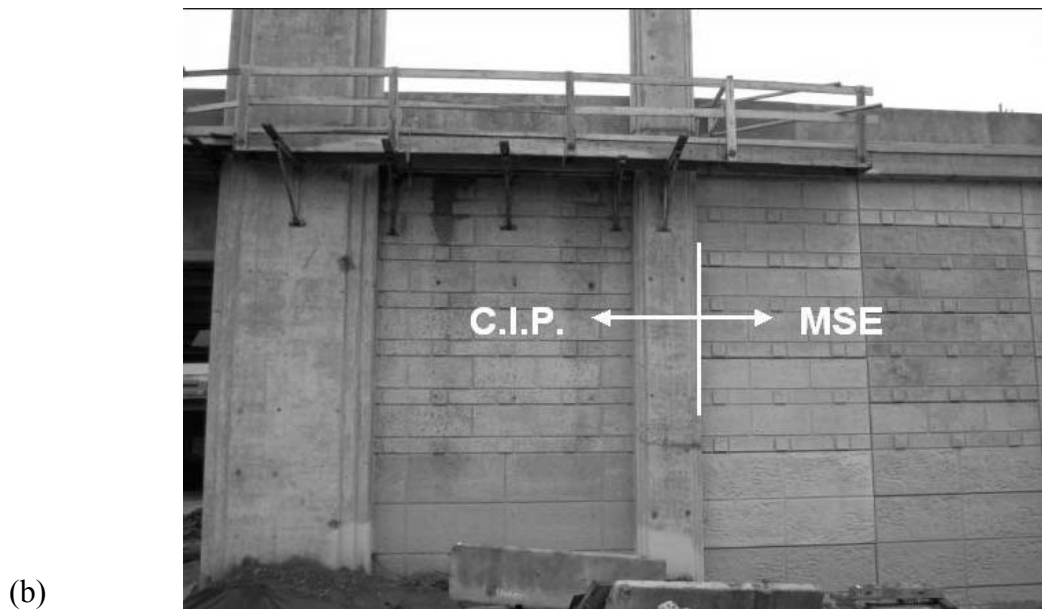
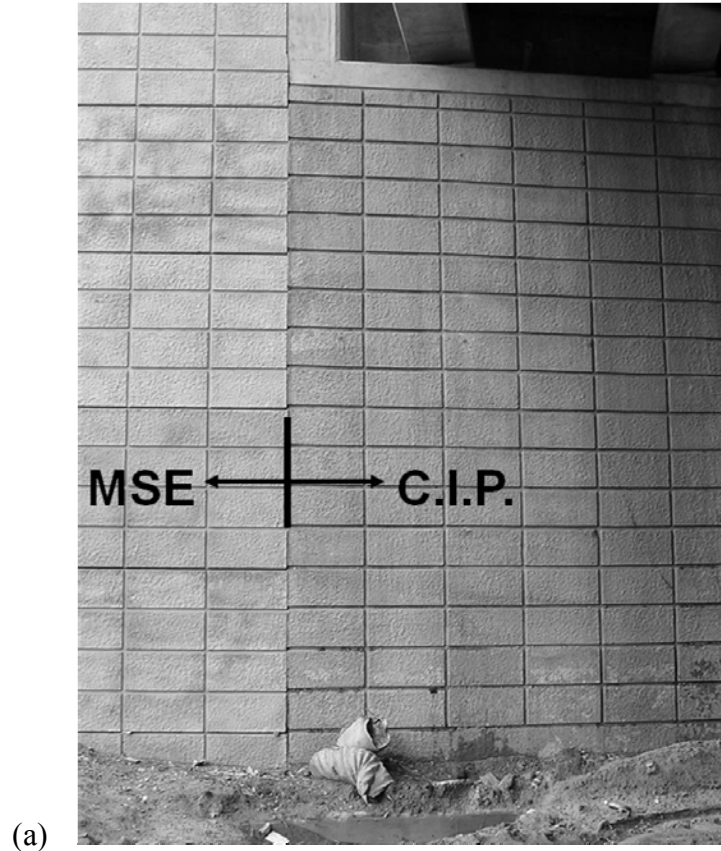


Figure 5-34. Examples of cast-in-place abutment to MSE wall panel transitions, (a) no transition between C.I.P. and precast panels and difficult to match lines, (b) a false column between C.I.P. and precast panels masks lines that may not match.

CHAPTER 6

DESIGN OF MSE WALLS WITH COMPLEX GEOMETRICS

The basic design methods outlined in Chapter 4 consider MSE structures with simple geometries with reinforcement layers of the same length supporting either a horizontal backfill or a surcharge slope. Although most MSE structures fall into this category, structures with more complex geometries or significant external loads are feasible and require consideration during the selection process. They include:

- Bridge abutments with MSE walls
- Superimposed (tiered) MSE walls
- MSE Walls with uneven length reinforcements (trapezoidal walls)
- Back-to-back MSE (BBMSE) walls
- Shored MSE (SMSE) walls for steep terrains and low volume roads
- Stable feature MSE (SFMSE) walls

Schematics of these complex cases are illustrated in Figure 6-1.

The shape and location of the maximum tensile forces line are generally altered by both the geometry and the loads applied on the complex MSE wall structure. It is possible to assume an approximate maximum tensile force line for each. However, supporting experience and analysis are more limited than for rectangular reinforced soil walls.

For complex or compound structures, it is always difficult to separate internal stability from external stability because the most critical slip-failure surface may pass through both reinforced and unreinforced sections of the structure. For this reason, both global and compound stability analyses are required for these types of complex structures. The current method for performing these analyses is to use an ASD reinforced soil slope stability computer method, as detailed in Chapter 9. An alternative method is to adapt the simple modification to the global and compound stability analyses for the LRFD procedure as discussed in Chapter 4.

The following sections give guidelines for each complex case identified in Figure 6-1.

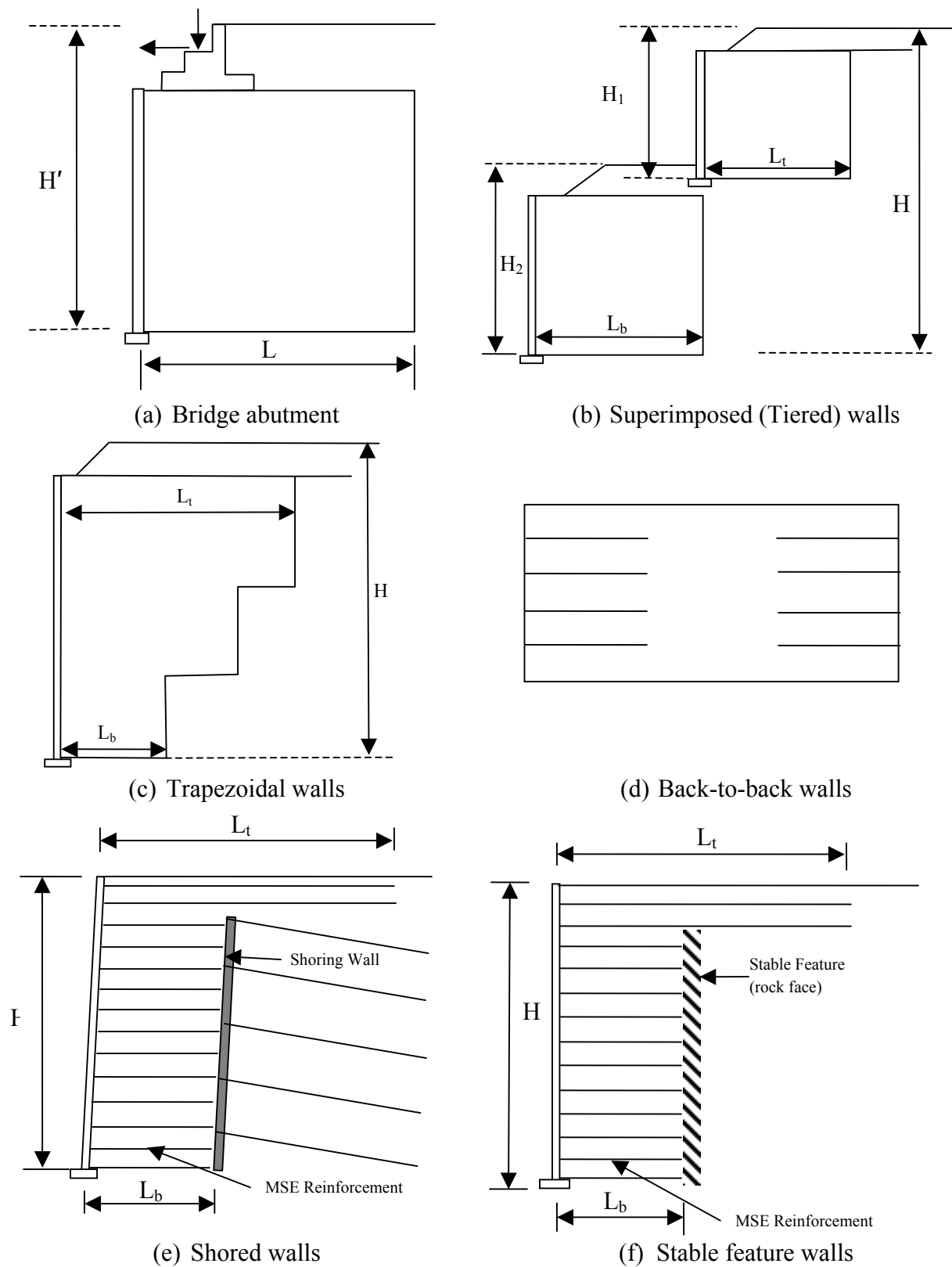


Figure 6-1. Types of complex MSE structures.

6.1 BRIDGE ABUTMENTS WITH MSE WALLS

Bridge abutments have been designed to support the bridge superstructure on a spread foundation constructed directly on the reinforced soil zone, or on a deep foundations constructed through the reinforced soil zone.

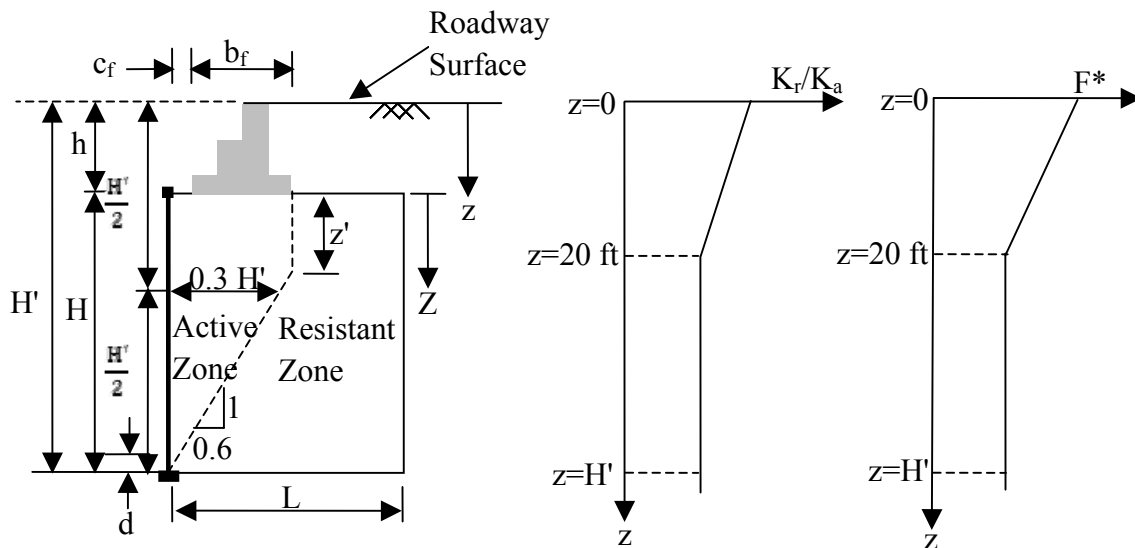
The configuration wherein bridge superstructure is supported on a spread footing on top of the reinforced soil zone may be more economical compared to abutments supported by deep foundation through the reinforced soil zone, and should be considered when the projected settlement of the foundation and reinforced volume is rapid/small or essentially complete, prior to the erection of the bridge beams. Based on field studies of actual structures, AASHTO (2007) suggests, that tolerable angular distortions (i.e., limiting differential settlements) between abutments or between piers and abutments be limited to the following angular distortions (in radians):

- 0.008 for simple spans, and
- 0.004 for continuous spans

This criteria, suggests that for a 100 ft (30 m) span for instance, differential settlements of 4.8 in. (120 mm) for a continuous span or 9.6 in. (240 mm) for a simple span, would be acceptable, with no ensuing overstress and damage to superstructure elements. On an individual project basis differential settlements of smaller amounts may be required from functional or performance criteria. Settlements well within the tolerable range can often be achieved with MSEW abutments on spread footings.

6.1.1 MSEW Abutments on Spread Footings

Where fully supporting the bridge loads, MSEW bridge abutments are designed by considering them as rectangular walls with surcharge loads at the top. The base width of the bridge support spread footing, b_f , and the location of the toe of the footing with respect to the back face of the walls panels, c_f , is commonly such that $b_f + c_f$ is greater than $H/3$. In this case, the shape of the maximum tensile force line, i.e., the critical failure surface, has to be modified to extend to the back edge of the spread footing. The variation of K_r/K_a and F^* also need to be modified. Figure 6-2 shows definitions of various parameters including measurements of heights and depths.



Notes:

- d is the depth of embedment
- Z is measured below bottom of footing; z is measured from top of spread footing
- H is measured from top of leveling pad to bottom of bridge support spread footing
- h is height of the wall as measured from bottom of bridge support spread footing to finished roadway surface
- H' is height of wall as measured from top of leveling pad to finished roadway surface
- $z = Z + h$; $z' = H - (c_f + b_f)/0.6$
- Within height z' the length of the reinforcement in the active zone is $L_a = c_f + b_f$

Figure 6-2. Geometry definition, location of critical failure surface and variation of K_r and F^* parameters for analysis of a MSEW abutment on spread footing.

Although MSEW abutments on spread footings have historically almost always used inextensible, steel reinforcements, they can also be used with extensible reinforcements. However, similar shifts in the maximum tension line to the back of the footing have been observed for extensible reinforcement. Therefore, the maximum tensile force line should also be modified for extensible reinforcement if the back edge of the footing extends beyond a distance of $H \cdot \tan(45^\circ - \phi^\circ/2)$ from the wall face. These maximum tensile force lines should be compared with the critical failure surface from compound stability analysis and the more conservative profile of the failure surface should be selected.

Successful experience with construction of MSEW abutments on spread footings has suggested that the following additional details be implemented:

- Require a minimum offset from the front of the facing to the centerline of the bridge bearing of 3.5 ft (1 m).
- Require a minimum clear distance, c_f , of 6 in. (150 mm) between the back face of the facing panels and the front edge of the footing.
- In areas that are susceptible for frost, the frost effect can develop from both the top of the wall as well as the front of the wall. Where significant frost penetration is anticipated, place the abutment footing on a bed of non-frost susceptible compacted coarse aggregate (e.g., No. 57 as specified in AASHTO M 43). The thickness of the aggregate bed should be minimum 3 ft (1 m) or 1 ft (0.3 m) below deepest anticipated frost penetration depth, whichever is greater. Separation geotextile should be provided at the interface of No. 57 coarse aggregate and the surrounding fills (reinforced, retained and above the footing base). Adjoining sections of the separation geotextile should be overlapped by a minimum of 1 ft (0.3 m).
- For the analysis of the spread footing on top of the reinforced soil zone, use the following values of bearing resistance of the reinforced soil zone
 - For service limit state, bearing resistance = 4 ksf (200 kPa) to limit the vertical movement to less than approximately 0.5 in. (12.5 mm)
 - For strength limit state, factored bearing resistance = 7 ksf (335 kPa)
(Note: AASHTO does not provide a value of factored bearing resistance at strength limit state and the recommended value is based on the authors' experience.)
- Use the maximum horizontal force at top reinforcement level below the abutment for the design of connections of the panels at all reinforcement levels.
- Extend the density, length and cross-section of reinforcements of the abutments to wingwalls, for a horizontal distance which is greater of the following:
 - 50 percent of the maximum height, H , of the abutment wall face.
 - $c_f + b_f + 3$ ft (1 m) where c_f and b_f are as shown in Figure 6-2
- There will be 2-way soil reinforcement within the length of reinforcement perpendicular to the abutment face. It is preferable that reinforcement is not placed on top of each other in the zone of 2-way reinforcement. The overlapping reinforcement should be separated by 3 to 6 in. (75 to 150 mm) of soil or some multiple of compacted fill height. This may be achieved by appropriately adjusting the steps of the leveling pad between the abutment face wall and the wing walls. This practice is especially recommended where a corrosion monitoring program is implemented in the abutment area (Elias et al., 2009).

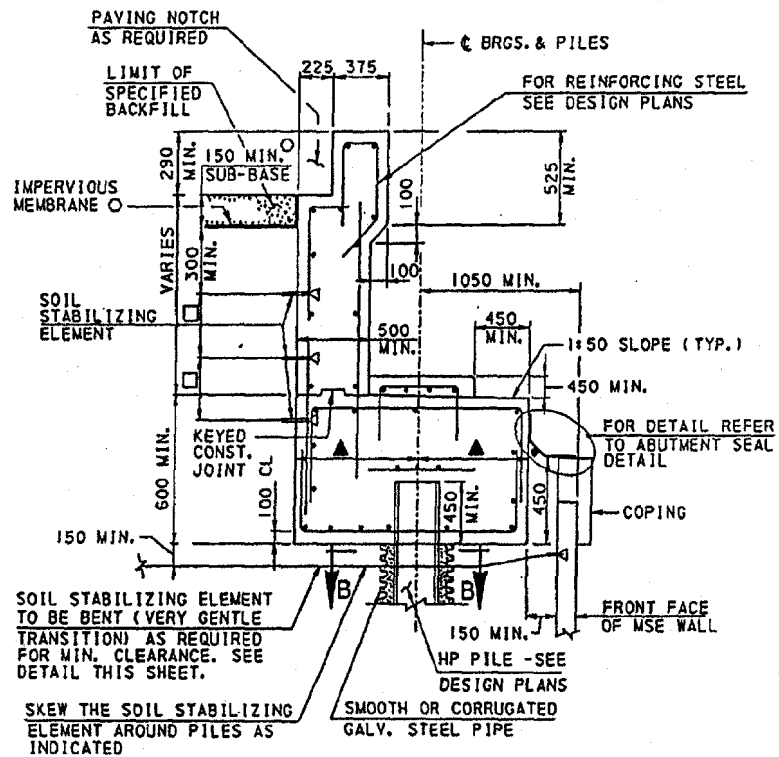
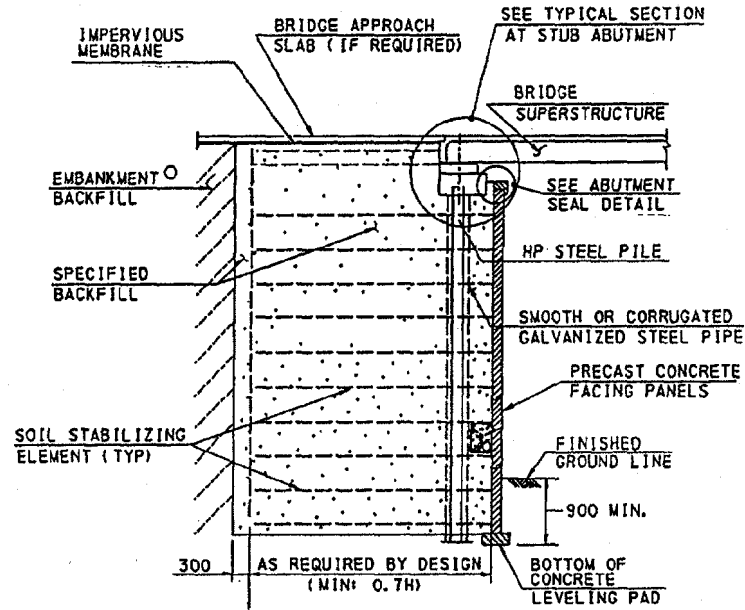
- To prevent adverse stress concentrations at the reinforcement connections, the minimum vertical clearance between the bottom of the bridge support spread footing and the top level of reinforcement should be 1 ft (0.3 m).
- Due to the relatively high bearing pressures near the panel connections, the adequacy and nominal capacity of panel connections should be determined by conducting pullout and flexural tests on full-sized panels.
- The seismic design forces should also include seismic forces transferred from the bridge through bearing supports which do not slide freely (e.g., elastomeric bearings).

In the LRFD context, the design of a MSEW abutment on spread footing requires careful separation of various load types. This results in a complex set of inter-related equations which are best illustrated by a worked example. Example E4 presents a comprehensive step-by-step illustration of both external and internal stability of a MSEW abutment on spread footing. The reader should become familiar with Example E5 because the principles and computations used in the example problem can also be applied to different complex geometries.

6.1.2 MSEW Abutments on Stub Footings Supported by Deep Foundations through Reinforced Wall Fill

For cases where MSEW abutments on spread footings may not be viable based on considerations of unacceptable post-construction settlements or other reasons, the bridge superstructure is placed on stub footings supported by deep foundations such as driven piles or drilled shafts. In this configuration, vertical loads are not considered in analysis since they are transmitted to an appropriate bearing stratum by deep foundations. However, the horizontal bridge and abutment backwall forces must be resisted by methods dependent on the type of abutment support, namely:

- **For conventional abutments**, the horizontal forces must be resisted by extending soil reinforcement from the back edge of the abutment footing (cap). The resistance is provided by the interaction between the soil and reinforcement over the full length of the reinforcement. A typical detail is shown in Figure 6-3. Alternatively, the horizontal forces may be resisted by the lateral resistance of the deep foundation or by other means.



Note: All dimensions are in mm [1 in. = 25.4 mm]

Figure 6-3. Details of a typical pile supported MSE abutment.

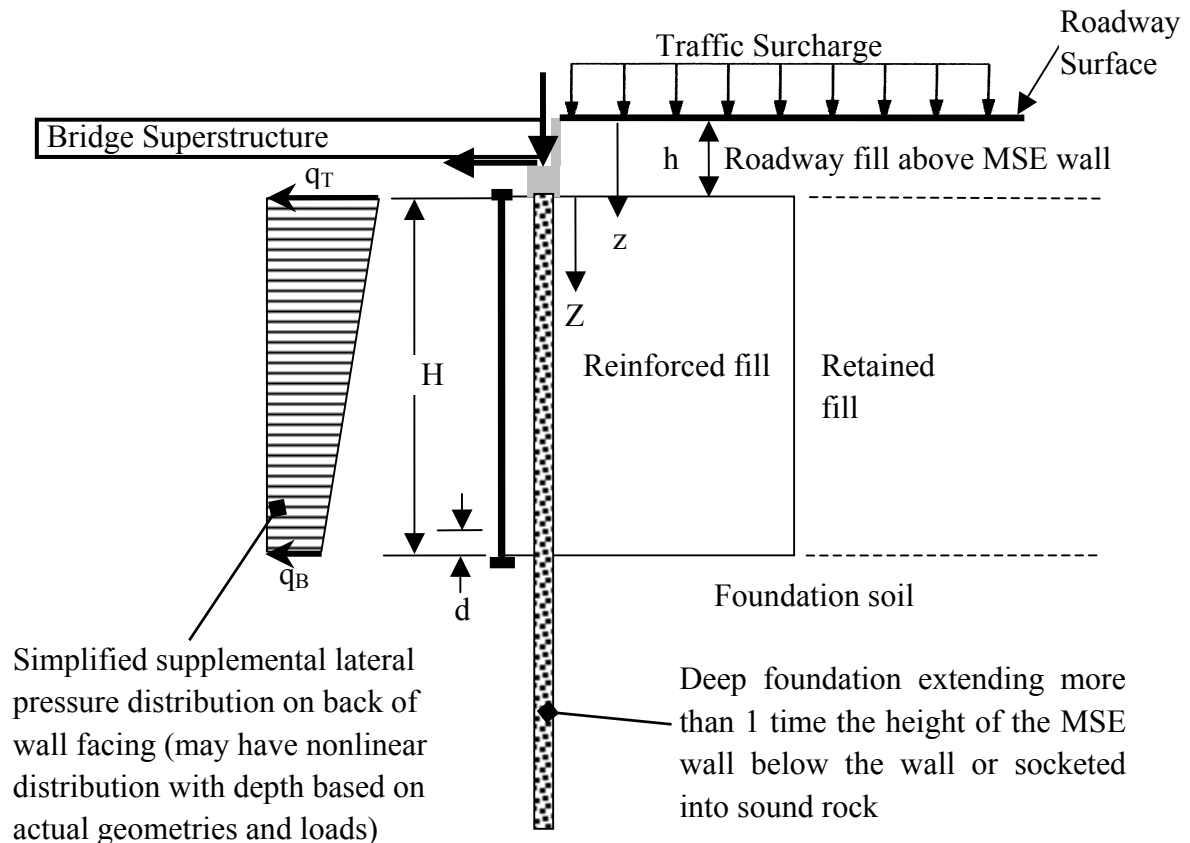
- **For integral abutments**, the horizontal forces and its distribution with depth may be developed using a lateral load (p)-lateral deflection (y), i.e., p-y methods. These horizontal forces are added as a supplementary force to be resisted by the reinforcements. These forces will vary depending on the following:
 - o magnitude of the horizontal loads and moments,
 - o diameter and spacing of deep foundations, and
 - o clear distance between the back face of wall panels and front of the deep foundation elements.

Several agencies have constructed integral abutments in front of MSE walls as discussed in Section 6.1.3 in order to avoid applying a lateral stress to soil behind the abutment.

Figure 6-4 shows a typical supplemental lateral pressure that must be considered in the internal stability analysis. This lateral pressure is addressed in a fashion similar to the inverted triangular lateral pressure distribution shown in Figure E5-2 of Example Problem E5. The effect of the roadway fill and the live load surcharge above the MSE wall is also addressed in a fashion similar to that for the same features in Example E5. The balance of the computations remains identical to those in Chapter 4.

Based on successful experience of the authors with abutment construction of MSE walls with deep foundations through the reinforced fill, following are suggested additional details, as applicable:

- Where significant settlement of the embankment is anticipated, provide casings (e.g., sonotubes or corrugated metal pipes) in the reinforced soil zone to permit construction of deep foundations after the MSE wall is constructed and settlement has occurred. In the case of driven piles it may be possible to isolate the piles from the casings by filling the annulus with loose sand just prior to construction of the footing at top of the piles. In the case of drilled shafts it may not be possible to isolate the shaft from the casing in an economical manner unless another internal casing is used.
- In the case where deep foundations are constructed prior to MSE wall construction, and negative skin friction, i.e., downdrag force, is anticipated, provide a casing around the deep foundation element, through the reinforced fill. The casing is filled with sand just prior to the construction of the footing at top of the deep foundation element. Alternatively, a bond breaker can be used on the deep foundation element when negative skin friction, i.e., downdrag force, is anticipated.



Notes:

- d is the depth of embedment
- H is measured from top of leveling pad to top of coping
- z is measured below finished grade behind footing
- The K_r/K_a and F^* values are assumed to start at $z = 0$, i.e., below the finished grade behind the footing.

Figure 6-4. Geometry definition and typical supplemental lateral pressure distribution from deep foundation on MSE wall face.

- In the case where deep foundations are constructed prior to MSE wall construction and/or the deep foundation element is not isolated from the casing as noted above, the horizontal stresses as shown in Figure 6-4 must be included in the analysis of MSE wall. If the deep foundations are constructed through casings and isolated from the casings, the horizontal stresses may be neglected in the design of the MSE wall. However, it must be realized that this configuration leads to a longer unsupported length of the deep foundation that may result in undesirable movements at the bridge seat level in addition to increased size of the deep foundation element.

- For driven piles extending through the reinforced soil zone, require a minimum offset from the back face of the wall panels and the front of the driven pile elements as 1.5 ft (0.5 m).
- For drilled shaft extending through the reinforced soil zone, require a minimum offset from the back face of the wall panels and the front of drilled shaft elements as 3 ft (1 m). This criterion provides the necessary clear space to achieve proper compaction of the soil in this area. Thus, for example, if drilled shafts with a maximum dimension of 2 ft (0.61 m) are used then the minimum clearance is 3 ft (1 m). For walls where reinforcements will be splayed (e.g., steel strip reinforcements), require a minimum offset from the back face of the wall panels and the front of deep foundation elements as the greater of 3 ft (1 m) or 1 times the diameter of the deep foundation. Thus, for example, if a drilled shaft of 4 ft (1.2 m) diameter is used then the minimum clearance is 4 ft (1.2 m). These criteria provides the necessary clear space to achieve proper compaction of the soil in this area and adequate distance for splaying of reinforcements within the acceptable limits noted in Chapter 5.
- Provide soil reinforcements in the soil behind the abutment footing (cap) as shown in Figure 6-3.

Interference between Soil Reinforcements and Deep Foundations

Design of MSE walls with deep foundations needs careful consideration of the interference between the soil reinforcements and the deep foundation element(s). Where deep foundation elements interfere with the reinforcements, specific methods for field installation must be developed and presented on the plans. **Simple cutting and then bending of the reinforcements during construction should not be allowed.** Guidance for navigating soil reinforcement around vertical obstructions is presented in Chapter 5. Soil cover as recommended in Section 5.4.1 of Chapter 5 between dissimilar metals should be implemented as appropriate.

6.1.3 Alternative Configuration of MSE Walls at Bridge Abutments

An alternative to construction of MSEW abutments that use deep foundations through reinforced backfill is to construct the MSE walls behind abutment foundations that are constructed. In this configuration, the foundations are not constructed within or on top of reinforced fills. Rather the MSE walls supports only the approach fills while the abutments are constructed in configuration of piers. Special details (e.g., bridge approach slabs) are required for this configuration to integrate the MSE walls with the bridge abutment. The major advantage of this type of abutment configuration is that the construction of the foundations for the abutments can be performed independently of the MSE wall construction and that better construction control can be exercised for MSE walls since there are no obstructions through the reinforced backfill.

Based on successful experience of the authors with construction of an abutment configuration with MSE wall behind the abutment foundations, following are suggested additional details, as applicable:

- Require that the bridge superstructure be placed after the construction of the MSE walls so that most of the possible foundation deformations have occurred.
- Construct the foundations prior to construction of the MSE wall, but construct the abutment columns after the construction of the MSE wall. In this construction sequence, the foundation deformation due to the construction of the adjacent MSE wall can be compensated for by adjusting the connection of the abutment structure rather than running the risk of abutment structure deforming to the extent that it does not fit with the bridge superstructure at the beam seat level.
- Consider construction of a false wall in front of the abutment substructure, i.e., the element between the foundation and the superstructure. This false panel serves to protect the abutment elements against vehicular impact as well as protecting vehicle occupants. The false wall may be structurally connected to the abutment substructure or an independent wall with a separation of 3 to 6 in. (75 mm to 150 mm) with the decision based on the design of the abutment substructure and its ability to absorb vehicular impacts.
- For integral abutments with a wall supported on deep foundations, a wrapped or wire faced MSE wall can be constructed behind the abutment wall, using the abutment as an offset form with a spacer to maintain the distance between the MSEW and the abutment wall as shown in Figure 6-5.

6.1.4 Protection of MSE Wall at Abutments

At abutment locations, the permeation of water through expansion joints into the MSE wall results in a number of seepage problems as discussed in Chapter 5 including the potential for salt-laden runoff, which could result in a chloride rich, corrosive environment near the face panel connection for a significant percentage of the wall height. To minimize this problem, seepage should be controlled as shown on Figure 6-6.

6.2 SUPERIMPOSED (TIERED) MSE WALLS

For tall walls consideration should be given to superimposed (tiered) walls from the viewpoint of constructability. Reconfiguring a tall wall in superimposed walls with smaller heights permits a fresh start with a new leveling pad, reduces vertical stress on facing elements, and permits better control of vertical alignment of the wall face. Analytically, depending on the offsets between the superimposed walls, another beneficial effect might be an overall (equivalent) sloped wall face that results in lesser lateral force on the whole wall system.

6.2.1 2-Tier Superimposed Walls

Figure 6-7 shows a configuration of a 2-tier superimposed MSE wall system. The design of superimposed MSE walls requires two analyses as follows:

- (1) A design using simplified design rules for calculating external stability and locating the internal failure plane for internal stability as shown in Figure 6-7.
- (2) A slope stability analysis, including both compound and global stability using a reinforced soil global stability computer program outlined in Chapter 4. This is an essential computation.

The definition of wall heights, H_1 and H_2 , and offset D between walls for a 2-tier superimposed wall configuration is shown in Figure 6-7. Using the definitions in Figure 6-7, for **preliminary design**, the following minimum values for reinforcement length, of L_1 and L_2 , should be used for offsets (D) greater than $[1/20 (H_1 + H_2)]$:

Upper wall: $L'_1 \geq 0.7 H_1$

Lower wall: $L'_2 \geq 0.6 H$ where $H = H_1 + H_2$

Based on the definitions in Figure 6-7, following are basic design guidelines:

- Where the offset distance (D) is greater than $H_2 \tan (90-\phi_r)$, walls are not considered superimposed and are independently designed from an internal stability viewpoint,
- For a small upper wall offset; $D \leq [1/20 (H_1 + H_2)]$, it is assumed that the failure surface does not fundamentally change and it is simply adjusted laterally by the offset distance D . The walls should be designed as a single wall with a height H .

In both of the above cases, compound and global stability should be checked.

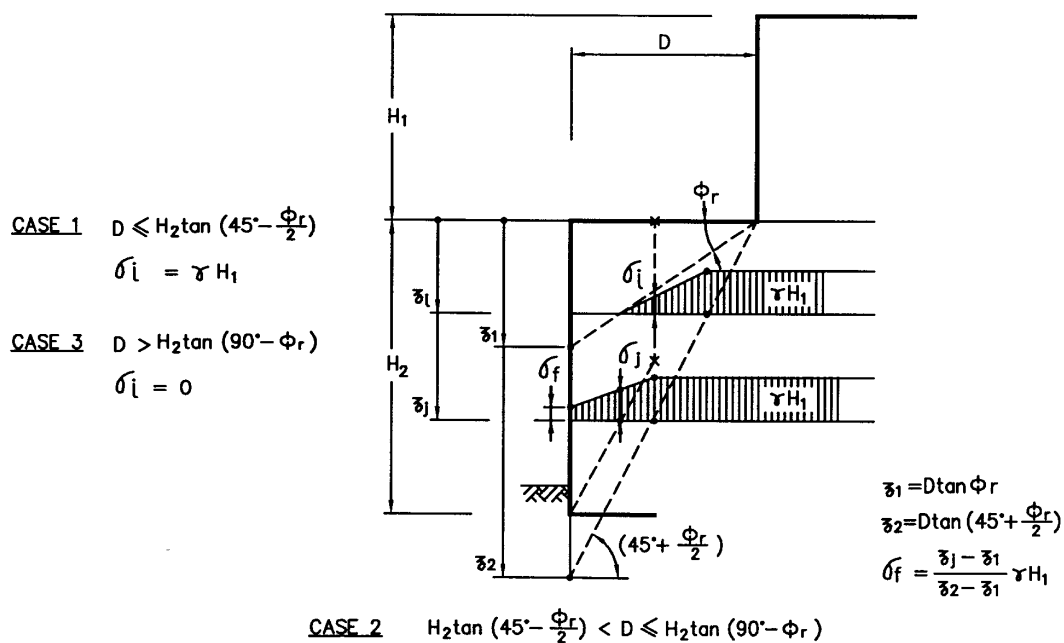
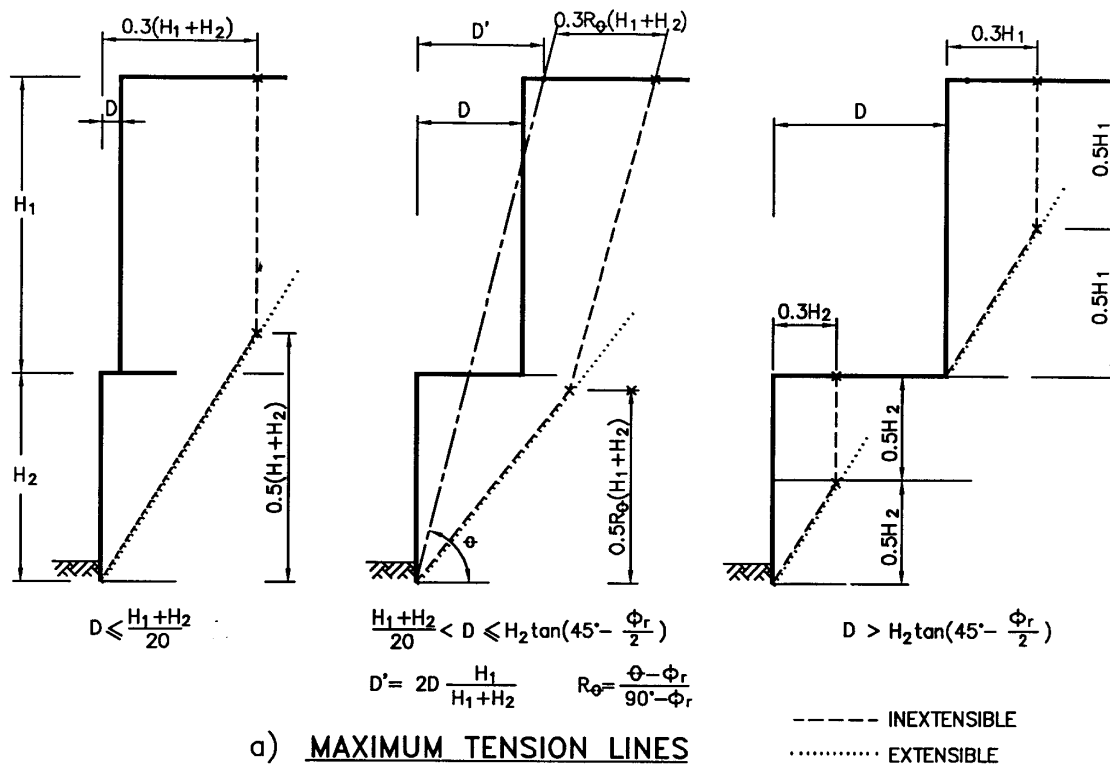


Figure 6-7. Design rules for a 2-tier superimposed MSE wall system.

The stability analysis for a 2-tier superimposed MSE wall system is performed as follows:

- External stability calculations for the upper wall are conventionally performed as outlined in Chapter 4. For the lower wall, consider the upper wall as a surcharge (Load type “ES”) in computing bearing pressures. In lieu of a conventional external sliding stability computation, perform a wedge type slope stability analysis with failure surfaces along and exiting at the base as well as below the base. The overall stability should be investigated at the Service I load combination and a sliding resistance factor of 0.65.
- For calculating the internal stability, the maximum tensile force lines are as indicated in figure 6-7a. These relationships are somewhat empirical and geometrically derived.
- For intermediate offset distances, see Figure 6-7a for the location of the failure surface and consider the vertical pressures in Figure 6-7b for internal stress calculations.
- For large setback distances, $[D \geq H_2 \tan (90-\phi_r)]$, the maximum tensile force lines are considered independently, without regard to the geometry of the two superimposed walls. For internal stability computations, the upper wall is neglected.
- The balance of the computations remains identical as in Chapter 4.

6.2.2 Superimposed walls with more than 2-tiers

The criteria for 2-tier wall presented in Figure 6-7 can be extended to walls with more than two tiers. For such configurations, the global and compound stability analysis becomes even more critical. Methods outlined in Chapter 4 may be used for evaluating the global and compound stability. For internal stability analysis, Wright (2005) and Leschinsky and Han (2004) found that the criteria for additional vertical stress in Figure 6-7b may be used for walls with more than 2-tiers provided that only the immediately overlying tier is considered to contribute to the increase in vertical stress on the lower tier. As an alternative, Wright (2005) presents an elastic solution based on an assumption of “rigid” walls for estimating additional vertical stresses in a given tier of a multi-tier wall due to the effect of all overlying wall tiers. Regardless of the approach used for estimating the increase in vertical stresses for evaluation of internal stability, the analysis of tiered walls should proceed from the top wall to the bottom wall so that the stresses are properly accumulated and accounted for in the design of the bottom-most wall. For preliminary design, the length of the reinforcement of the bottom-most tier can be assumed to be 0.6 times the total height of the wall system.

6.3 MSE WALLS WITH UNEVEN REINFORCEMENT LENGTHS (TRAPEZOIDAL WALLS)

Use of this type of reinforcement geometry should be considered only if the base of the MSE structure is in rock or *competent foundation material*, i.e., foundation materials which will exhibit minimal post construction settlements. Examples of competent foundation materials include materials with SPT N_{60} value greater than 50 and sound rock.

The design of these walls requires two analyses as follows:

1. A design using simplified design rules for determining internal and external stability.
2. A slope stability analysis performed using a reinforced soil stability program checking both global (i.e., circular and wedge type analysis) and compound failure planes.

Simplified design rules for these structures are as follows:

- As shown in Figure 6-8, the wall is represented by a rectangular block (L_o , H) having the same total height and the same cross-sectional area as the stepped section for external stability calculations.
- The maximum tensile force line is the same as in rectangular walls (bilinear or linear according to the extensibility of the reinforcements).
- Minimum base length (L_3) of $0.4H$ or 8 ft (2.5 m) whichever is greater, with the difference in length in each zones being less than $0.15 H$.

For internal stability calculations, the wall is divided in rectangular sections and for each section the appropriate L (L_1 , L_2 , L_3), is used for pullout calculations, using methods developed in Chapter 4.

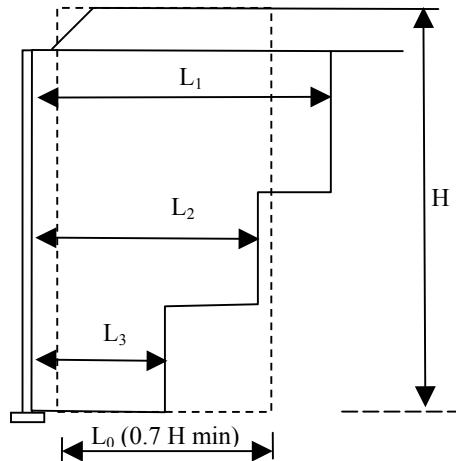


Figure 6-8. Dimensioning of MSE wall with uneven reinforcement lengths.

6.4 BACK-TO-BACK MSE (BBMSE) WALLS

Back-to-back walls are often used for highway ramps. For walls which are built back-to-back as shown in Figure 6-9, a modified value of lateral pressure influences the external stability calculations. As indicated in Figure 6-9, two cases can be considered and are discussed below.

- Case I

For Case I, the overall base width is large enough so that each wall behaves and can be designed independently. In particular, there is no overlapping of the reinforcements. Theoretically, if the distance, D , between the two walls is shorter than $D = H_1 \tan (45^\circ - \phi^\circ/2)$ where H_1 is the taller of the parallel walls, then the active wedges at the back of each wall cannot fully spread out and the active thrust is reduced. However, for design it is assumed that for values of $D > H_1 \tan (45^\circ - \phi^\circ/2) \approx 0.5H_1$ then full active thrust is mobilized.

- Case II

For Case II, there is an overlapping of the reinforcements such that the two walls interact. When the overlap, L_R , is greater than $0.3H_2$, where H_2 is the shorter of the parallel walls, no active earth thrust from the backfill needs to be considered for external stability calculations.

For intermediate geometries between Case I and Case II, the active earth thrust may be linearly interpolated from the full active case to zero.

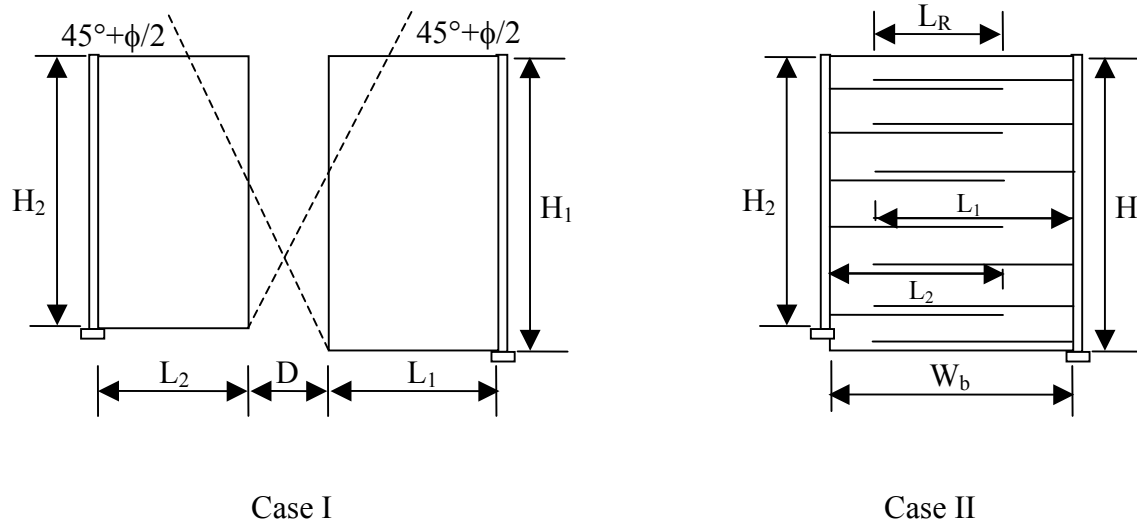


Figure 6-9. Back-to-back MSE walls.

For Case II geometries with overlaps (L_R) greater than $0.3H_2$, the following guidelines should be used:

- $L_1/H_1 \geq 0.6$ where L_1 and H_1 is the length of the reinforcement and height, respectively, of the taller wall.
- $L_2/H_2 \geq 0.6$ where L_2 and H_2 is the length of the reinforcement and height, respectively of the shorter wall.
- $W_b/H_1 \geq 1.1$ where W_b is the base width as shown in Figure 6-9 and H_1 is the height of the taller wall.

The above guidelines are valid for static load conditions or in areas where the seismic horizontal accelerations at the foundation level are less than $0.05g$. Back-to-back walls in seismically active areas should be designed based on a more detailed analysis that includes effects of potential non-uniform distribution of seismic and inertial forces within the wall.

For back-to-back walls designers might be tempted to use single layers of reinforcements that are connected to both wall facings. This alternative creates an unyielding structure creating an at rest stress state (K_0) from the top to the bottom of the wall, resulting in much higher reinforcement tensions than previously used in the design method in this manual. The design must include the increases in lateral stress in the determination of the tension in reinforcement and connection and in the design of facing elements. Additionally compaction may induce higher stress at the connection, which must be accounted for in the lateral earth pressure

calculations. Furthermore, difficulties in maintaining wall alignment could be encountered during construction, especially when the walls are not in a tangent section. The exception is the use of geosynthetic wrapped faced walls, where alignment with connections is not an issue. However, while there is a potential for stress relief due to extensible reinforcements, very few instrumented structures have been constructed and therefore even in the case of geosynthetic reinforcements K_0 should conservatively be used to calculate the tension in the reinforcements unless numerical modeling is performed to evaluate the anticipated stress state and instrumentation is used to confirm the actual stress conditions.

6.5 SHORED MSE WALLS FOR STEEP TERRAINS AND LOW VOLUME ROADS

In steep terrains MSE wall construction necessitate excavation to establish a flat bench to accommodate the soil reinforcements with a minimum length of greater than 8 ft (2.5 m) or 70% of the height of the wall. Additionally, the required depths of embedment are proportional to the steepness of the slope below the wall toe. In some cases, the excavation required for construction of a MSE wall becomes substantial, and unshored excavation for the MSE wall is not practical, particularly if traffic must be maintained during construction of the MSE wall. Shoring, most often in the form of soil nail walls, has been employed to stabilize the backslope (or back-cut), with a MSE walls being designed and constructed in front of it. Figure 6-10 shows a generic cross-section of this configuration. In this configuration, if the shoring wall is designed as a **permanent** wall it can significantly reduce the long-term lateral pressures on the MSE wall. Such MSE wall configuration is known as a shored MSE or SMSE wall. Details of SMSE walls systems are presented in FHWA-CFL/TD-06-001 (Morrison et al., 2006).

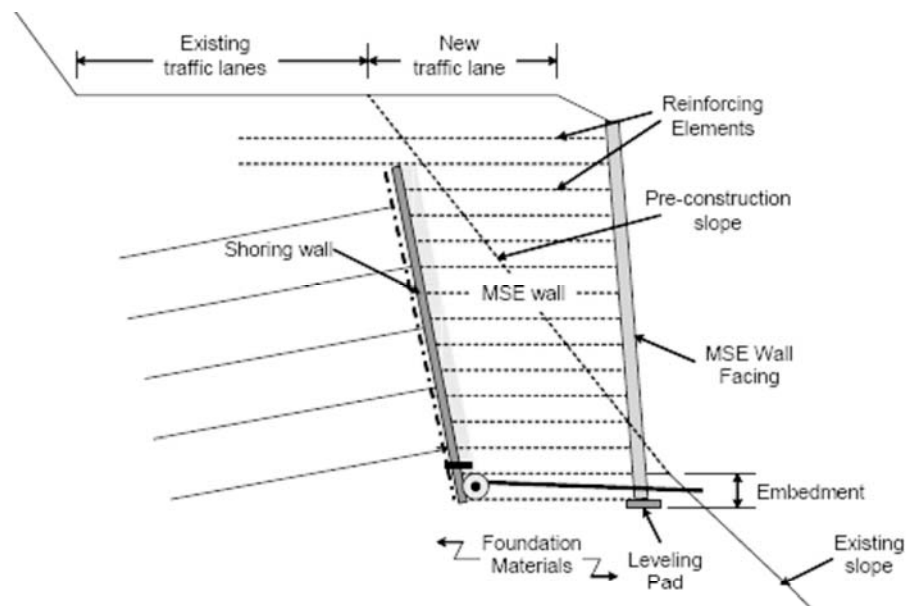
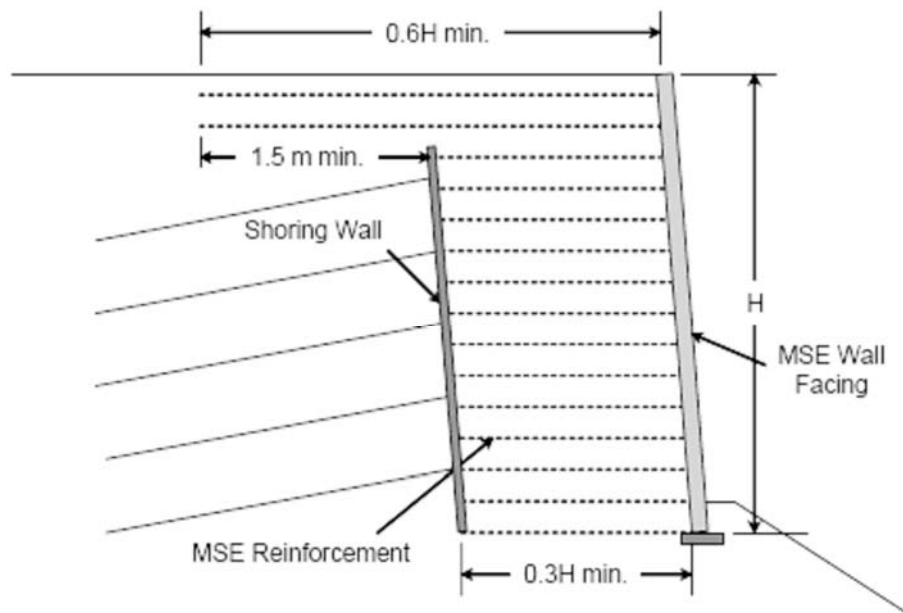


Figure 6-10. Generic cross-section of a shored MSE (SMSE) wall system for steep terrains (Morrison et al., 2006).

For successful implementation of the SMSE walls, the following guidelines should be implemented. These guidelines are valid for static load conditions or in areas where the seismic horizontal accelerations at the foundation level are less than 0.05g. SMSE walls in seismically active areas should be designed based on a more detailed analysis that includes effects of potential non-uniform distribution of seismic and inertial forces within the wall system (both the MSE and the shoring components). Finally, it should be recognized that these walls were developed for low volume roads in mountains and are not recommended in urban areas for roadway widening applications because of the relatively high risk for tension cracks under dynamic effects of traffic at the interface between an existing wall and new MSE wall.

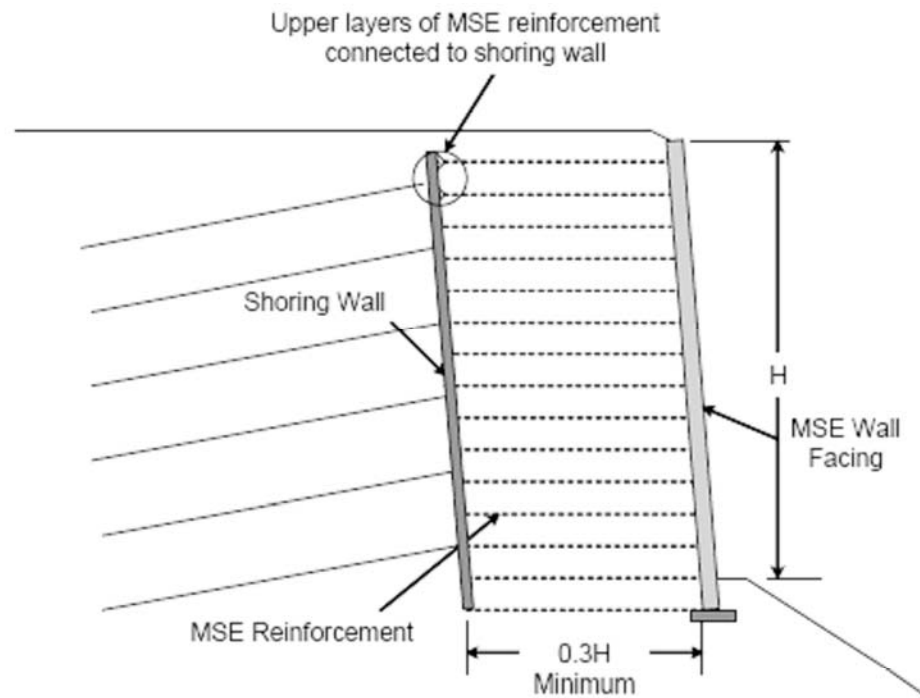
- The shoring wall should be designed as a **permanent** wall whose design life is equal to or greater than that for the MSE wall. For design of shoring systems using ground anchors and soil nail walls, see Sabatini et al. (1999) and Lazarte et al., (2003), respectively.
- Ensure that the drainage features of the MSE wall system and the permanent shoring wall behind it are integrated so that there are no lateral pressures due to hydrostatic conditions in either wall. Note, as discussed in Chapter 5, thin vertical drains behind the face of the soil nail wall do not necessarily fully relieve hydrostatic stress. Thus, some level of hydrostatic stress based on flow net analysis should be included in the design or horizontal drains and should be considered in the design of SMSE walls.
- Figure 6-11 presents the minimum recommended geometry of a SMSE system. The minimum length of the reinforcement is $0.3H$ or 5 ft (1.5 m) whichever is greater. Where adequate construction space is available (or can be made temporarily available with permanent underground easement), it is recommended that the upper two layers of reinforcement are extended to a minimum length of $0.6H$ or a minimum of 5 ft (1.5 m) beyond the shoring wall interface, whichever is greater, as illustrated in Figure 6-11a. This feature limits the potential for tension cracks to develop at the shoring/MSE wall interface, and resists lateral loading effects. Extension of the upper two layers is intended to result in a wall cross-section as depicted in Figure 6-11a, where the height of the shoring wall is at least $2/3$ of the MSE wall height, H . These guidelines should only be applied to wall designs that meet this constraint over the majority of their length. Walls with short shoring walls, i.e., heights less than $2/3H$ over most of their length are outside the scope of these guidelines. It should be noted that near the ends of the retaining wall the height usually tapers, and the shoring wall height may be less than $2/3$ of the MSE height for a short distance. However, application of these guidelines will result in MSE reinforcements not less than 10 ft (3 m) long at the top of the MSE wall (5 ft (1.5 m) minimum *plus* 5 ft (1.5 m) minimum).



(a)

$L_b = \text{Greater of } 0.3H \text{ or } 5 \text{ ft}$

1 m = 3.28 ft



(b)

$L_b = \text{Greater of } 0.3H \text{ or } 5 \text{ ft}$

Figure 6-11. Minimum recommended geometry of a shored MSE wall system in steep terrains, (a) with extension of upper two rows of reinforcement, and (b) with the upper two rows connected to the shoring wall (Morrison et al., 2006).

- Where the shoring wall is less than 2/3 of the height of the MSE wall, as may occur as the wall ends taper, the engineer should check to assure that reinforcement lengths in the upper part of the MSE mass is greater than the conventional 0.7H as discussed in Chapter 4. Generally, this will be satisfied, as long as the total retaining wall height in such sections is less than about 14 ft (4 m).
- If extension of the upper reinforcements is not feasible, a positive mechanical connection between the upper two or more reinforcements and the shoring wall is recommended as shown in Figure 6-11b. Incorporation of interface connections may limit differential movement between the shoring wall and MSE wall components, as a result limiting development of a tension crack, especially if the slack in the MSE reinforcements can be effectively removed. This could potentially be accomplished through the fastening mechanism or by surcharge loading. **Extension of the upper MSE reinforcements is considered superior to mechanical connection of the reinforcements and is recommended by the authors.**
- The critical failure surfaces for SMSE walls with extensible and inextensible reinforcements are presented in Figure 6-12. The critical failure surface is approximated using Rankine's active earth pressure theory within the reinforced soil mass, assuming that the remaining portion lies along the shoring/MSE interface. The critical failure surfaces are consistent with those presented in Chapter 4 (except pullout calculations). Design for internal stability conservatively neglects the additional retaining benefits provided by longer upper reinforcement layers shown in Figure 6-10a or the resistance from connections shown in Figure 6-11b.
- For SMSE walls, lateral pressures are essentially the result of reaction of reinforced soil mass against the shoring wall, and are thus internal to the MSE mass. At each reinforcement level, the horizontal stress, σ_h , along the potential failure line is computed using exactly the procedures in Chapter 4. If superimposed concentrated vertical loads are present then the increment of vertical stress ($\Delta\sigma_v$) maybe computed by a modified version of the 2:1 method as shown in Figure 6-13.

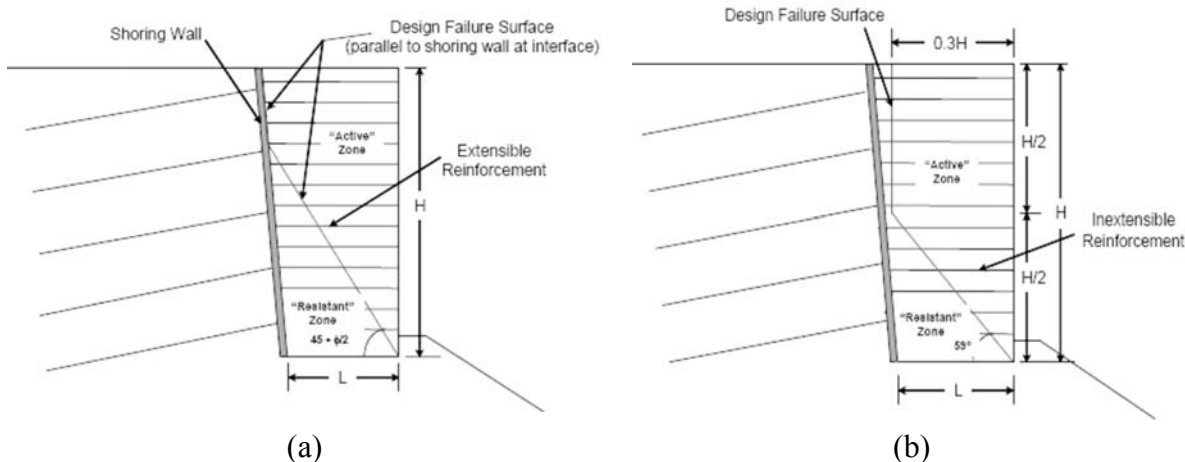
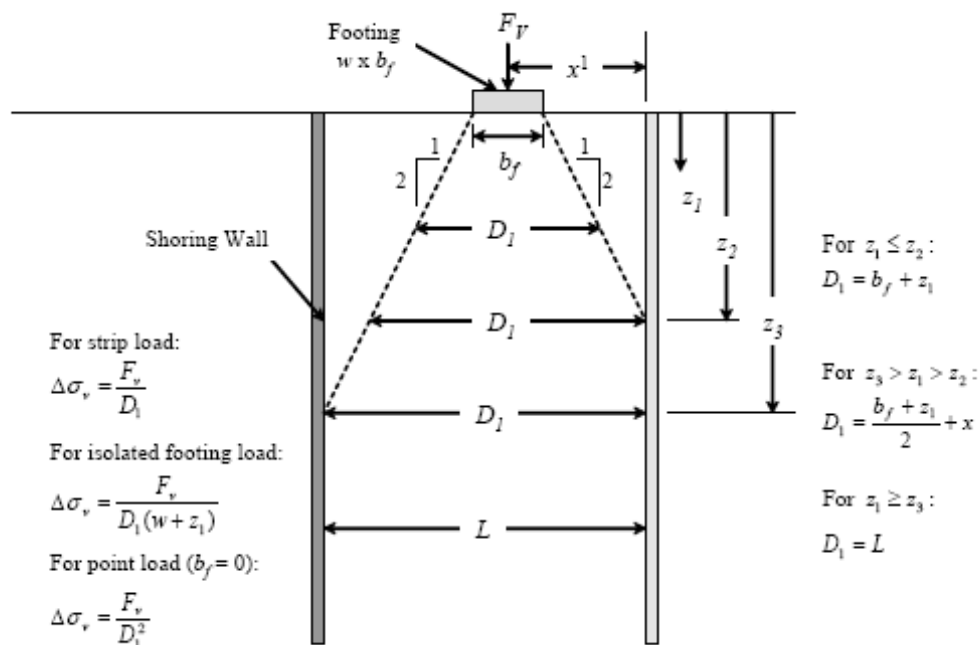


Figure 6-12. Location of potential failure surface for internal stability design of shored MSE walls (a) extensible reinforcements, (b) inextensible reinforcements (Morrison et al., 2006).

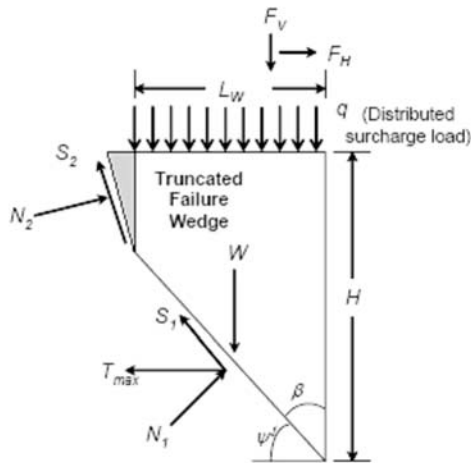


Notes:

1. The measurement of x may be from either the face of the MSE wall or the shoring wall, depending on the location of the load footing and the slopes of the various walls.
2. The figure is applicable to MSE walls constructed without a batter, and where the load footing does not straddle the shoring wall. When wall batters are employed, as is generally recommended, the vertical stresses can be estimated by geometrically calculating D_1 at each reinforcement depth. In the case where the footing straddles the shoring wall, D_1 is always greater than z_2 , as defined in the figure.

Figure 6-13. Distribution of stress from concentrated vertical load for internal and external stability calculations (Morrison et al., 2006).

- Internal design differs from design of a conventional MSE wall with regard to pullout of the reinforcements. Conventional MSE design requires that each layer of reinforcement resist pullout by extending beyond the estimated failure surface as indicated in Chapter 4. In the case of a SMSE wall system, only the lower reinforcement layers (i.e., those that extend into the resistant zone) are designed to resist pullout for the entire “active” MSE mass. The relevant equations of T_{MAX} and pullout resistance are given in Figure 6-14. The tensile resistance of the reinforcement as well as the connection strength is evaluated in accordance with the procedures in Chapter 4.
- External stability design of the MSE component of a SMSE wall should address bearing capacity and settlement of foundation materials based strength limit state and service limit state considerations. Limiting eccentricity (i.e., overturning) and sliding are not included as failure mechanisms due to stabilization provided by the shoring wall. Hydrostatic forces are eliminated by incorporating internal drainage into the design. Procedures for evaluating bearing capacity and settlement analysis are the same as those in Chapter 4.
- As part of the design of the individual MSE wall and shoring components, stability internal to these individual components will have been achieved. However, a global stability evaluation of the SMSE wall system as a compound structure must also be evaluated. Various failure modes are shown in Figure 6-15. Although, all failure five failure modes shown in Figure 6-15 must be evaluated, the most critical failure mechanisms are along the shoring/MSE interface (Mode 4 in Figure 6-15) and global stability external to the SMSE wall system must be evaluated (Mode 1 in Figure 6-15). Morrison et al. (2006) present suggestions for global stability analyses and measures to improve stability. Stability analyses for the SMSE wall system should use conventional (i.e., ASD) limit equilibrium analysis methods. As with any earth stability evaluation, selection of appropriate material parameters is of utmost importance in obtaining a realistic evaluation. In addition, the compound nature of the SMSE wall system requires defining other factors such as drainage issues which affect its behavior.



Notes:

1. For extensible reinforcements, $\psi = 45 + \phi/2$; for inextensible reinforcements, $\psi = 59$ degrees.
2. Assume tension crack development and neglect forces N_2 and S_2 .
3. Assume upper wedge (shown as gray) is in equilibrium.

Case 1: For $L_w < H \tan \beta$

$$T_{\max} = \frac{L_w \left[\gamma \left(H - \frac{L_w}{2 \tan \beta} \right) + q \right] + F_V}{\tan(\phi' + \beta)} + F_H$$

Case 2: For $L_w = 0.3H$

$$T_{\max} = \frac{3H \left[\gamma \left(H - \frac{3H}{20 \tan \beta} \right) + q \right] + F_V}{10 \tan(\phi' + \beta)} + F_H$$

Case 3: For $L_w \geq H \tan \beta$

$$T_{\max} = \frac{H \tan \beta (\gamma H + 2q) + 2F_V}{2 \tan(\phi' + \beta)} + F_H$$

Notes:

1. The loads F_V , F_H and W should be multiplied by the appropriate load factors when evaluating the strength and service limit state load combinations.
2. The pullout resistance of the MSE wall component of a SMSE wall system is considered adequate if $T_{\max} \leq \phi_p \Sigma F_{p0}$ where ΣF_{p0} is the summation of the pullout resistances from all layers of reinforcement based on the length of the reinforcement beyond the active zone and ϕ_p is the resistance factor as follows:
 - a. $\phi_p = 0.90$ for $L/H > 0.4$
 - b. $\phi_p = 0.65$ for $L/H \leq 0.4$

Figure 6-14. Computation for $T_{\max}$ and evaluation of pullout resistance (after Morrison et al., 2006).

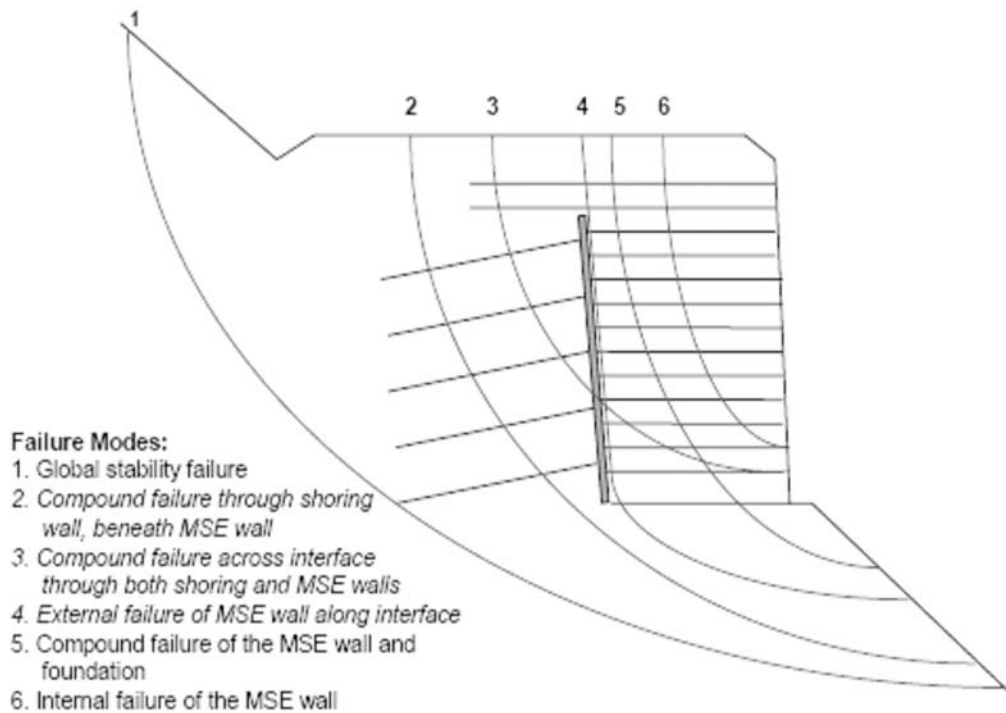


Figure 6-15. Example global stability and compound failure surfaces (Morrison et al., 2006).

6.6 STABLE FEATURE MSE (SFMSE) WALLS

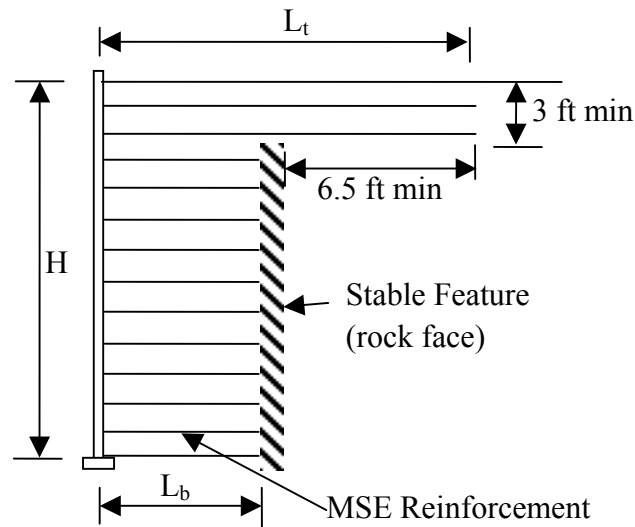
MSE walls can be considered in front of apparently stable features such as a rock face as shown in Figure 6-16. Depending on the space between the MSE wall face and the stable feature, the behavior of the SFMSE wall may be similar to that of a SMSE wall. Following are some guidelines for such cases:

- Establish that the feature behind the proposed SFMSE wall line is stable and will be stable during the design life of the SFMSE wall. The feature should be stabilized to the extent necessary to be compatible with the design life of the SFMSE wall that is being proposed at that particular location.
- Evaluate the deformation and strength behavior of the feature (rock face or existing wall) under additional stresses behind it. Hydrostatic pressure and or other lateral pressures may contribute to the instability of a rock cut in front of which a SFMSE wall is being proposed. The stability analysis should include an evaluation of potential lateral movements under anticipated additional loadings on the existing feature.

- Perform a deformation analysis of the foundation under the SFMSE wall and evaluate the effect of the estimated deformations on the facilities above the top of the wall and in particular at and immediately above the interface between the existing feature and the SFMSE wall.
- Evaluate the effect of the increased stresses at the base of the MSE wall on the settlement of the existing feature. If the existing feature is a retaining wall then it might experience detrimental settlement in the immediate and long-term as well as downdrag forces at the interface between the MSE wall and the existing feature.
- Ensure that the drainage features of the SFMSE wall system and the stable feature behind it are integrated so that there are no lateral pressures due to hydrostatic conditions.
- For SFMSE wall systems, the configuration in Figure 6-16 is recommended wherein at least the top two reinforcements are extended over the top of the stable feature rather than being mechanically connected to the stable feature. For roadway widening projects where the stable feature may be an existing wall, it is recommended that the top of the wall be trimmed as necessary to accommodate the top layers of reinforcements and mitigate long-term maintenance issues.
- Extend all soil reinforcement layers above the top of the stable feature a distance back of L_t , per Figure 6-15, with a minimum of two layers as previously noted.
- Establish the reinforcement layout based on the T_{MAX} values obtained using the guidance provided in Figure 6-14 and other guidance provided in Section 6.5. The minimum clearance between the top of the stable feature and the reinforcement layer above it should be 6 in. (150 mm) to prevent adverse stress concentrations in this area and contact between dissimilar materials.
- Global stability analysis should be performed as the MSE wall will increase driving forces. Global analysis is especially needed where structures are constructed with a slope at the toe or on soft ground. All failure modes similar to those shown in Figure 6-15 should be evaluated.

The above guidelines are valid for static load conditions or in areas where the seismic horizontal accelerations at the foundation level are less than 0.05g. SFMSE walls in seismically active areas should be designed based on a more detailed analysis that includes effects of potential non-uniform distribution of seismic and inertial forces within the wall system (both the MSE and the stable feature components). Finally, these types of walls are not recommended in urban areas for

cases such as roadway widening because of the relatively high risk for tension cracks under dynamic effects of traffic at the interface between the existing feature such as a wall and new MSE wall.



$$1 \text{ m} = 3.28 \text{ ft}$$

$$L_t = 0.8 H \text{ min}; L_b = \text{Greater of } 0.3H \text{ or } 5 \text{ ft}$$

Figure 6-16. Minimum recommended geometry of a stable feature MSE (SFMSE) wall system.

CHAPTER 7

DESIGN OF MSE WALLS FOR EXTREME EVENTS

As per AASHTO (2007) an extreme event is one whose recurrence interval can be thought to exceed design life. AASHTO (2007) has two limit states to deal with such events. These limit states are labeled Extreme Event I and Extreme Event II. In the context of MSE walls, the extreme events with the applicable limit state shown in parentheses that require consideration in the design process are as follows:

- Seismic events (Extreme Event I)
- Vehicular impact events (Extreme Event II)
- Superflood events and scour (Extreme Event II)

This chapter addresses each of the above extreme events along with a review of the applicable limit state, i.e., Extreme Event I or Extreme Event II.

7.1 SEISMIC EVENTS

Seismic events are analyzed under Extreme Event I limit state as per AASHTO (2007¹). Seismic events tend to affect both external and internal stability of MSE walls. Guidance for seismic analysis presented in this section is based on Anderson et al. (2008) and Kavazanjian (2009) and represents updated procedures to those in AASHTO (2007).

7.1.1 External Stability

The external stability uses a displacement based approach. The recommended design methodology is presented in the following steps.

- Step 1** Establish an initial wall design based on static loading using information in Chapters 4, 5 and 6.
- Step 2** Establish the seismic hazard using Article 3.10.2 of AASHTO (2007). Using the 1,000-yr return period seismic hazard maps in AASHTO (2007), estimate the following site-specific values:
- The site peak ground acceleration (PGA), and
 - Spectral acceleration at 1-second, S_1

¹ AASHTO 4th Edition 2007 including 2008 and 2009 Interims. 2008 Interims contain significant seismic revisions.

Step 3 For the project under consideration, establish the Site Effects in accordance with Article 3.10.3 of AASHTO (2007). This includes the determination of Site Class as per Article 3.10.3.1 of AASHTO (2007) and Site Factors, F_{pga} and F_v from Tables 3.10.3.2-1 and 3.10.3.2-3, respectively, of AASHTO (2007). The procedure described herein is applicable to Site Classes A, B, C, D and E. For all sites in Site Class F, site-specific geotechnical investigations and dynamic site response analysis should be performed.

Step 4 Determine the maximum accelerations, k_{max} , and peak ground velocity (PGV) as follows:

$$k_{max} = F_{pga} (PGA) \quad (7-1)$$

$$PGV \text{ (in/sec)} = 38F_v S_1 \quad (7-2)$$

where F_{pga} and F_v are site factors determined in Step 3 and PGA and S_1 are site peak ground acceleration and spectral acceleration at the 1-second period, respectively, as obtained in Step 2.

Step 5 Using a wall height dependent reduction factor, α , obtain an average peak ground acceleration, k_{av} , within the reinforced soil zone as follows:

$$k_{av} = \alpha k_{max} \quad (7-3)$$

where the value of α is based on the Site Class of the foundation soils as follows:

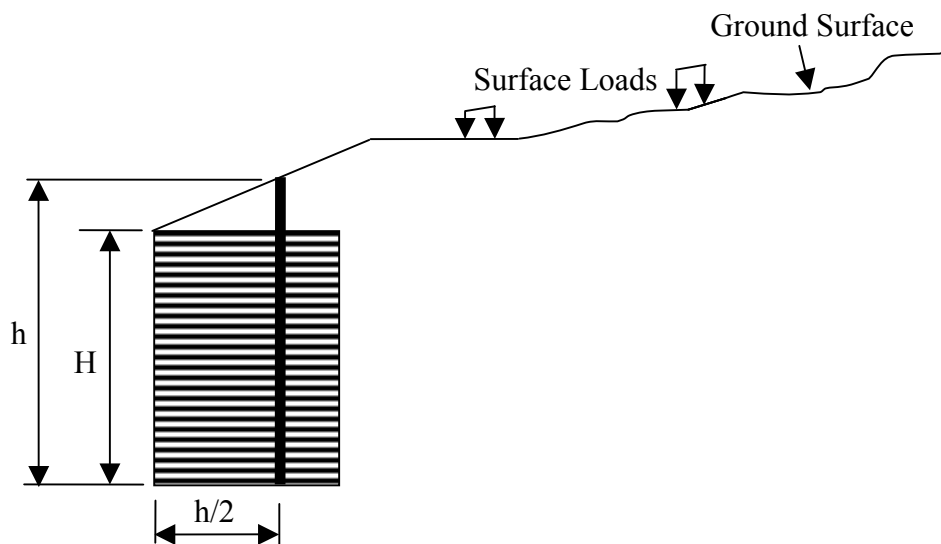
- For Site Class C, D and E (i.e., soils)

$$\alpha = 1 + 0.01H \left[0.5 \left(\frac{F_v S_1}{k_{max}} \right) - 1 \right] \quad (7-4)$$

where H is the wall height in feet at the wall face as shown in Figure 7-1.

- For Site Class A and B foundation conditions (i.e., hard and soft rock), the values of α determined by Equation 7-4 should be increased by 20 percent.

For practical purposes, walls less than approximately 20 ft in height and on very firm ground conditions (i.e., Site Class B or C), $k_{av} \approx k_{max}$. For wall heights greater than 100 ft, site-specific geotechnical investigations and dynamic site response analysis should be performed.



$$h = H + \frac{\tan I(0.5H)}{(1 - 0.5 \tan I)} ; \text{ where } I \text{ is the backfill slope angle}$$

$h/2$ is measured from back of wall facing

Figure 7-1. Definition of heights for seismic analyses.

Step 6 Determine the total (static + dynamic) thrust P_{AE} using one of the following two methods:

Method 1: Mononobe-Okabe (M-O) formulation

$$P_{AE} = 0.5(K_{AE})\gamma_b h^2 \quad (7-5)$$

where h is the wall height along the vertical plane within the reinforced soil mass as shown in Figure 7-1, γ_b is the unit weight of the retained fill and K_{AE} is obtained as follows:

$$K_{AE} = \frac{\cos^2(\phi'_b - \xi - 90 + \theta)}{\cos \xi \cos^2(90 - \theta) \cos(\delta + 90 - \theta + \xi) \left[1 + \sqrt{\frac{\sin(\phi'_b + \delta) \sin(\phi'_b - \xi - I)}{\cos(\delta + 90 - \theta + \xi) \cos(I - 90 + \theta)}} \right]^2} \quad (7-6)$$

where,

$\xi = \tan^{-1} \left(\frac{k_h}{1 - k_v} \right)$ with k_h = horizontal seismic coefficient and k_v = vertical seismic coefficient

δ = angle of wall friction = lesser of the angle of friction for the reinforced soil mass (ϕ'_r) and the retained backfill (ϕ'_b)

I = the backfill slope angle = β (see Figure 4-3) {Note: use GLE for broken back slopes, see Comment 2 below}

ϕ'_b = angle of internal friction for retained backfill

θ = the slope angle of the face (see Figure 4-5 in Chapter 4)

To use the Mononobe-Okabe formulation, two seismic coefficient, k_h and k_v , must be defined. It is assumed that these coefficients are applied simultaneously and uniformly to all parts of the structure, i.e., to the reinforced and retained fill. Typically, the vertical seismic coefficient, k_v , is assumed to be zero. The horizontal seismic coefficient, k_h is taken to be equal to k_{max} determined in Step 2.

The total thrust, P_{AE} , calculated as per Equation 7-5 is assumed to act at $h/2$, i.e., mid-height of the vertical plane of height h shown in Figure 7-1. Therefore, the stress due to thrust P_{AE} is assumed to be distributed uniformly over the height h .

Comments on use of M-O formulation:

1. For backfills that are sloped at 3H:1V or steeper, it may not be possible to obtain a solution for a certain combination of variables in the M-O formulation. This is because the term $\sin(\phi - \xi - I)$ in Equation 7-6 may become negative and represents a limiting condition since at $I = \phi - \xi$ an unstable slope condition occurs (i.e., FS=1 wherein the failure surface coincides with the slope surface. As the limiting condition is approached the earth pressures based on M-O formulation become unrealistically large.
2. M-O formulation is strictly applicable to homogeneous cohesionless soils and may not yield realistic solutions for more complex cases involving (a) soils which derive their shear strength from both cohesion and friction, i.e., $c-\phi$ soils, (b) non-uniform backslope profiles, and (c) complex surface loadings.

For the cases where M-O formulation leads to unrealistic results, it is recommended that numerical procedures using the same principles of M-O formulation may be used, such as the well-known graphical Culmann method or Coulomb's trial wedge method. However, the more versatile approach for such cases is to utilize the conventional slope stability programs as described in Method 2.

Method 2: Generalized Limit Equilibrium (GLE) slope stability

- a. Define the wall geometry, nominal surface loadings (i.e., loadings with load factor = 1.0), groundwater profile, and design soil properties. The plane where the earth pressure needs to be calculated should be modeled as a free boundary. This boundary is a vertical plane located at a distance of $h/2$ from the back of the wall facing as shown in Figure 7-2.
- b. Choose an appropriate slope stability analysis method. Spencer's method generally yields good results because it satisfies the equilibrium of forces and moments.
- c. Choose an appropriate sliding surface search scheme, e.g., circular, linear, bi-linear, block, etc.
- d. For seismic analysis, use $k_h = k_{av}$ and $k_v = 0$.
- e. Apply the earth pressure as a boundary force, P_{AE} , on the face of vertical plane of height h as shown in Figure 7-2. The angle of the applied force with respect to horizontal depends on assumed friction angle between the wall and soil which is lesser of the angle of friction for the reinforced soil mass (ϕ'_r) and the retained backfill (ϕ'_b). Different application points between $h/3$ and $2h/3$ from the base need to be examined to determine the maximum value of P_{AE} . Change the magnitude of the applied load until a capacity:demand ratio (CDR) of 1.0 is obtained i.e., the load and the resistance are balanced. Thus, the force corresponding to a CDR of 1.0 is equal to the total thrust on the retaining structure.
- f. Verify design assumptions and material properties by examining the loads on individual slices in the output.
- g. Once the maximum value of total thrust, P_{AE} , is determined, apply the force at mid height ($h/2$) as shown in Figure 7-2 for analysis in following steps.

Step 7. Determine the horizontal inertial force, P_{IR} , of the total reinforced wall mass as follows:

$$P_{IR} = 0.5(k_{av})(W) \quad (7-7)$$

where W is the weight of the full reinforced soil mass and any overlying permanent slopes and/or permanent surcharges within the limits of the reinforced soil mass. The inertial force is assumed to act at the centroid of the mass used to determine the weight W .

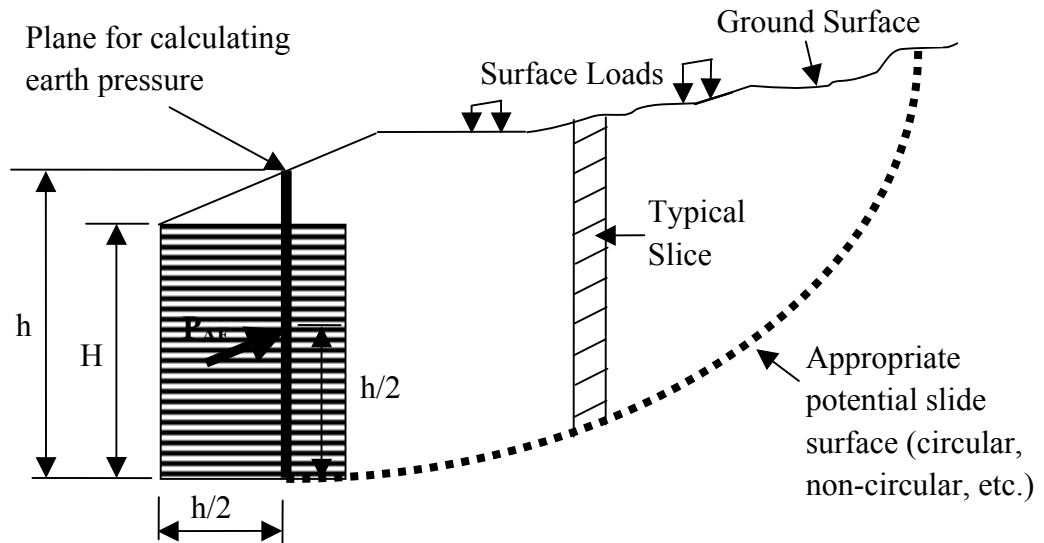


Figure 7-2. Use of a slope stability approach to compute seismic earth pressure.

Step 8 Check the sliding stability using a resistance factor, ϕ_τ , equal to 1.0 and the full, nominal weight of the reinforced zone and any overlying permanent surcharges. If the sliding stability is met, the design is satisfactory and go to Step 11. If not, go to Step 9.

Compute the total horizontal force, T_{HF} , is as follows:

- For M-O method:

$$T_{HF} = \text{Horizontal component of } P_{AE}(\cos \delta) + P_{IR} + \gamma_{EQ}(q_{LS})K_{AE}H + \text{other horizontal nominal forces due to surcharges (with load factor = 1.0)}$$

where, γ_{EQ} is the load factor for live load in Extreme Event I limit state and q_{LL} is the intensity of the live load surcharge.

- For GLE method:

$$T_{HF} = \text{Horizontal component of } P_{AE} \text{ (since all surcharges are included in the slope stability analysis)} + P_{IR}$$

Compute the sliding resistance, R_τ , as follows:

$$R_\tau = \Sigma V (\mu)$$

where μ is the minimum of $\tan\phi'_r$, $\tan\phi'_f$ or (for continuous reinforcement) $\tan\phi$ as discussed in Section 4.5.6.a and ΣV is the summation of the vertical forces as follows:

$$\Sigma V = W + P_{AE}\sin\delta + \text{permanent nominal surcharge loads within the limits of the reinforced soil mass}$$

The sliding stability capacity to demand ratio (CDR) is calculated as follows:

$$CDR_{\text{sliding}} = R_{\tau} / T_{HF}$$

If $CDR_{\text{sliding}} > 1$, the design is satisfactory and go to Step 11 otherwise go to Step 9.

Step 9 Determine the wall yield seismic coefficient, k_y , where wall sliding is initiated. This coefficient is obtained by iterative analysis as follows:

- Determine values of P_{AE} as a function of the seismic coefficient k ($< k_{\max}$) as shown in Figure 7-3a.
- Determine horizontal driving and resisting forces as a function of k (using spreadsheet calculations) and plot as a function of k as shown in Figure 7-3b. The value of k_y corresponds to the point where the two forces are equal, i.e., the CDR against sliding equals 1.0.

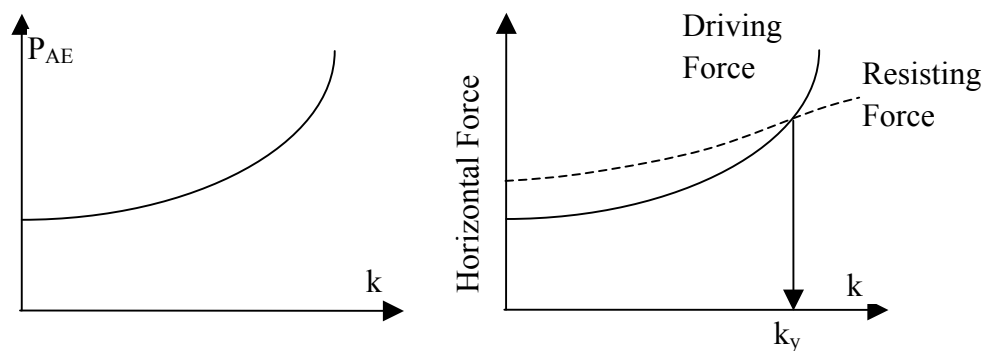


Figure 7-3. Procedure for determination of k_y (Anderson et al., 2008).

Step 10 Determine the wall sliding displacement, d , in inches based on the following relationships between d , k_y/k_{max} , k_{max} , and PGV based on whether the site is located in Western United States (WUS) or Central and Eastern United States (CEUS) as per Figure 7-4:

- For WUS soil and rock sites and CEUS soil sites

$$\log(d) = -1.51 - 0.74\log(k_y/k_{max}) + 3.27\log(1 - k_y/k_{max}) - 0.80\log(k_{max}) + 1.59\log(\text{PGV}) \quad (7-8)$$

- For CEUS rock sites

$$\log(d) = -1.31 - 0.93\log(k_y/k_{max}) + 4.52\log(1 - k_y/k_{max}) - 0.46 \log(k_{max}) + 1.12\log(\text{PGV}) \quad (7-9)$$

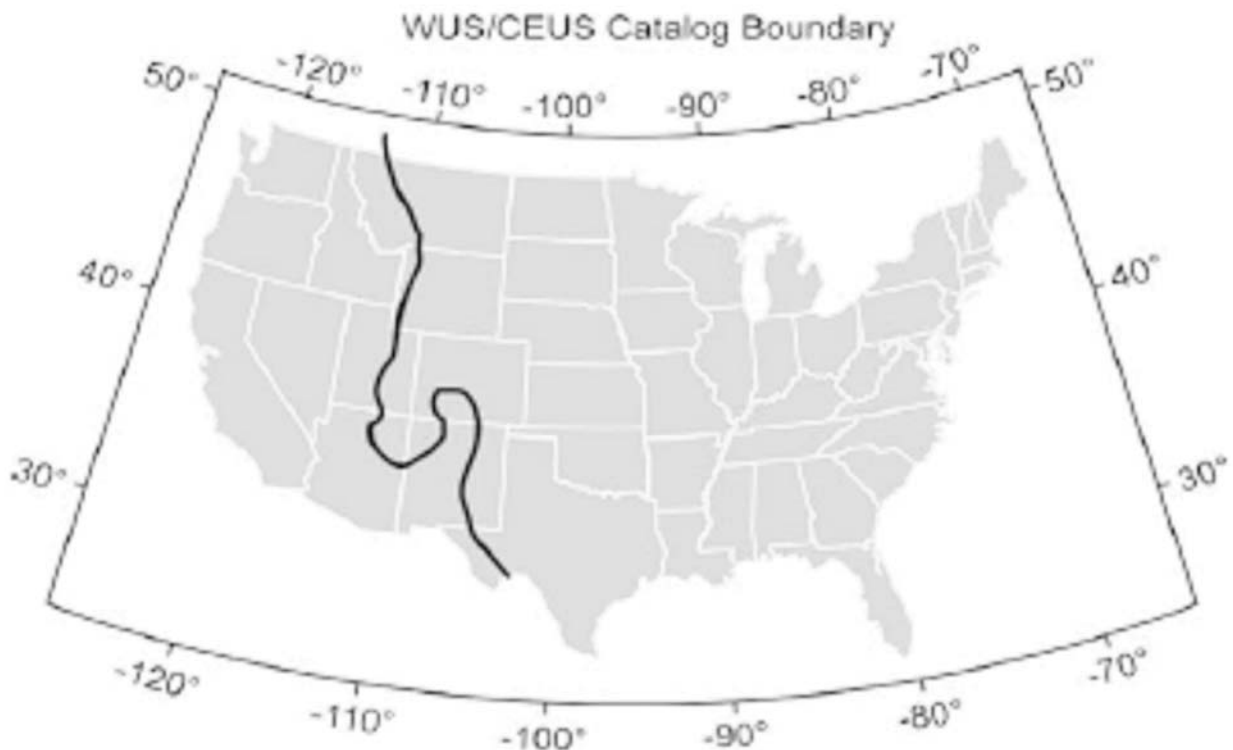


Figure 7-4. Boundary between WUS and CEUS (Anderson et al. 2008).

Step 11 Evaluate the limiting eccentricity and bearing resistance using the same principles discussed in Chapter 4. Include all applicable loads for Extreme Event I. If M-O method is used then add other applicable forces to P_{AE} . If GLE method is used then no additional forces need to be added to P_{AE} since the slope stability analysis includes all applicable forces. Check the limit states using the following criteria:

1. For limiting eccentricity, for foundations on soil and rock, the location of the resultant of the applicable forces should be within the middle two-thirds of the wall base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths of the wall base for $\gamma_{EQ} = 1.0$. Interpolate linearly between these values as appropriate.
2. For bearing resistance compare the effective uniform bearing pressure to the nominal bearing resistance that is based on the full width of the reinforced zone. A resistance factor of 1.0 is used per Article 10.5.5.3.3 (AASHTO, 2007).

Step 12 If Step 11 criteria are not met, adjust the wall geometry and repeat Steps 6 to 11 as needed.

Step 13 If Step 11 criteria are met, assess acceptability of sliding displacement, d . The amount of displacement which is tolerable will depend on the nature of the wall and what it supports, as well as what is in front of the wall. Typical practice is to limit the lateral displacement in the range of 2.0 in. (50 mm) to 4.0 in (100 mm) assuming that structures on top or at toe of the wall can tolerate such displacements.

7.1.2 Internal Stability

For internal stability, the active wedge is assumed to develop an internal dynamic force, P_i , that is equal to the product of the mass in the active zone and the wall height dependent average seismic coefficient, k_{av} . Thus, P_i is expressed as follows:

$$P_i = k_{av} W_a \quad (7-10)$$

where W_a is the soil weight of the active zone as shown by shaded area in Figure 7-5 and k_{av} given by Equation 7-3. The force P_i is assumed to act as shown in Figure 7-5. If the weight of the facing is significant then include it in W_a computation.

The supplementary inertial force, P_i , will lead to dynamic increases in the maximum tensile forces in the reinforcements. Reinforcements should be designed to withstand horizontal forces generated by the internal inertia force, P_i , in addition to the static forces. During the internal stability evaluation, it is assumed that the location and the maximum tensile force lines do not change during seismic loading.

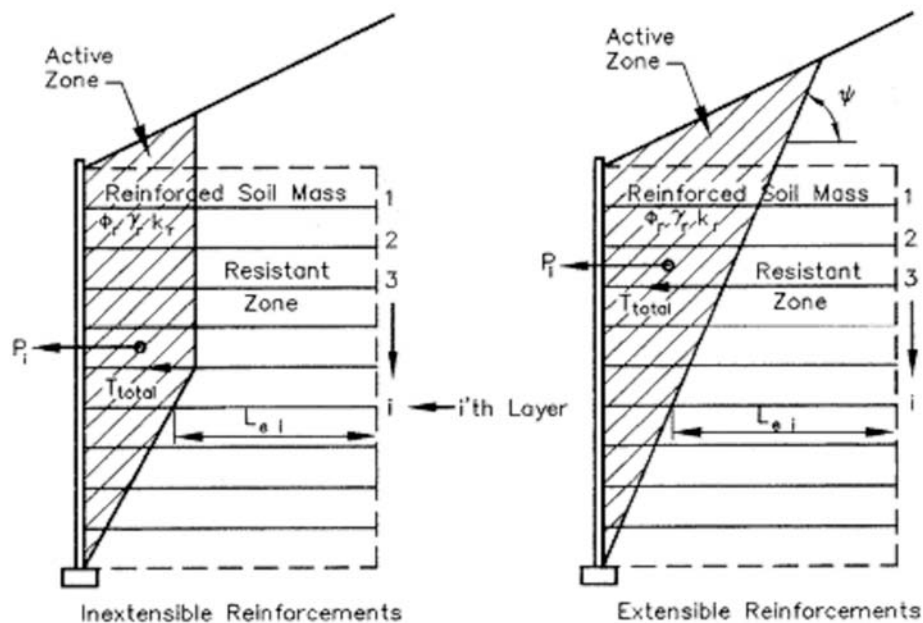


Figure 7-5. Seismic internal stability of a MSE wall.

The inertial force is distributed to the reinforcements equally as follows:

$$T_{md} = \frac{P_i}{n} \quad (7-11)$$

where:

- T_{md} = factored incremental dynamic inertia force at layer i
- P_i = internal inertia force due to the weight of backfill within the active zone, i.e., the shaded area in Figure 7-5
- n = number of soil reinforcement layers within the reinforced soil zone,

The load factor for seismic forces is equal to 1.0. Therefore, the total factored load applied to the reinforcement on a load per unit of wall width basis is determined as follows:

$$T_{total} = T_{max} + T_{md} \quad (7-12)$$

where T_{max} is the factored static load applied to the reinforcements determined using the appropriate equations in Chapters 4 and 6. The reinforcement must be designed to resist the dynamic component of the load at any time during its design life. This includes consideration of both tensile and pullout failures as discussed next.

7.1.2.a Tensile Failure

Design for static loads requires the strength of the reinforcement at the end of the design life to be reduced to account for corrosion for metallic reinforcement, and for creep and other degradation mechanisms for geosynthetic reinforcements. The adjustment for metallic corrosion losses are exactly the same described in Chapter 4 for static analysis. For metallic reinforcements, use the following resistance factors while evaluating tensile failure under combined static and earthquake loading (per Table 11.5.6-1 of AASHTO {2007}):

- Strip reinforcements: 1.00
- Grid reinforcements: 0.85

In contrast, the procedures for geosynthetic do not require a creep reduction for the short duration seismic loading condition and only reductions for geosynthetic degradation losses are required. Strength loss in geosynthetics due to creep requires long-term, sustained loading. The dynamic component of load for seismic design is a transient load and does not cause strength loss due to creep. Therefore, the resistance of the reinforcement to the static component of load, $T_{\max}$, must be handled separately from the dynamic component of load, T_{md} . The strength required to resist $T_{\max}$ must include the effects of creep, but the strength required to resist T_{md} should not include the effects of creep. Thus, for geosynthetic reinforcement rupture, the reinforcement is designed to resist the static and dynamic components of the load determined as follows:

For the static component:

$$S_{\text{rs}} \geq \frac{T_{\max} \text{RF}}{\phi R_c} \quad (7-13)$$

For the dynamic component:

$$S_{\text{rt}} \geq \frac{T_{\text{md}} \text{RF}_{\text{ID}} \text{RF}_{\text{D}}}{\phi R_c} \quad (7-14)$$

where:

- ϕ = resistance factor for combined static/earthquake loading = 1.20 from Table 11.5.6-1 of AASHTO (2007)
- S_{rs} = ultimate reinforcement tensile resistance required to resist static load component
- S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component
- R_c = reinforcement coverage ratio

- RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep, and chemical aging, equal to $RF_{CR} \times RF_{ID} \times RF_D$ (see Chapter 3)
- RF_{ID} = strength reduction factor to account for installation damage to reinforcement
- RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation

Using the above equations, the required ultimate tensile resistance of the geosynthetic reinforcement is determined as follows:

$$T_{ult} = S_{rs} + S_{rt} \quad (7-15)$$

7.1.2.b Pullout Failure

For pullout of steel or geosynthetic reinforcement, the following equation is used:

$$L_e \geq \frac{T_{total}}{\phi(0.8F^* \alpha \sigma_v CR_c)} \quad (7-16)$$

where:

- L_e = length of reinforcement in resisting zone
- T_{total} = maximum factored reinforcement tension from Equation 7-12
- ϕ = resistance factor for reinforcement pullout = 1.20 from Table 11.5.6-1 of AASHTO (2007)
- F^* = pullout friction factor
- α = scale effect correction factor
- σ_v = unfactored vertical stress at the reinforcement level in the resistant zone
- C = overall reinforcement surface area geometry factor
- R_c = reinforcement coverage ratio

For seismic loading conditions, the value of F^* , the pullout resistance factor, is reduced to 80 percent of the value used for static design, unless dynamic pullout tests are performed to directly determine the F^* value.

7.1.3 Facing Reinforcement Connections

Facing elements are designed to resist the total (static + seismic) factored load, i.e., T_{total} . Facing elements should be designed in accordance with applicable provisions of Sections, 5, 6, and 8 of AASHTO (2007) for reinforced concrete, steel, and timber, respectively.

For segmental concrete block faced walls, the blocks located above the uppermost reinforcement layer should be designed to resist toppling failure during seismic loading.

For geosynthetic connections subjected to seismic loading, the factored long-term connection strength, ϕT_{ac} , must be greater than T_{total} (i.e., $T_{max} + T_{md}$). If the connection strength is partially or fully dependent on friction between the facing blocks and the reinforcement (e.g., MBW facing), the connection strength to resist seismic loads should be reduced to 80 percent of its static value as follows:

For the static component of the load:

$$S_{rs} \geq \frac{T_{max} RF_D}{0.8 \phi (CR_{cr}) R_c} \quad (7-17)$$

For the dynamic component of the load:

$$S_{rt} \geq \frac{T_{md} RF_D}{0.8 \phi (CR_u) R_c} \quad (7-18)$$

where:

- S_{rs} = ultimate reinforcement tensile resistance required to resist static load component
- T_{max} = applied load to reinforcement
- RF_D = reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Chapter 3
- ϕ = resistance factor = 1.20 applied to both the static and the dynamic components, from Table 11.5.6.4-1 of AASHTO (2007)
- CR_{cr} = long-term connection strength reduction factor to account for reduced ultimate strength resulting from connection
- R_c = reinforcement coverage ratio
- S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component
- T_{md} = factored incremental dynamic inertia force
- CR_u = short-term reduction factor to account for reduced ultimate strength resulting from connection.

For mechanical connections that do not rely on a frictional component, the 0.8 multiplier is removed from Equations 7-17 and 7-18.

The required ultimate tensile resistance of the geosynthetic reinforcement at the connection is:

$$T_{\text{ult-conn}} = S_{\text{rs}} + S_{\text{rt}} \quad (7-19)$$

The connection capacity of a facing/reinforcement connection system that is fully dependent on the shear resisting devices for the connection capacity will not be significantly influenced by the normal stress between facing blocks. The percentage of connection load carried by the shear resisting devices relative to the frictional resistance to meet the specification requirements should be determined based on past successful performance of the connection system.

For cases where seismic analysis is required as per Section 4 of AASHTO, facing connections in MBW unit faced walls should use shear resisting devices between the facing blocks and soil reinforcement such as shear keys and structural pins (i.e., pins manufactured from material meeting the design life of the structure, e.g., steel and HDPE) and should not be fully dependent on frictional resistance between the soil reinforcement and facing blocks.

For steel reinforcement connections, AASHTO (2007) recommends that the resistance factors for combined static and seismic loads as follows:

- Strip reinforcements: 1.00
- Grid reinforcements: 0.85

7.2 VEHICULAR IMPACT EVENTS

Traffic railing impact loads are analyzed under Extreme Event II limit state as per Article A13.2 (AASHTO, 2007). Traffic railing impact events tend to affect only the internal stability of MSE walls. Guidance for traffic barrier analysis presented in this section is based on NCHRP 22-20 (Bligh et al., 2009), which is an extension of the previous FHWA (Elias et al., 2001) method based on laboratory and full-scale field tests. Guidance for post and beam railings is based upon AASHTO (2007).

7.2.1 Traffic Barriers

The impact traffic load on barriers constructed over the front face of MSE walls, must be designed to resist the overturning moment by their own mass per Article 11.10.10.2 (AASHTO, 2007).

Static Impact Load

The recommended static impact force is 10,000 lb (45 kN) applied on a barrier with a minimum height of 32 in. (810 mm) above the roadway. Bligh et al. (2009) found that a 10,000 lbs (45 kN) static impact load is equivalent to a dynamic TL-4 railing test level of 54,000 lb (240 kN), as illustrated in Figure 7-6.

The wall design should ensure that the reinforcement does not rupture or pullout during the impact event. Where the impact barrier moment slab is cast integrally with a concrete pavement, the additional force may be neglected. The recommended static impact forces for rupture and for pullout are based upon the recent NCHRP 22-20 project (Bligh et al., 2009) and past practice.

Load Combination and Load Factors

The load factors and load combination for an Extreme Event II are summarized in Table 4-1. A load factor, $\gamma_{P-EV} = 1.35$ is used for the static soil load. The traffic surcharge, modeled as an equivalent soil height of 2 ft, also uses the load factor $\gamma_{P-EV} = 1.35$ (and not $\gamma = 0.50$), for internal stability analysis. The static equivalent impact loads are multiplied by a load factor, $\gamma_{CT} = 1.00$.

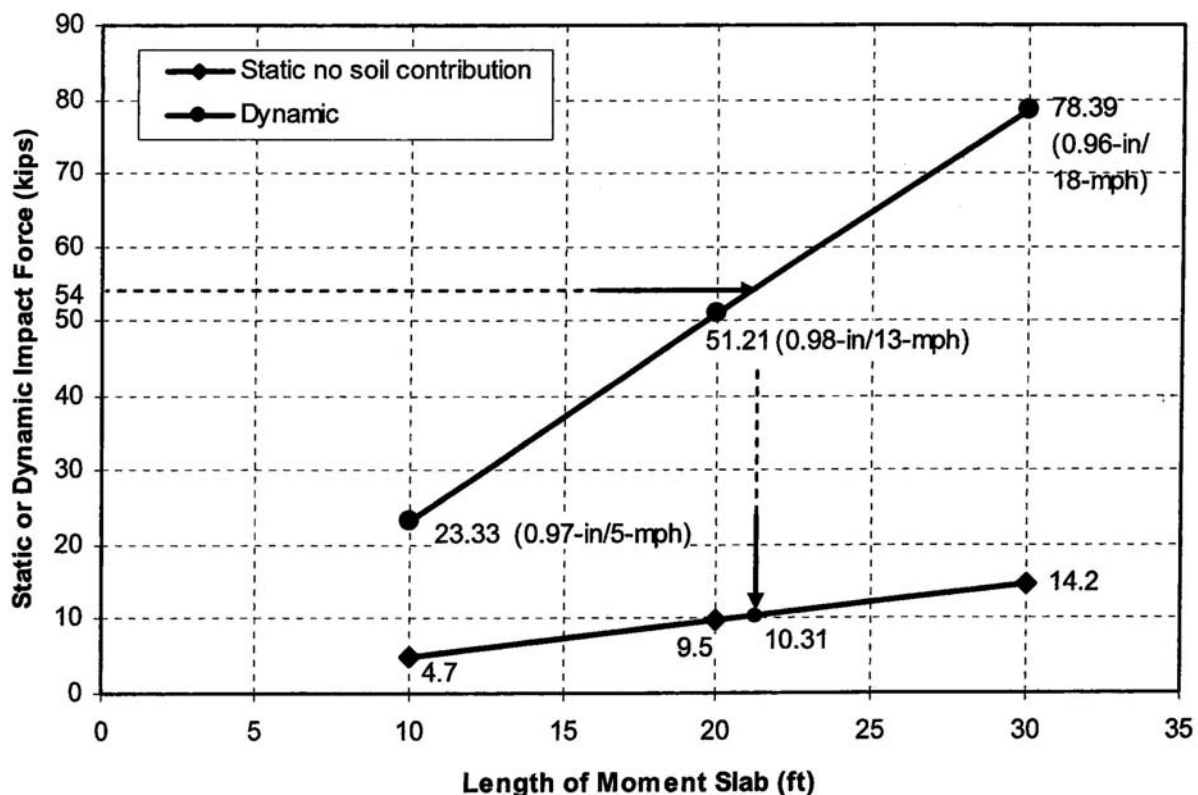


Figure 7-6. Comparison of static and dynamic impact force with 1-inch (25 mm) maximum displacement (Bligh et al., 2009). (1 kip = 4.44 kN; 1 ft = 0.3 m)

Reinforcement Rupture

The static impact force, adds an additional horizontal force to the upper 2 layers of soil reinforcement. It is recommended that the upper layer of soil reinforcement be designed for a rupture impact load equivalent to a static load of 2,300 lb/ft (33.5 kN/m) of wall; and the second layer be designed with a rupture impact load equivalent to a static load of 600 lb/ft (8.8 kN/m). A distribution of stresses, as discussed in Article 11.101.10.2 and illustrated in Figure 3.11.6.3-2 (AASHTO, 2007), is not recommended.

The load factor for impact is equal to 1.0. Therefore, the total factored load applied to the reinforcement on a load per unit of wall width basis is determined as follows:

$$T_{\text{total}} = T_{\text{max}} + T_I \quad (7-20a)$$

where:

T_I = factored impact load at layer 1 or 2, respectively

T_{MAX} = reinforcement tension from static earth and traffic loads

With terms defined, this equation is:

$$T_{\text{total}} = S_v K_r \gamma_r [(Z + h_{\text{eq}}) \gamma_{\text{EV-MAX}}] + t_i (\gamma_{\text{CT}}) \quad (7-20b)$$

where:

t_i = equivalent static load for impact load at layer i , ($t_1 = 2,300$ lb/ft and $t_2 = 600$ lb/ft)
and other terms as previously defined (Chapter 4 and/or 7).

An example calculation is presented in Appendix E.6. Note that for geosynthetic reinforcements, the nominal strength used to structurally size the reinforcements to resist the impact load is not increased by eliminating the reduction factor for creep, as was done for internal seismic design in Section 7.2.1. This is recommended because full-scale traffic barrier impact testing with geosynthetic soil reinforcement has not been performed to date.

Reinforcement Pullout

The pullout resistance of the soil reinforcement to the impact load is resisted over the full-length of the reinforcements (i.e., L). The traffic surcharge, modeled as an equivalent soil height of 2 ft, is included in the nominal vertical stress, σ_v , for pullout resistance calculation. Pullout is resisted over a greater length of wall than the reinforcement rupture loads. Therefore, for pullout, it is recommended that the upper layer of soil reinforcement be designed for a pullout impact load

equivalent to a static load of 1,300 lb/ft (19.0 kN/m) of wall; and the second layer be designed with a pullout impact load equivalent to a static load of 600 lb/ft (8.8 kN/m).

Resistance Factors for Tensile and Pullout Resistance

The resistance factors presented in Table 4-7 for “Combined static/traffic barrier impact” are recommended for Extreme Event II impact loading. (Note that AASHTO does not specifically address tensile resistance factors for impact loading.) The tensile and connection rupture resistance factors are a function of the type of reinforcement.

A pullout resistance factor of 1.00 is recommended for metallic and geosynthetic reinforcements. (Note that AASHTO does not specifically address pullout resistance factors for impact loading.)

Barrier, Coping, and Moment Slab Design

Example traffic barriers are illustrated in Figure 5-2. Typically, the base slab length is 20 ft (6 m) and jointed to adjacent slabs with shear dowels. Parapet reinforcement shall be designed in accordance with AASHTO Section 13 Railings. See NCHRP 22-20 report (Bligh et al., 2009) for barrier, coping, and moment loading recommendations. The anchoring slab shall be strong enough to resist the ultimate strength of the standard parapet, and sized to provide adequate resistance to sliding and overturning.

MSE Facing Panel Design

The upper facing panel must be separated from the barrier slab with 1 to 2 in. (25 to 50 mm) of expanded polystyrene (see Figure 5-2(b)). The distance should be adequate to allow the barrier and slab to resist the impact load in sliding and overturning without loading the facing panel. Separation between the precast facing and cast-in-place resistance slab is required to prevent stressing on the facing panels due to slab curing and shrinking.

7.2.2 Post and Beam Railings

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 3.0 ft (0.9 m) from the wall face, driven 5.0 ft (1.5 m) below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction. Each of the upper two rows of reinforcement shall be designed for an additional horizontal load of 150 lb/ft (2.2 kN/m) of wall, for a total additional load of 300 lb/ft (4.4 kN/m).

7.3 SUPERFLOOD EVENTS AND SCOUR

The stability of walls and abutments in areas of turbulent flow must be addressed in design. Wall design should be based on the total scour depths estimated per Article 2.6.4.4.2 (AASHTO, 2007). Scour should be investigated for two flood conditions:

- Design Flood
- Check Flood

The design flood (storm surge, tide, or mixed population flood) is the more severe of the 100-year event or an overtopping flood of lesser recurrence interval. Stability design of the wall should be assessed assuming that the streambed material above the total scour line has been removed. This should be analyzed as a strength limit state.

The check flood (storm surge, tide, or mixed population flood) is the more severe of the 500-year event or an overtopping flood of lesser recurrence interval. Stability design of the wall should be assessed assuming that the streambed material above the total scour line has been removed. This is an extreme event, and the extreme event limit state applies. Resistance factors for this extreme limit state may be taken at 1.0, per Articles 10.6.4 and 10.5.5.3.3 (AASHTO, 2007).



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and

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16. ABSTRACT This manual is the reference text used for the FHWA NHI courses No. 132042 and 132043 on Mechanically Stabilized Earth Walls and Reinforced Soil Slopes and reflects current practice for the design, construction and monitoring of these structures. This manual was prepared to enable the engineer to identify and evaluate potential applications of MSE walls and RSS as an alternative to other construction methods and as a means to solve construction problems. The scope is sufficiently broad to be of value for specifications specialists, construction and contracting personnel responsible for construction inspection, development of material specifications and contracting methods. With the aid of this text, the engineer should be able to properly select, design, specify, monitor and contract for the construction of MSE walls and RSS embankments. The MSE wall design within this manual is based upon Load and Resistance Factor Design (LRFD) procedures. This manual is a revision (to LRFD) and an update to the FHWA NHI-00-043 manual (which was based upon allowable stress design (ASD) procedures).			
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SI CONVERSION FACTORS				
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
m ²	square meters	1.195	square yards	yd ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m ³	cubic meters	35.71	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
TEMPERATURE				
°C	Celsius	1.8 C + 32	Fahrenheit	°F
WEIGHT DENSITY				
kN/m ³	kilonewton / cubic m	6.36	poundforce / cubic foot	pcf
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kPa	kilopascals	0.145	poundforce / sq in.	psi
kPa	kilopascals	20.9	poundforce / sq ft	psf

PREFACE

Engineers and specialty material suppliers have been designing reinforced soil structures for the past 35 years. Currently, many state DOTs are transitioning their design of substructures from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD) procedures.

This manual is based upon LRFD for MSE wall structures. It has been updated from the 2001 FHWA NHI-00-043 manual. In addition to revision of the wall design to LRFD procedures, expanded discussion on wall detailing and general updates throughout the manual are provided. The primary purpose of this manual is to support educational programs conducted by FHWA for transportation agencies.

A second purpose of equal importance is to serve as the FHWA standard reference for highway projects involving MSE wall and reinforced soil structures.

This Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS), Design and Construction Guidelines Manual which is an update of the current FHWA NHI-00-043, has evolved from the following AASHTO and FHWA references:

- AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007, with 2008 and 2009 Interim Revisions.
- *Earth Retaining Structures*, by B.F. Tanyu, P.J. Sabatini, and R.R. Berg, FHWA-NHI-07-071 (2008).
- AASHTO LRFD Bridge Construction Specifications, 2nd Edition, 2004, with 2006 Interim Revisions.
- *Geosynthetic Design and Construction Guidelines*, by R.D. Holtz, B.R. Christopher, and R.R. Berg, FHWA HI-07-092 (2008).
- *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations*, by R.R. Berg, FHWA-SA-93-025, January 1993.
- *Reinforced Soil Structures - Volume I, Design and Construction Guidelines - Volume II, Summary of Research and Systems Information*, by B.R. Christopher, S.A. Gill, J.P. Giroud, J.K. Mitchell, F. Schlosser, and J. Dunncliff, FHWA RD 89-043 (1990).
- *Design and Construction Monitoring of Mechanically Stabilized Earth Structures*, by J.A. DiMaggio, FHWA, (1994).
- AASHTO Bridge T-15 Technical Committee unpublished working drafts for the update of Section 11.0 of the AASHTO LRFD Bridge Design Specifications.

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CHAPTER 8

REINFORCED SOIL SLOPES PROJECT EVALUATION

8.1 INTRODUCTION

Where right of way is available and the cost of a MSE wall is high, a steepened slope should be considered. In this chapter the background and design requirements for evaluating a reinforced soil slope (RSS) alternative are reviewed. Step-by-step design procedures are presented in Chapter 9. Section 8.2 reviews the types of systems and the materials of construction. Section 8.3 provides a discussion of the internal stability design approach for use of reinforcement as compaction aids, steepening slopes and slope repair. Computer assisted design programs are also reviewed. The section concludes with a discussion of external stability requirements. Section 8.4 reviews the construction sequence. Section 8.5 covers treatment of the outward face of the slope to prevent erosion. Section 8.6 covers design details of appurtenant features including traffic barrier and drainage considerations. Finally, section 8.7 presents several case histories to demonstrate potential cost savings.

8.2 REINFORCED SOIL SLOPE SYSTEMS

8.2.1 Types of Systems

Reinforced soil systems consist of planar reinforcements arranged in nearly horizontal planes in the reinforced fill to resist outward movement of the reinforced fill mass. Facing treatments ranging from vegetation to flexible armor systems are applied to prevent unraveling and sloughing of the face. These systems are generic in nature and can incorporate any of a variety of reinforcements and facing systems. Design assistance is often available through many of the reinforcement suppliers, which often have proprietary computer programs.

This manual does not cover reinforcing the base section of an embankment for construction over soft soils, which is a different type reinforcement application. The user is referred to the FHWA *Geosynthetics Design and Construction Guidelines* (Holtz et al., 2008) for that application. An extension of this application is to lengthen reinforcement at the base of the embankment to improve the global stability of a reinforced soil slope. This application will be covered; however, steepening a slope significantly increases the potential for bearing capacity failure over soft soils and extensive geotechnical exploration along with rigorous analysis is required.

An alternate slope reinforcement technique, the “Deep Patch” method, is used for stabilizing and potentially repairing roadway fill slopes on secondary roads where removal and replacement are not feasible (e.g., in mountainous terrain). In this method, reinforcements (typically geogrids) are placed in the upper portion of the slope to essentially tie it back. An empirical design approach has been developed by the U.S. Department of Agriculture (USDA) Forest Service technology in partnership with FHWA Federal Lands Highway (Musser and Denning, 2005). The method is not explicitly included in the design sections of Chapters 8 and 9 as this approach is often considered as a temporary repair method to retard the movement of a slope until a more permanent solution can be implemented; however, a brief description of the method is included in Appendix F.

8.2.2 Construction Materials

Reinforcement types. Reinforced soil slopes can be constructed with any of the reinforcements described in Chapter 2. While discrete strip type reinforcing elements can be used, a majority of the systems are constructed with continuous sheets of geosynthetics (i.e., geotextiles or geogrids) or wire mesh. Small, discrete micro reinforcing elements such as fibers, yarns, and microgrids located very close to each other have also been used. However, the design is based on more conventional unreinforced designs with cohesion added by the reinforcement (which is not covered in this manual).

Reinforced Fill Requirements. Reinforced fill requirements for reinforced soil slopes are discussed in Chapter 3. Because a flexible facing (e.g. wrapped facing) is normally used, minor face distortion that may occur due to reinforced fill settlement, freezing and thawing, or wetting and drying can be tolerated. Thus, lower quality reinforced fill than recommended for MSE walls can be used. The recommended reinforced fill is limited to low-plasticity, granular material (i.e., $PI \leq 20$ and ≤ 50 percent finer than a No. 200 US sieve {0.075 mm}). However, with good drainage, careful evaluation of soil and soil-reinforcement interaction characteristics, field construction control, and performance monitoring (see Chapter 11), most indigenous soil can be considered.

8.3 DESIGN APPROACH

8.3.1 Application Considerations

As reviewed in Chapter 2, there are two main purposes for using reinforcement in slopes:

- Improved stability for steepened slopes and slope repair.
- Compaction aids, for support of construction equipment and improved face stability.

The design of reinforcement for safe, steep slopes requires a rigorous analysis. The design of reinforcement for this application is critical, as failure of the reinforcement would result in failure of the slope.

The overall design requirements for reinforced slopes are similar to those for unreinforced slopes: A limit equilibrium, allowable stress approach is used and the factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of failure. LRFD methods have not been fully developed for either unreinforced or reinforced slopes and are thus not included in this manual.

As illustrated in Figure 8-1, there are three failure modes for reinforced slopes:

- Internal, where the failure plane passes through the reinforcing elements.
- External, where the failure surface passes behind and underneath the reinforced zone.
- Compound, where the failure surface passes behind and through the reinforced soil zone.

In some cases, the calculated stability safety factor can be approximately equal in two or all three modes, if the reinforcement strengths, lengths and vertical spacings are optimized (Berg et al., 1989).

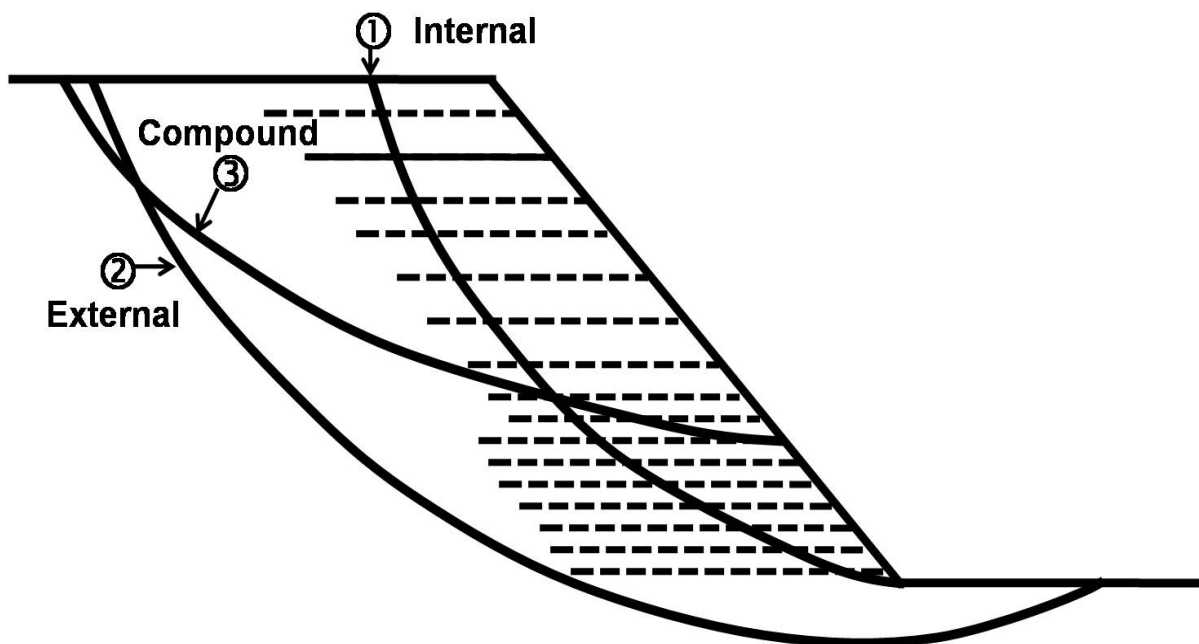


Figure 8-1. Failure modes for reinforced soil slopes including internal failure within the reinforced soil zone, external failure entirely outside the reinforced soil zone, and compound failure starting behind and passing through the reinforced soil zone.

8.3.2 Design of Reinforcement for Compaction Aid

For the use of geosynthetics as compaction aids, the design is relatively simple. Assuming the slope is safe without reinforcement, no reinforcement design is required. Place any geotextile or geogrid that will survive construction at every lift or every other lift of compacted soil in a continuous plane along the edge of the slope. Only narrow strips, about 4 to 6 ft (1.2 to 1.8 m) in width, at 8 to 18 in. (200 to 500 mm) vertical spacing are required. Where the slope angle approaches the angle of repose of the soil, it is recommended that a face stability analysis be performed using the method presented in the reinforcement design section of Chapter 9. Where reinforcement is required by analysis, the narrow strip reinforcement may be considered as secondary reinforcement used to improve compaction and stabilize the slope face between primary reinforcing layers.

8.3.3 Design of Reinforcement for Steepening Slopes and Slope Repair

For steepened reinforced slopes (face inclination up to 70 degrees) and slope repair, design is based on modified versions of the classical limit equilibrium slope stability methods as shown in Figure 8-2:

- Circular or wedge-type potential failure surface is assumed.
- The relationship between driving and resisting forces or moments determines the slope factor of safety.
- Reinforcement layers intersecting the potential failure surface are assumed to increase the resisting force or moment based on their tensile capacity and orientation. (Usually, the shear and bending strengths of stiff reinforcements are not taken into account.)
- The tensile capacity of a reinforcement layer is taken as the minimum of its allowable pullout resistance behind (or in front of) the potential failure surface and its long-term allowable design strength, T_{al} .

As shown in Figure 8-1, a wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. For the internal analysis, the critical slope stability factor of safety is taken from the internal unreinforced failure surface requiring the maximum reinforcement. This is the failure surface with the largest unbalanced driving moment to resisting moment and not the surface with the minimum calculated unreinforced factor of safety. This failure surface is equivalent to the critical reinforced failure surface with the lowest factor of safety. Detailed design of reinforced zone is performed by determining the factor of safety with successively modified reinforcement layouts until the target factor of safety is achieved. External and compound stability of the reinforced zone are then evaluated.

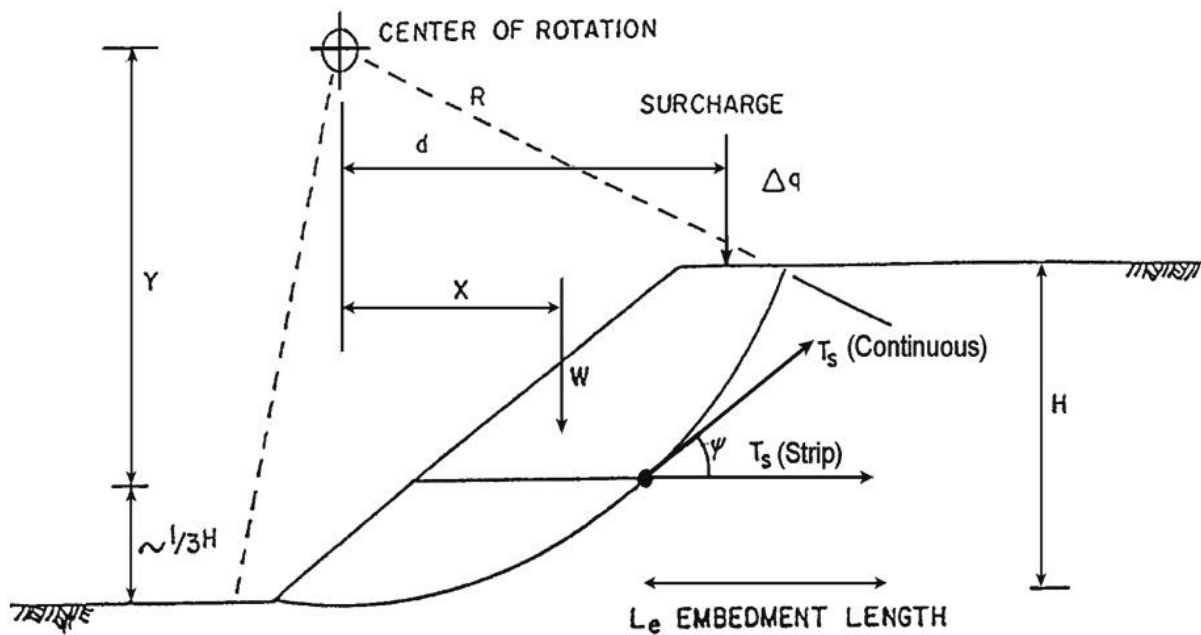


Figure 8-2. Modified limit equilibrium analysis for reinforced slope design.

For slope repair applications, it is also very important to identify the cause of the original failure to make sure that the new reinforced soil slope will not have the same problems. If a water table or erratic water flows exist, particular attention has to be paid to drainage. In natural soils, it is also necessary to identify any weak seams that might affect stability.

The method presented in this manual uses any conventional slope stability computer program and the steps necessary to manually calculate the reinforcement requirements for almost any condition. Figure 8-2 shows the conventional rotational slip surface method used in the analysis. Fairly complex conditions can be accommodated depending on the analytical method used (e.g., Modified Bishop, Spencer). The computer program ReSSA (ADAMA, 2001) was developed by the FHWA to specifically perform this analysis.

The rotational slip surface approach is used for slopes up to 70 degrees, although technically it is a valid method for evaluating even steeper slopes. Slopes steeper than 70 degrees are defined as walls and lateral earth pressure procedures in Chapter 4 apply.

The assumed orientation of the reinforcement tensile force influences the calculated slope safety factor. In a conservative approach, the deformability of the reinforcements is not taken into account, and thus, the tensile forces per unit width of reinforcement T_r are assumed to always be in the horizontal direction of the reinforcements. When close to failure, however, the reinforcements may elongate along the failure surface, and an inclination from the horizontal can be considered.

The above reinforcement orientations represent a simplifying assumption considering the reinforcement is not incorporated directly into the analysis of the slope. If a more rigorous evaluation is performed in which the vertical and horizontal components of the tension forces are included in the equations of equilibrium, then it can be shown that an increase in normal stress will occur for reinforcements with an orientation other than tangential to the failure surface (Wright and Duncan, 1990). In effect, this increase in normal stress will result in practically the same reinforcement influence on the safety factor whether it is assumed to act tangentially or horizontally. Although these equilibrium considerations may indicate that the horizontal assumption is conservative for discontinuous strip reinforcements, it should be recognized that the stress distribution near the point of intersection of the reinforcement and the failure surface is complicated. The conclusion concerning an increase in normal stress should only be considered for continuous and closely spaced reinforcements: it is questionable and should not be applied to reinforced slopes with widely spaced and/or discrete, strip type reinforcements.

Tensile force direction is, therefore, dependent on the extensibility and continuity of the reinforcements used, and the following inclination is suggested:

- Discrete, strip reinforcements: T parallel to the reinforcements.
- Continuous, sheet reinforcements: T tangent to the sliding surface.

8.3.4 Computer-Assisted Design

The ideal method for reinforced slope design is to use a conventional slope stability computer program that has been modified to account for the stabilizing effect of reinforcement. Such programs should account for reinforcement strength and pullout capacity, compute reinforced and unreinforced safety factors automatically, and have some searching routine to help locate critical surfaces. The method may also include the confinement effects of the reinforcement on the shear strength of the soil in the vicinity of the reinforcement.

A number of reinforced slope programs are commercially available, several of which follow the design approach detailed in Chapter 9 of this manual. As previously indicated the development of program ReSSA was initially sponsored by the FHWA. ReSSA implicitly contains the design approach in this manual, noted as the FHWA Bishop Method in the program's rotational stability analysis section, and contains the previous version of this manual in the help screens. ReSSA also provides alternate methods of analysis and the help screens describe those methods in detail including a theoretical discussion of the approaches.

FHWA does not exclude the use of other methods of analysis, especially those which are more comprehensive. However, the user must have a fundamental understanding of which design method(s) are being used and how the algorithms incorporate the reinforcement into the stability analysis, with some programs using simplifying assumptions, while others apply comprehensive formulation and correspondingly complicated computations. Appropriate factors of safety must then be applied to account for uncertainties of the analytical method and the geotechnical and reinforcement materials.

Some of the less sophisticated programs do not design the reinforcement but allow for an evaluation of a given reinforcement layout. An iterative approach then follows to optimize either the reinforcement strength or layout. Many of these programs are limited to simple soil profiles and, in some cases, simple reinforcement layouts. Also, external stability evaluation may be limited to specific soil and reinforcement conditions and a single mode of failure. In some cases, these programs are reinforcement-specific.

With computerized analyses, the actual factor of safety value (FS) is dependent upon how the specific program accounts for the reinforcement tension in the moment equilibrium equation. The method of analysis in Chapter 9 and in FHWA Bishop method in ReSSa, as well as many others, assume the reinforcement force as contributing to the resisting moment, *i.e.*:

$$FS_R = \frac{M_R + T_S R}{M_D} \quad (8-1)$$

- where, FS_R = the required stability factor of safety
- M_R = resisting moment provided by the strength of the soil
- M_D = driving moment about the center of the failure circle
- T_S = sum of tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface
- R = the moment arm of T_S about the center of failure circle as shown in Figure 8-2

With this assumption, FS_R is applied to both the soil and the reinforcement as part of the analysis. As a result, the stability with respect to breakage of the reinforcement requires that the allowable reinforcement strength T_{al} from Chapter 3, equation 3-12 must be greater than or equal to the required maximum design tension T_{max} for each reinforcement layer.

Some computer programs use an assumption that the reinforcement force is a negative driving component, thus the FS is computed as:

$$FS = \frac{M_R}{M_D - T_S R} \quad (8-2)$$

With this assumption, the stability factor of safety is not applied to T_S . Therefore, the allowable design strength T_{al} should be computed as the ultimate tensile strength T_{ULT} divided by the required safety factor (i.e., target stability factor of safety) along with the appropriate reduction factors RF in equation 8-12; i.e., $T_{al} = T_{ULT} / (FS_R \times RF)$. This provides an equivalent factor of safety to equation 8-1, which is appropriate to account for uncertainty in material strengths and reduction factors. The method used to develop design charts should likewise be carefully evaluated to determine FS used to obtain the allowable reinforcement strength.

8.3.5 Evaluation of External Stability

The external stability of reinforced soil slopes depends on the ability of the reinforced zone to act as a stable block and withstand all external loads without failure. Failure possibilities as shown in Figure 8-3 include wedge and block type sliding, deep-seated overall instability, local bearing capacity failure at the toe (lateral squeeze type failure), as well as excessive settlement from both short- and long-term conditions.

The reinforced zone must be sufficiently wide at any level to resist wedge and block type sliding. To evaluate sliding stability, a wedge type failure surface defined by the limits of the reinforcement can be analyzed using the conventional sliding block method of analysis as detailed in the FHWA *Soils and Foundations Workshop Reference Manual*, (Samtani and Nowatzki, 2006). The computer program ReSSA incorporates wedge analysis of the reinforced system, using force equilibrium to analyze sliding both beyond and through the reinforced section.

Conventional soil mechanics stability methods should also be used to evaluate the global stability of the reinforced soil zone. Both rotational and wedge type failure surfaces extending behind and below the structure should be considered. Care should be taken to

identify any weak soil layers in the retained fill and natural soils behind and/or foundation soil below the reinforced soil zone. Evaluation of potential seepage forces is especially critical for global stability analysis. Compound failure surfaces initiating externally and passing through or between reinforcement sections should also be evaluated, especially for complex slope or soil conditions. Extending the lengths of lower level reinforcements may improve the overall global stability, however, special considerations for the orientation of the reinforcement in the analysis must be considered based on the foundation conditions, as detailed in Chapter 9.

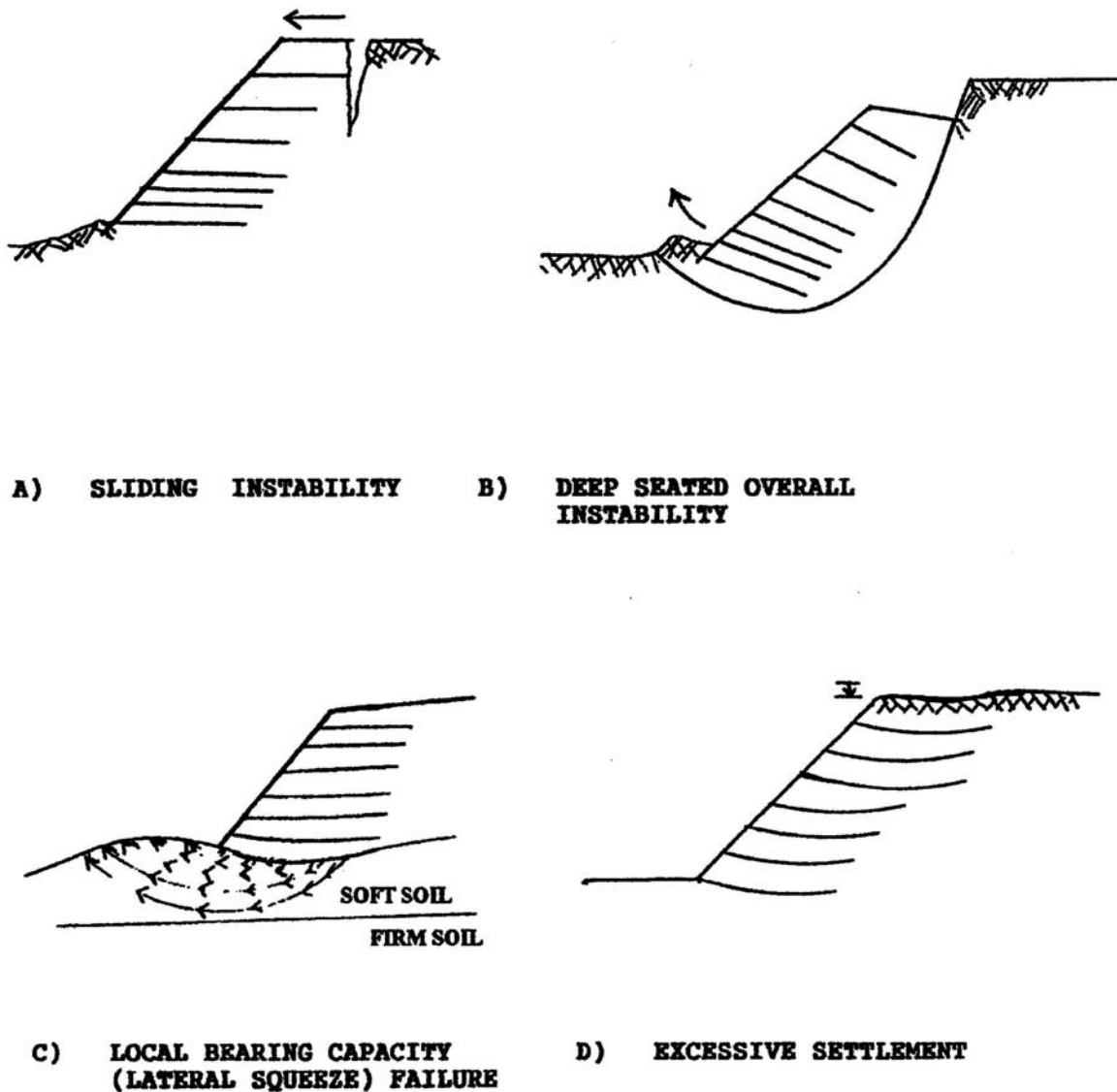


Figure 8-3. External failure modes for reinforced soil slopes.

Evaluation of deep-seated failure does not automatically check for bearing capacity of the foundation or failure at the toe of the slope. High lateral stress in a confined soft stratum beneath the embankment could lead to a lateral squeeze type failure. The shear forces developed under the embankment should be compared to the corresponding shear strength of the soil. Approaches discussed by Jurgenson (1934), Silvestri (1983), and Bonaparte et al. (1987), and Holtz et al. (2008) are appropriate. The approach by Silvestri is demonstrated in example problem E.10 in Appendix E.

Settlement should be evaluated for both total and differential movement. While settlement of the reinforced slope is not of concern, adjacent structures or structures supported by the slope may not tolerate such movements. Differential movements can also affect decisions on facing elements as discussed in Section 8.4.

In areas subject to potential seismic activity, a simple pseudo-static type analysis should be performed using a seismic coefficient obtained from Division 1A of the AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) or using local practice. Reinforced slopes are flexible systems and, unless used for bridge abutments, they are not laterally restrained. For free standing abutments that can tolerate lateral displacement of up to 10A in., Division 1A – Seismic Design, Article 6.4.3 Abutments (AASHTO, 2002) and Appendix A11.1.1.2 (AASHTO, 2007) both imply that a seismic design acceleration $A_m = A/2$ and a corresponding horizontal seismic coefficient $K_h = A/2$ can be used for seismic design. Appropriately a seismic design acceleration of $A/2$ is recommended for reinforced soil slopes, unless the slope supports structures that cannot tolerate such movements.

If any of the external stability safety factors are less than the required factor of safety, the following foundation improvement options could be considered:

- Excavate and replace soft soil.
- Flatten the slope.
- Construct a berm at the toe of the slope to provide an equivalent flattened slope. The berm could be placed as a surcharge at the toe and removed after consolidation of the soil has occurred.
- Stage construct the slope to allow time for consolidation of the foundation soils.
- Embed the slope below grade (> 3 ft), or construct a shear key at the toe of the slope (evaluate based on active-passive resistance).
- Use ground improvement techniques (e.g., wick drains, stone columns, etc.)

Additional information on ground improvement techniques can be found in the FHWA Ground Improvement Methods reference manuals NHI-06-019 and NHI-06-020 (Elias et al., 2006).

8.4 CONSTRUCTION SEQUENCE

As the reinforcement layers are easily incorporated between the compacted lifts of fill, construction of reinforced slopes is very similar to normal slope construction. The elements of construction consist of simply:

1. Placing the soil
2. Placing the reinforcement
3. Constructing the face

The following is the usual construction sequence as shown in Figure 8-4:

- Site Preparation
 - Clear and grub site.
 - Remove all slide debris (for slope reinstatement projects).
 - Prepare a level subgrade for placement of the first level of reinforcement.
 - Proof-roll subgrade at the base of the slope with a roller or rubber-tired vehicle.
 - Observe and approve foundation prior to fill placement.
 - Place drainage features (e.g., basedrain and/or backdrain) as required.
- Reinforcing Layer Placement
 - Reinforcement should be placed with the principal strength direction perpendicular to the face of the slope.
 - Secure reinforcement with retaining pins to prevent movement during fill placement.
 - A minimum overlap of 6 in. (150 mm) is recommended along the edges perpendicular to the slope for wrapped face structures. Alternatively with grid reinforcement, the edges may be clipped or tied together. When reinforcements are not required for face support, no overlap is required and edges should be butted.
- Reinforced fill Placement
 - Place fill to the required lift thickness on the reinforcement using a front end loader or dozer operating on previously placed fill or natural ground.
 - Maintain a minimum of 6 in. (150 mm) of fill between the reinforcement and the wheels or tracks of construction equipment.

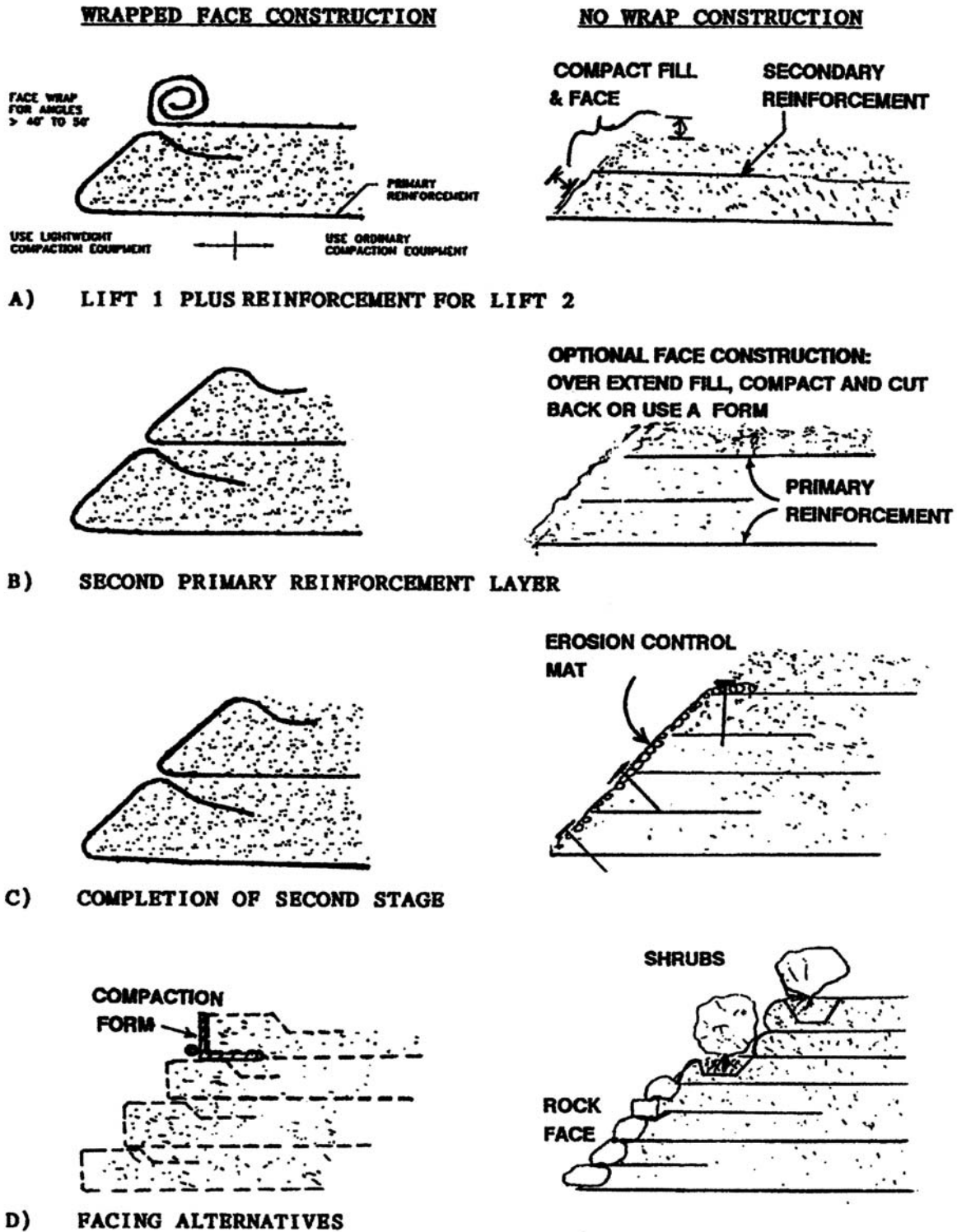


Figure 8-4. Construction of reinforced soil slopes.

- Compact with a vibratory roller or plate type compactor for granular materials or a rubber-tired or smooth drum roller for cohesive materials.
 - When placing and compacting the reinforced fill material, care should be taken to avoid any deformation or movement of the reinforcement.
 - Use lightweight compaction equipment near the slope face with welded wire mesh systems to help maintain face alignment.
- **Compaction Control**
 - Provide close control on the water content and density of the reinforced fill. It should be compacted to at least 95 percent of the standard AASHTO T99 maximum density within 2 percent of optimum moisture.
 - If the reinforced fill is a coarse aggregate, then a relative density or a method type compaction specification should be used.
 - **Face Construction**
 - Slope facing requirements will depend on soil type, slope angle and the reinforcement spacing as shown in Table 8-1.

If slope facing is required to prevent sloughing (i.e., slope angle β is greater than ϕ_{soil}) or erosion, several options are available. Sufficient reinforcement lengths could be provided for wrapped faced structures. A face wrap may not be required for slopes up to about 1H:1V as indicated in Figure 8-4. In this case, the reinforcements (primary and secondary) can be simply extended to the face. For this option, a facing treatment as detailed under Section 8.5 Treatment of Outward Face, should be applied at sufficient intervals during construction to prevent face erosion. For wrapped or no wrap construction, the reinforcement should be maintained at close spacing (i.e., every lift or every other lift but no greater than 16 in. {400 mm}). For armored, hard faced systems the maximum spacing should be no greater than 32 in. (800 mm). A positive frictional or mechanical connection should be provided between the reinforcement and armored type facing systems.

The following procedures are recommended for wrapping the face.

- Turn up reinforcement at the face of the slope and return the reinforcement a minimum of 3 ft (1 m) into the embankment below the next reinforcement layer (see Figure 8-4).
- For steep slopes, formwork may be required to support the face during construction, improving compaction at the face and providing a smoother face finish. Welded wire mesh is often used as a face form (see Figure 8-5). The

welded wire face is left in place with ungalvanized used for temporary support and galvanized wire used for permanent support.

- For grid reinforcements, a fine mesh screen or geotextile may be required at the face to retain reinforced fill materials.

Slopes steeper than approximately 1:1 typically require facing support during construction. Exact slope angles will vary with soil types, i.e., amount of cohesion. Removable facing supports (e.g., wooden forms) or left-in-place welded wire mesh forms are typically used. Facing support may also serve as permanent or temporary erosion protection, depending on the requirements of the slope.

- **Additional Reinforcing Materials and Retained Backfill Placement**

If drainage layers are required, they should be constructed directly behind or on the sides of the reinforced section.

Table 8-1. RSS slope facing options (after Collin, 1996).

Slope Face Angle and Soil Type	Type of Facing			
	When Geosynthetic is not Wrapped at Face		When Geosynthetic is Wrapped at Face	
	Vegetated Face ¹	Hard Facing ²	Vegetated Face ¹	Hard Facing ²
> 50° (> ~0.9H:1V) All Soil Types	Not Recommended	Gabions	Sod, Permanent Erosion Blanket w/ seed	Wire Baskets, ³ Stone, Shotcrete
35° to 50° (~ 1.4H:1V to 0.9H:1V) Clean Sands (SP) ⁴ Rounded Gravel (GP)	Not Recommended	Gabions, Soil-Cement	Sod, Permanent Erosion Blanket w/ seed	Wire Baskets, ³ Stone, Shotcrete
35° to 50° (~ 1.4H:1V to 0.9H:1V) Silt (ML) Sandy Silts (ML)	Soil Bio reinforcement, Drainage Composites ⁵	Gabions, Soil-Cement, Stone Veneer	Sod, Permanent Erosion Blanket w/ seed	Wire Baskets, ³ Stone, Shotcrete
35° to 50° (~ 1.4H:1V to 0.9H:1V) Silty Sands (SM) Clayey Sands (SC) Well graded sands and gravels (SW & GW)	Temporary Erosion Blanket w/ Seed or Sod, Permanent Erosion Mat w/ Seed or Sod	Hard Facing, Not Needed	Geosynthetic Wrap Not Needed	Geosynthetic Wrap Not Needed
25° to 35° (~ 2H:1V to 1.4H:1V) All Soil Types	Temporary Erosion Blanket w/ Seed or Sod, Permanent Erosion Mat w/ Seed or Sod	Hard Facing Not Needed	Geosynthetic Wrap Not Needed	Geosynthetic Wrap Not Needed

- Notes:
1. Vertical spacing of reinforcement (primary/secondary) shall be no greater than 16 in. (400 mm) with primary reinforcements spaced no greater than 32 in. (800 mm) when secondary reinforcement is used.
 2. Vertical spacing of primary reinforcement shall be no greater than 32 in. (800 mm).
 3. 18 in. (450 mm) high wire baskets are recommended.
 4. Unified Soil Classification
 5. Geosynthetic or natural horizontal drainage layers to intercept and drain the saturated soil at the face of the slope.

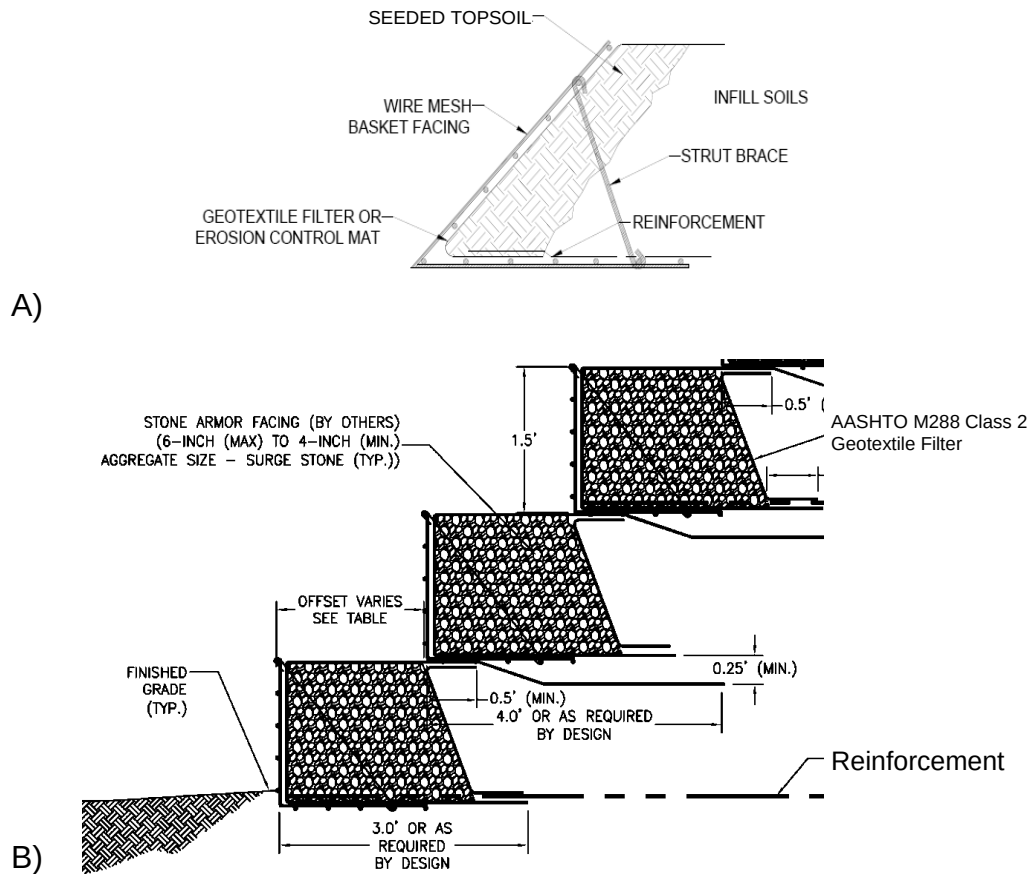


Figure 8-5. Example of welded wire mesh detail for temporary (during construction) or permanent face support showing a) smooth inclined face, and b) stepped face.

8.5 TREATMENT OF OUTWARD FACE

8.5.1 Grass Type Vegetation

Stability of a slope can be threatened by erosion due to surface water runoff. Erosion control and revegetation measures must, therefore, be an integral part of all reinforced slope system designs and specifications. If not otherwise protected, reinforced slopes should be vegetated after construction to prevent or minimize erosion due to rainfall and runoff on the face. Vegetation requirements will vary by geographic and climatic conditions and are, therefore, project specific.

For the unwrapped face (the soil surface exposed), erosion control measures are necessary to prevent raveling and sloughing of the face. A wrapped face helps reduce erosion problems; however, treatments are still required on the face to shade geosynthetic soil reinforcement and prevent ultraviolet light exposure that will degrade the geosynthetic over time. In either case, conventional vegetated facing treatments generally rely on low growth, grass type

vegetation with more costly flexible armor occasionally used where vegetation cannot be established. Due to the steep grades that can be achieved with reinforced soil slopes, it can be difficult to establish and maintain grass type vegetative cover. The steepness of the grade limits the amount of water absorbed by the soil before runoff occurs. Although root penetration should not affect the reinforcement, the reinforcement may restrict root growth, depending on the reinforcement type. This can have an adverse influence on the growth of some plants. Grass is also frequently ineffective where slopes are impacted by waterways.

A synthetic (permanent) erosion control mat is normally used to improve the performance of grass cover. This mat must also be stabilized against ultra-violet light and should be inert to naturally occurring soil-born chemicals and bacteria. The erosion control mat serves to: 1) protect the bare soil face against erosion until the vegetation is established; 2) assist in reducing runoff velocity for increased water absorption by the soil, thus promoting long-term survival of the vegetative cover; and 3) reinforce the surficial root system of the vegetative cover.

Once vegetation is established on the face, it must be protected to ensure long-term survival. Maintenance issues, such as mowing (if applicable), must also be carefully considered. The shorter, weaker root structure of most grasses may not provide adequate reinforcement and erosion protection. Grass is highly susceptible to fire, which can also destroy the synthetic erosion control mat. Downdrag from snow loads or upland slides may also strip matting and vegetation off the slope face. The low erosion tolerance combined with other factors previously mentioned creates a need to evaluate revegetation measures as an integral part of the design. Slope face protection should not be left to the construction contractor or vendor's discretion. Guidance should be obtained from maintenance and regional landscaping groups in the selection of the most appropriate low maintenance vegetation.

8.5.2 Soil Bioengineering (Woody Vegetation)

An alternative to low growth, grass type vegetation is the use of soil bioengineering methods to establish hardier, woody type vegetation in the face of the slope (Sotir and Christopher, 2000). Soil bioengineering uses living vegetation purposely arranged and imbedded in the ground to prevent shallow mass movement and surficial erosion. However, the use of soil bioengineering in itself is limited to stable slope masses. Combining this highly erosive system with geosynthetic reinforcement produces a very durable, low maintenance structure with exceptional aesthetic and environmental qualities.

Appropriately applied, soil bioengineering offers a cost-effective and attractive approach for stabilizing slopes against erosion and shallow mass movement, capitalizing on the benefits and advantages that vegetation offers. The value of vegetation in civil engineering and the

role woody vegetation plays in the stabilization of slopes has gained considerable recognition in recent years (Gray and Sotir, 1995). Woody vegetation improves the hydrology and mechanical stability of slopes through root reinforcement and surface protection. The use of deeply-installed and rooted woody plant materials, purposely arranged and imbedded during slope construction offers:

- Immediate erosion control for slopes; stream, and shoreline.
- Improved face stability through mechanical reinforcement by roots.
- Reduced maintenance costs, with less need to return to revegetate or cut grass.
- Modification of soil moisture regimes through improved drainage and depletion of soil moisture and increase of soil suction by root uptake and transpiration.
- Enhanced wildlife habitat and ecological diversity.
- Improved aesthetic quality and naturalization.

The biological and mechanical elements must be analyzed and designed to work together in an integrated and complementary manner to achieve the required project goals. In addition to using engineering principles to analyze and design the slope stabilization systems, plant science and horticulture are needed to select and establish the appropriate vegetation for root reinforcement, erosion control, aesthetics and the environment. Numerous areas of expertise must integrate to provide the knowledge and awareness required for success. RSS systems require knowledge of the mechanisms involving mass and surficial stability of slopes. Likewise when the vegetative aspects are appropriate to serve as reinforcements and drains, an understanding of the hydraulic and mechanical effects of slope vegetation is necessary. Figure 8-6 shows a cross section of the components of a vegetated reinforced slope (VRSS) system. The design details for face construction include vegetation selection, placement, and development as well as several agronomic and geotechnical design issues (Sotir and Christopher, 2000).

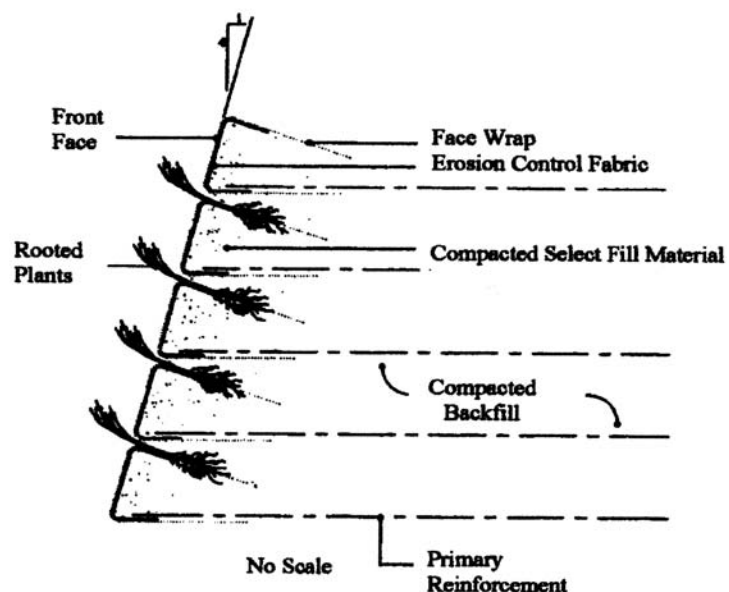


Figure 8-6. Components of a vegetated reinforced slope (VRSS) system.

- Vegetation Selection

The vegetation used in the VRSS system is typically in the form of live woody branch cuttings from species that root adventitiously or from bare root and/or container plants. Plant materials may be selected for a variety of tolerances including: drought, salt, flooding, fire, deposition, and shade. They may be chosen for their environmental wildlife value, water cleansing capabilities, flower, branch and leaf color or fruits. Other interests for selection may include size, form, rate of growth rooting characteristics and ease of propagation. Time of year for construction of a VRSS system also plays a critical roll in plant selection.

- Vegetation Placement

The decision to use native, naturalized or ornamental species is also an important consideration. The plant materials are placed on the frontal section of the formed terraces. Typically 6 to 12 in. (150 to 300 mm) protrude beyond the constructed terrace edge or finished face, and 1.5 to 10 ft (0.5 to 3 m) of the live branch cuttings (when used) are embedded in the reinforced fill behind, or as in the case of rooted plants, are placed 1 to 3 ft (0.3 to 1 m) into the reinforced fill. The process of plant installation is best and least expensive when it occurs simultaneously with the conventional construction activities, but may be incorporated later if necessary.

- Vegetation Development

Typically soil bioengineering VRSS systems offer immediate results from the surface erosion control structural/mechanical and hydraulic perspectives. Over time, (generally within the first year) they develop substantial top and root growth further enhancing those benefits, as well as providing aesthetic and environmental values.

- Design Issues

There are several agronomic and geotechnical design issues that must be considered, especially in relation to selection of geosynthetic reinforcement and type of vegetation. Considerations include root and top growth potential. The root growth potential consideration is important when face reinforcement enhancement is required. This will require a review of the vertical spacing based on the anticipated root growth for the specific type of plant. In addition to spacing, the type of reinforcements is also important. Open-mesh geogrid-type reinforcements, for example, are excellent as the roots will grow through the grid and further "knit" the system together. On the other hand, geocomposites, providing both reinforcement and lateral drainage, offer enhanced water and oxygen opportunities for the healthy development of the woody vegetation. Dependent upon the species selected, aspect,

climatic conditions, soils, etc., dense woody vegetation can provide ultraviolet light protection within the first growing season and maintain the cover thereafter.

In arid regions, geosynthetics that will promote moisture movement into the slope such as non-woven geotextiles or geocomposites may be preferred. Likewise, the need for water and nutrients in the slope to sustain and promote vegetative growth must be balanced against the desire to remove water so as to reduce hydrostatic pressures. Plants can be installed to promote drainage toward geosynthetic drainage net composites placed at the back of the reinforced soil section.

Organic matter is not required; however, a medium that provides nourishment for plant growth and development is necessary. As mentioned earlier, the agronomic needs must be balanced with the geotechnical requirements, but these are typically compatible. For both, a well-drained reinforced fill is needed. The plants also require sufficient fines to provide moisture and nutrients. While this may be a limitation, under most circumstances, some slight modifications in the specifications to allow for some non-plastic fines in the reinforced fill in the selected frontal zone offers a simple solution to this problem.

While many plants can be installed throughout the year, the most cost effective, highest rate of survival and best overall performance and function occurs when construction is planned around the dormant season for the plants, or just prior to the rainy season. This may require some specific construction coordination in relation to the placement of fill, and in some cases could preclude the use of a VRSS structure.

8.5.3 Armored

A permanent facing such as gunite or emulsified asphalt may be applied to a geosynthetic reinforcement RSS slope face to provide long-term ultra-violet protection, if the geosynthetic UV resistance is not adequate for the life of the structure. Galvanized welded wire mesh reinforcement or facing or gabions may also be used to facilitate face construction and provide permanent facing systems.

- Other armored facing elements may include riprap, stone veneer, articulating modular units, or fabric-formed concrete.
- Structural elements.

Structural facing elements (see MSE walls) may also be used, especially if discrete reinforcing elements such as metallic strips are used. These facing elements may include prefabricated concrete slabs, modular precast blocks, or precast slabs.

8.6 DESIGN DETAILS

As with MSE wall projects, certain design details must often be considered that are not directly connected with internal or external stability evaluation. These important details include:

- Guardrail and traffic barriers.
- Drainage considerations.
- Obstructions.

8.6.1 Guardrail and Traffic Barriers

Guardrails are usually necessary for steeper highway embankment slopes. Guardrail posts usually can be installed in their standard manner (i.e. drilling or driving) through geosynthetic reinforcements. Special wedge shaped shoes can be used to facilitate installation. This does not significantly impair the overall strength of the geosynthetic and no adjustments in the design are required. Alternatively, post or concrete form tubes at post locations can be installed during construction. Either this procedure or cantilever type guardrail systems are generally used for metallic reinforcement.

Impact traffic load on barriers constructed at the face of a reinforced soil slope is designed on the same basis as an unreinforced slope. The traffic barrier may be designed to resist the overturning moment in accordance with Article 2.7 in Division I of AASHTO Standard Specifications for Highway Bridges (AASHTO, 2002), Section 13 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007) or as addressed in the 2006 AASHTO *Roadside Design Guide*, and will be covered in detail in Chapter 9.

8.6.2 Drainage Considerations

Uncontrolled subsurface water seepage can decrease stability of slopes and could ultimately result in slope failure.

- Hydrostatic forces on the back of the reinforced zone will decrease stability against sliding failure.

- Uncontrolled seepage into the reinforced zone will increase the weight of the reinforced mass and may decrease the shear strength of the soil, thus decreasing stability.
- Seepage through the reinforced zone can reduce pullout capacity of the reinforcement at the face and increase soil weight, creating erosion and sloughing problems.

As a precaution, drainage features should be included unless detailed analysis proves that drainage is not required. Drains are typically placed at the rear of the reinforced soil zone to control subsurface water seepage as detailed in Chapter 9. Surface runoff should also be diverted at the top of the slope to prevent it from flowing over the face.

8.6.3 Obstructions

If encountered in a design, guidance provided in Chapters 5 should be considered.

8.7 CASE HISTORIES

The following case histories are presented to provide representative examples of cost-effective, successful reinforced slope projects. In several cases, instrumentation was used to confirm the performance of the structure. All project information was obtained from the indicated references which, in most cases, contain additional details.

8.7.1 The Dickey Lake Roadway Grade Improvement Project (Yarger and Barnes, 1993)

Dickey Lake is located in northern Montana approximately 25 miles (40 km) south of the Canadian border. Reconstruction of a portion of U.S. 93 around the shore of Dickey Lake required the use of an earth-retention system to maintain grade and alignment. The fill soils available in the area consist primarily of glacial till. Groundwater is active in the area. A slope stability factor of safety criteria of 1.5 was established for the embankments. A global stability analysis of reinforced concrete retaining walls to support the proposed embankment indicated a safety factor that was less than required. Analysis of a reinforced soil wall or slope indicated higher factors of safety. Based on an evaluation of several reinforcement systems, a decision was made to use a reinforced slope for construction of the embankment. The Montana Department of Transportation (MDT) decided that the embankment would not be designed “in-house,” due to their limited experience with this type of structure.

Proposals were solicited from a variety of suppliers, who were required to design the embankment. An outside consultant, experienced in geosynthetic reinforcement design, was retained to review all submittals. Plans and specifications for the geosynthetic reinforced embankment(s) were developed by MDT, with the plans indicating the desired finished geometry. The slopes generally ranged from 30 to 60 ft (9 to 18.3 m) in height. Face angles varied from 1.5H:1V to 0.84H:1V with the typical angle being 1H:1V. The chosen supplier provided a design that utilized both uniaxially and biaxially oriented geogrids. The resulting design called for primary reinforcing geogrids 15 to 60 ft (4.6 to 18.3 m) long and spaced 2 to 4 ft (0.6 to 1.2 m) vertically throughout the reinforced embankment. The ultimate strength of the primary reinforcement was on the order of 6850 lb/ft (100 kN/m). The length of primary reinforcement was partially dictated by global stability concerns. In addition, intermediate reinforcement consisting of lower strength, biaxial geogrids was provided in lengths of 5 ft (1.5 m) with a vertical spacing of 1 ft (0.3 m) at the face of slopes 1H:1V or flatter. Erosion protection on the 1H:1V or flatter sections was accomplished by using an organic erosion blanket. Steeper sections (maximum 0.84H:1V) used L-shaped, welded wire forms with a biaxial grid wrap behind the wire. A design evaluation of this project is presented in Chapter 9.

The design also incorporated subsurface drainage. This drainage was judged to be particularly important due to springs or seeps present along the backslope of the embankment. The design incorporated geocomposite prefabricated drains placed along the backslope, draining into a French drain at the toe of the backslope. Laterals extending under the embankment were used to "daylight" the French drain.

The project was constructed in 1989 at a cost of approximately \$17/ft² (\$180/m²) of vertical face and has been periodically monitored by visual inspection and slope inclinometers. Project photos are shown in Figure 8-7. To date, the embankment performance has been satisfactory with no major problems observed. Some minor problems have been reported with respect to the erosion control measures and some minor differential movement in one of the lower sections of the embankment.

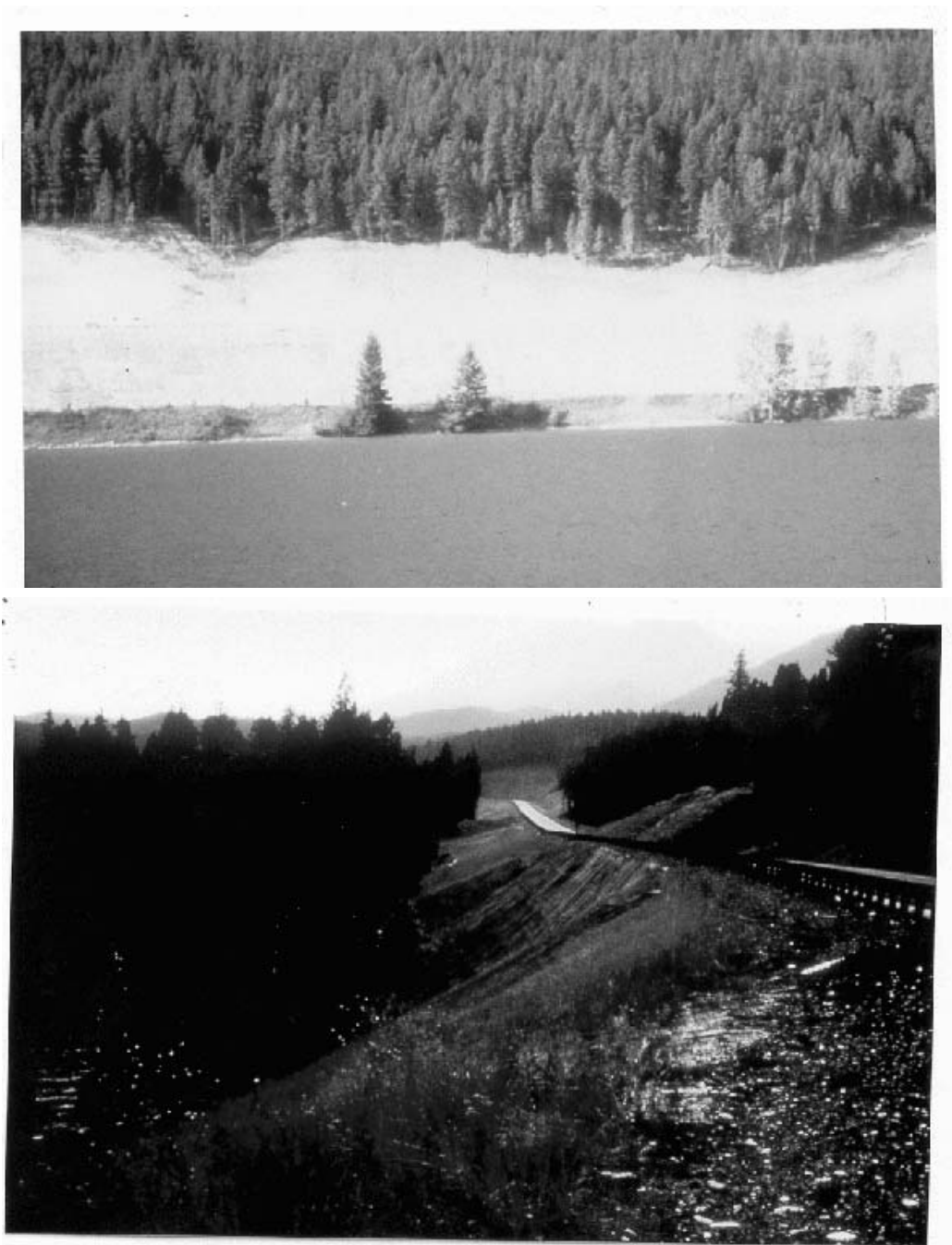


Figure 8-7. Dickey Lake site.

8.7.2 Salmon-Lost Trail Roadway Widening Project

(Zornberg et al., 1995)

As part of a highway widening project in Idaho, the FHWA Western Federal Lands Highway Division designed and supervised the construction of a 565 ft long, 50 ft high (172 m long, 15.3 m high) permanent geosynthetic-reinforced slope to compare its performance with retaining structures along the same alignment. Widening of the original road was achieved by turning the original 2H:1V unreinforced slope into a 1H:1V reinforced slope. Aesthetics was an important consideration in the selection of the retaining structures along scenic Highway 93, which has been recognized by articles in *National Geographic*. A vegetated facing was, therefore, used for the reinforced slope section. On-site soil consisting of decomposed granite was used as the reinforced fill. An important factor in the design was to deal with water seepage from existing slope. Geotextile reinforcements with an in-plane transmissivity were selected to evaluate the potential of modifying the seepage regime in the slope.

The geotextile-reinforced slope was designed in accordance with the guidelines presented in Chapters 8 and 9 of this manual. The final design consisted of two reinforced zones with a constant reinforcing spacing of 1 ft (0.3 m). The reinforcement in the lower zone had an ultimate tensile strength of 6,850 lb/ft (100 kN/m), and the reinforcement in the upper zone had a reinforcement strength of 1,370 lb/ft (20 kN/m). The reinforcement strength was reduced based on partial reduction factors which are reviewed in Chapter 3. Field tests were used to reduce the reduction factor for construction damage from the assumed value of 2.0 to the test value of 1.1 at a substantial savings to the project (40 percent reduction in reinforcement).

The construction was completed in 1993 (see Figure 8-8 for project photos). The structure was constructed as an experimental features project and was instrumented with inclinometers within the reinforced zone, extensometers on the reinforcement, and piezometers within and at the back of the reinforced section. Survey monitoring was also performed during construction. Total lateral displacements recorded during construction were on the order of 0.1 to 0.2 percent of the height of the slope, with maximum strains in the reinforcement measured at only 0.2 percent. Post construction movement has not been observed within the accuracy of the instruments. These measurements indicate the excellent performance of the structure as well as the conservative nature of the design. Long-term monitoring is continuing.

The steepened slope was constructed at a faster rate and proved more economical than the other retaining structures constructed along the same alignment. The constructed cost of the reinforced slope section was on the order of \$15/ft² (\$160/m²) of vertical face. Metallic grid reinforced MSE wall costs in other areas of the site were on the order of \$22/ft² (\$240/m²) of vertical face for similar or lower heights.

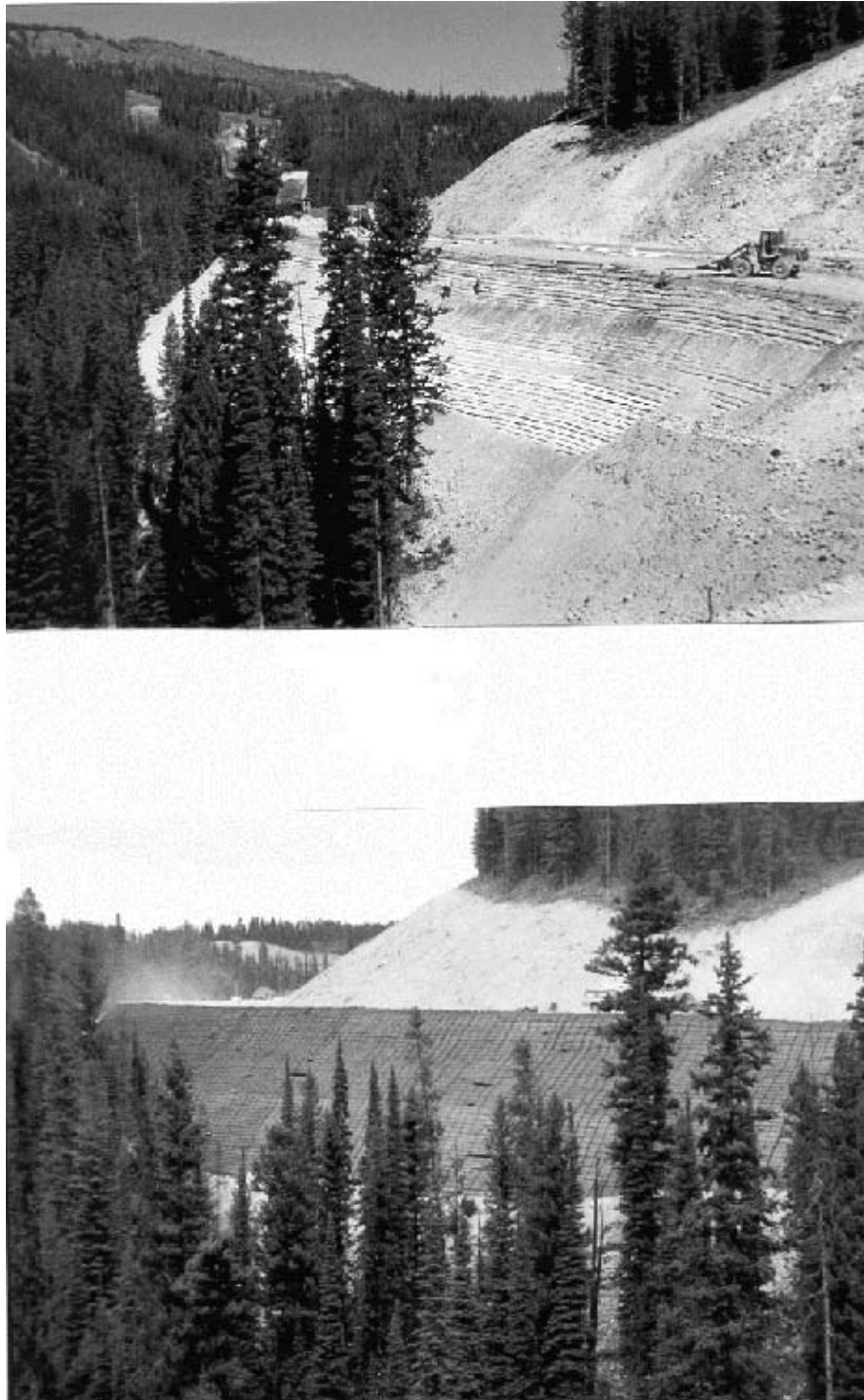


Figure 8-8. Salmon Lost Trail site.

8.7.3 Cannon Creek Alternate Embankment Construction Project (Hayden et al., 1991)

A large embankment was planned to carry Arkansas State Highway 16 over Cannon Creek. The proposed 100,000 yd³ (77,000 m³) embankment had a maximum height of 75 ft (23 m) and was to be constructed with on-site clay soils and 2H:1V side slopes (with questionable stability). A cast-in-place concrete box culvert was first constructed to carry the creek under the embankment. Embankment construction commenced but was halted quickly when several small slope failures occurred. It then became apparent that the embankment fill could not be safely constructed at 2H:1V.

With the box culvert in place, there were two options for continuation of embankment construction. A gravelly soil could be used for embankment fill, or the on-site soil could be used with geosynthetic reinforcement. Both options were bid as alternatives and the geosynthetic option was selected for construction (see Figure 8-9). The reinforcement used was a high-density polyethylene geogrid with a reported wide-width strength of 6850 lb/ft (100 kN/m). The geogrid reinforcement option was estimated to be \$200,000 less expensive than the gravelly soil fill option.

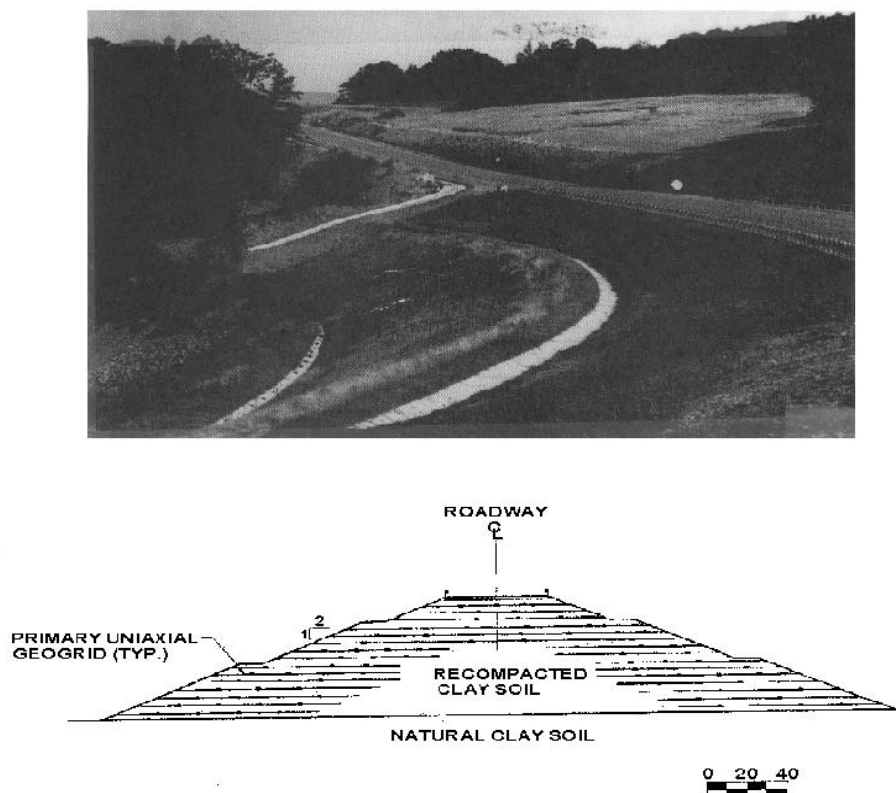


Figure 8-9. Cannon Creek project.

8.7.4 Pennsylvania SR 54 Roadway Repair Project (Wayne and Wilcosky, 1995)

During the winter of 1993 - 1994, a sinkhole formed in a section of State Route 54 in Pennsylvania. Further investigation revealed that an abandoned railroad tunnel had collapsed. The traditional repair would have involved the removal and replacement of the 50 ft (15 m) high embankment. The native soil, a sandy clay, was deemed an unsuitable reinforced fill due to its wet nature and potential stability and settlement problems with the embankment. Imported granular fill to replace the native soil was estimated to be \$21/yd³ (\$16/m³). Due to the high cost of replacement materials, the Pennsylvania Department of Transportation decided to use geosynthetics to provide drainage of the native soil and reinforce the side slopes. A polypropylene needlepunched nonwoven geotextile was selected to allow for pore pressure dissipation of the native soil during compaction, thus accelerating consolidation settlement and improving its strength. Field tests were used to confirm pore pressure response.

With the geotextile placed at a compacted lift spacing of 1 ft (0.3 m), full pore pressure dissipation was achieved within approximately 4 days as compared with a minimum dissipation (approximately 25 percent) without the geosynthetic during the same time period. By placing the geotextile at 1 ft (0.3 m) lift intervals, the effective drainage path was reduced from the full height of the slope of 50 ft (15 m) to 0.5 ft (0.15 m) or by a factor of over 100. This meant that consolidation of the embankment would essentially be completed by the end of construction as opposed to waiting almost a year for completion of the settlement without the geosynthetic.

The geotextile, with an ultimate strength of 1100 lb/ft (16 kN/m) and placed at every lift 12 in. (0.3 m), also provided sufficient reinforcement to safely construct 1.5H:1V side slopes. Piezometers at the base and middle of the slope during construction confirmed the test pad results. Deformations of the geotextile in the side slope were also monitored and found to be less than the precision of the gages (± 1 percent strain). Project photos are shown in Figure 8-10 along with the measurements of pore pressure dissipation during construction.

The contractor was paid on a time and material basis with the geotextile purchased by the agency and provided to the contractor for installation. The cost of the geotextile was approximately \$1/yd² (\$1.2/m²). In-place costs of the geotextile, along with the on-site fill averaged just over \$3/yd³ (\$4/m³) for a total cost of \$70,000, resulting in a savings of approximately \$200,000 over the select-fill alternative. Additional savings resulted from not having to remove the on-site soils from the project site.



Figure 8-10. Pennsylvania SR54.

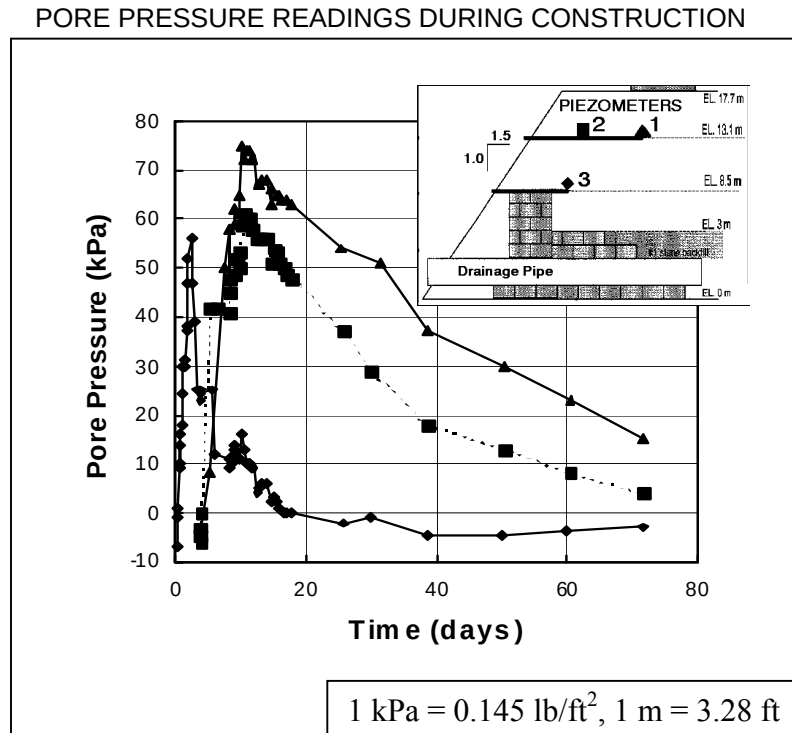


Figure 8-10. Pennsylvania SR54 continued.

8.7.5 Massachusetts Turnpike - Use of Soil Bioengineering (Sotir et al., 1998; Sotir and Stulgis, 1999)

The Massachusetts Turnpike in Charlton, Massachusetts is an example where a vegetated reinforced slope (VRSS) system was used to construct 1H:4V slopes to replace unstable 1.5H:1V slopes along a 500 ft (150 m) section of the Turnpike. This slope eroded for a number of years. The erosion was widening and threatening to move back into private property beyond the right-of-way. Eventually, the increased maintenance to clean up the sloughed material, the visual scar on the landscape and the threat of private property loss prompted the Turnpike Authority to seek a solution. The combined soil bioengineering and geosynthetic reinforcement approach was adopted to meet the narrow right-of-way requirement, assist in controlling internal drainage, and reconstruct an aesthetically pleasing and environmentally sound system that would blend into the natural landscape. The 10 to 60 ft (3 to 18 m) high 1H:4V slope was stabilized with layers of primary and secondary geogrids, erosion control blankets, brushlayers in the frontal geogrid wrapped portion of the face, and additional soil bioengineering treatments above the constructed slope.

The design was essentially the same as the soil bioengineering cross section shown in Figure 8-6. The primary geogrid was designed to provide global, internal and compound stability to the slope. This grid extends approximately 20 ft (6.1 m) from the face to the back of the slope. The vertical spacing of the primary geogrid is 2 ft (0.6 m) and 4 ft (1.2 m), respectively, over the lower and upper halves of the slope. The face wrap extends approximately 3 ft (0.9 m) into the slope at the bottom of each vertical lift and 5 ft (1.5 m) at the top to form 3 ft (0.9 m) thick earthen terraces. Brushlayers consisting of 8 to 10 ft (2.4 m to 3 m) long willow (*Salix* sp.) and dogwood (*Cornus* sp.) live cut branches were placed on each constructed wrapped section at a vertical spacing of 1 ft (0.9 m), extending back to approximately the mid point of the slope. The branches and geogrids were sloped back to promote drainage to backdrains placed in the slopes while providing moisture for the plants. Live fascine bundles (see Figure 8-11) were installed above the reinforced slope in a 3H:1V cut section to prevent surface erosion and assist in revegetating that portion of the slope.

The backdrain system consisted of 3.3 ft (1 m) wide geocomposite panels spaced 15 ft (4.6 m) on center. Design of the panels and spacing was based on the anticipated groundwater flow and surface infiltration conditions. The panels connect into a 1 ft (0.3 m) thick crushed-stone drainage layer at the base of the slope, which extends the full length and width of the slope. The reinforced fill soils consisted of granular borrow, ordinary borrow, 50/50 mix of ordinary granular fill and specified fill. The first three materials constitute the structurally competent core while the specified fill was placed at the face to provide a media amenable to plant growth. The specified fill consisted of fertilizers and a blend of four parts ordinary borrow to one part organic loam by volume and was used in the front 10 ft (3 m) of each lift for the installed brushlayers to optimize the growing conditions. This was a modification from the normal geotechnical specification to accommodate the soil bioengineering.

The VRSS slope was constructed in the winter/spring of 1995/96 at a cost of US \$25/ft² (\$270/m²) of vertical face. After the fourth growing season, the vegetated slope face was evaluated and found to perform as intended, initially protecting the surface from erosion while providing a pleasing aesthetic look (see Figure 8-11). Natural invasion from the surrounding plant community occurred, causing the system to blend into the naturally wooded scenic setting of the area and meeting the long-term aesthetic and ecological goals.

Lessons Learned: In the future on similar projects, the use of more rooted plants rather than all live cut branches is recommended to provide greater diversity and to improve construction efficiency. Reducing the height of the wrapped earth terraces would allow for the vegetation to be more evenly distributed with less densities, and possibly using a preformed wire form in the front. These items would all reduce construction costs by improving efficiency.



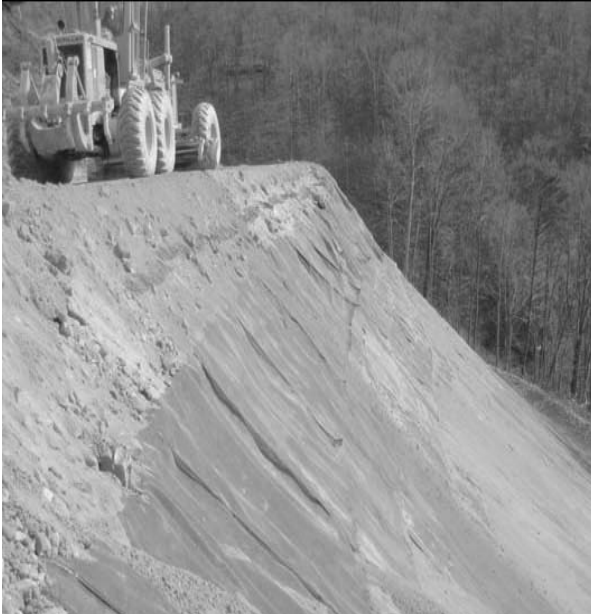
Figure 8-11. Massachusetts Turnpike during construction, immediately after construction and the after the second growing season.

8.7.6 242 ft (74 m) High 1H:1V Reinforced Soil Slope for Airport Runway Extension (Lostumbo, 2009)

The tallest reinforced soil slope in North America as of this writing was constructed to extend the runway at Yeager Airport in Charleston, WV. Yeager Airport was constructed in the 1940's atop mountainous terrain. Due to the mountainous conditions, the ground surface around the airport slopes down steeply over 300 ft (91 m) to the surrounding Elk and Kanawha Rivers, roadways, churches, houses and other structures. In order to meet recent FAA Safety Standards, updates to the airport runways required extending Runway 5 approximately 500 ft (150 m) to create an emergency stopping apron for airplanes. Construction options for extending the runway past the existing hillside included evaluation of bridge structures, retaining walls and reinforced slopes. Engineering evaluation indicated the reinforced slope provided the most cost effective and easiest constructed option. In addition, the vegetated facing of the completed slope will provide a structure that will blend into the surrounding green hills of Charleston, WV. The final design was a 242 ft (74 m) high, 1H:1V reinforced steepened slope (RSS).

The design utilized polyester woven geogrid reinforcements with long term design strengths T_{al} of 2,970 lb/ft (43.4 kN/m), 3720 lb/ft (54.4 kN/m) and 3860 lb/ft (56.4 kN/m) as the primary reinforcement. The vertical spacing of the primary reinforcement was 1.5 ft (460 mm) in the lower portion of the slope and 3 ft (900 mm) in the upper portion. The design embedment length of the primary reinforcement in the taller section of the slope ranged from 175 feet (53 m) at the bottom to 145 ft (44 m) at the top. The design also incorporated a geosynthetic drainage composite for drainage behind the reinforced zone along the back of the excavation to intercept and drain seepage water from the existing mountain side away from the reinforced zone. A geosynthetic erosion control mat was installed on the face of the slope at 2 ft (0.6 m) vertical intervals, with 3 foot (0.9 m) embedded into the slope face and 2.5 ft (0.76 m) down the face for facial stability and erosion protection. An open mesh biaxial geosynthetic specifically designed as a face wrap material was also used in the slope face.

The RSS allowed for an economical solution and less complicated construction than the other, traditional methods that were considered. The reinforced slope was successfully completed and is performing as expected. The structure allowed the airport to meet recent FAA Safety Standards while creating an engineered structure that blends into the scenic green hills of Charleston, WV.



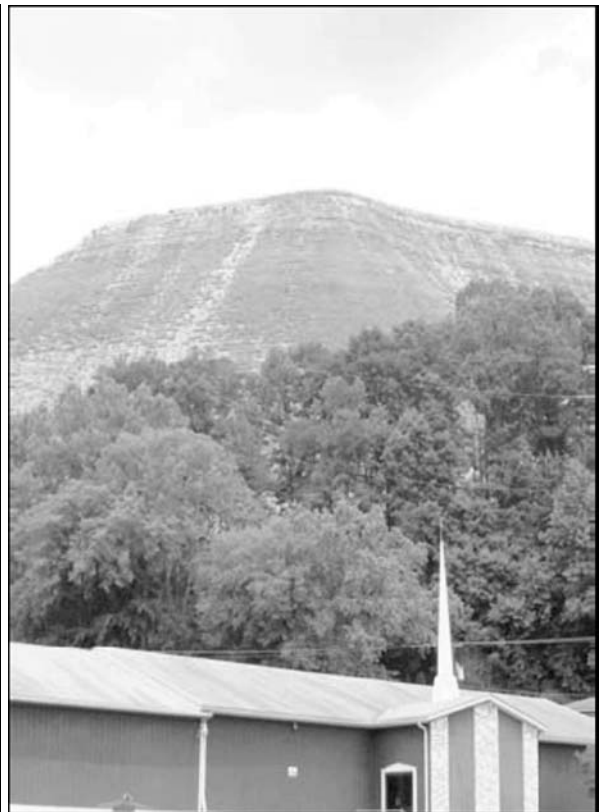
a) Early construction at the bottom of the slope.



b) Slope face during construction.



c) Aerial photo of slope during construction, approximately 80% complete.



d) Slope shortly after completion with early vegetation growth

Figure 8-12. Reinforced soil slope for runway extension at Yeager Airport, Charleston, West Virginia.

8.8 STANDARD RSS DESIGNS

RSS structures are customarily designed on a project-specific basis. Most agencies use a line-and-grade contracting approach, thus the contractor selected RSS vendor provides the detailed design after contract bid and award. This approach works well. However, in addition to agencies performing project specific designs, standard designs can be developed and implemented by an agency for RSS structures.

Use of standard designs for RSS structures offers the following advantages over a line-and-grade approach:

- Agency is more responsible for design details and integrating slope design with other components.
- Pre-evaluation and approval of materials and material combinations, as opposed to evaluating contractor submittal post bid.
- Economy of agency design versus vendor design/stamping of small reinforced slopes.
- Agency makes design decisions versus vendors making design decisions.
- More equitable bid environment as agency is responsible for design details, and vendors are not making varying assumptions.
- Filters out substandard work, systems and designs with associated approved product lists.

The Minnesota Department of Transportation (Mn/DOT) developed and implemented (in-house) standardized RSS designs (Berg, 2000). The use of these standard designs are limited by geometric, subsurface and economic constraints. Structures outside of these constraints should be designed on a project-specific basis. The general approach used in developing these standards could be followed by other agencies to develop their own, agency-specific standard designs.

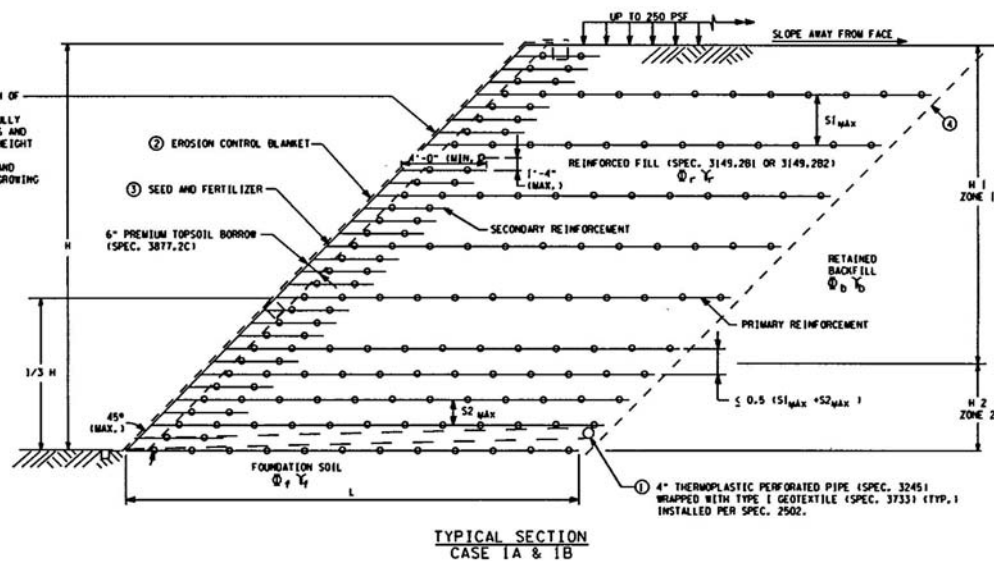
Standardized designs require generic designs and generic materials. Generic designs require definition of slope geometry and surcharge loads, soil reinforcement strength, structure height limit, and slope facing treatment. As an example, the Mn/DOT standard designs address two geometric and surcharge loadings, two reinforced soil fills, and can be used for slopes up to 26.2 ft (8 m) in height. Three reinforcement long-term strengths, T_{al} , of 700, 1050 and 1400 lb/ft (10, 15 and 20 kN/m) are used in the standard designs, although a structure must use the same reinforcement throughout its height and length.

Generic material properties used definitions of shear strength and unit weight of the reinforced fill, retained backfill and foundation soils applicable to the agency's specifications and regional geology. Definition of generic material properties requires the development of approved product lists for soil reinforcements and face erosion control materials. A standard

face treatment is provided, however, it is footnoted with *Develop site specific recommendations for highly shaded areas, highly visible urban applications, or in sensitive areas.*

An example design cross section and reinforcement layout table from the Mn/DOT standard designs is presented in Figure 8-13. Note that the Mn/DOT standard designs are not directly applicable to, nor should they be used by, other agencies.

MAINTENANCE IS REQUIRED AND IS INCIDENTAL TO THE CONSTRUCTION OF THE SLOPE. MAINTENANCE CONSISTS OF WATERING AND EROSION REPAIR RESTORATION SUCH THAT THE FACE OF THE SLOPE WILL BE FULLY VEGETATED. MAINTENANCE INCLUDES A MINIMUM OF 45 GROWING DAYS AND TERMINATES WHEN A VEGETAL DENSITY OF 80% AND A PLANT GROWTH HEIGHT OF 6" IS ACHIEVED. AREAS THAT SUCCEED TO EROSION OR SEEDING FAILURE WILL BE RESTORED WITHIN THREE CALENDAR DAYS AND WILL REQUIRE AN ADDITIONAL 20 GROWING DAYS OF MAINTENANCE. GROWING DAYS ARE DEFINED AS PER STANDARD SPECIFICATION 2575.31.1.



NOTES:

- ① INSPECT EXCAVATION SLOPES FOR ACTIVE SEEPAGE AND PLACE ADDITIONAL DRAINS WHERE SEEPAGE OCCURS AS DIRECTED BY THE ENGINEER.
- ② STRAW-COCOONIT EROSION CONTROL BLANKET (SPEC. 3885.2A, CATEGORY 4). MAINTENANCE REQUIRED. BEST TO STABILIZE SLOPE IN SECTIONS AT THE END OF EACH DAY. SEE APPROVED PRODUCTS LIST: www.mtr.dot.state.mn.us/ PICK MATERIALS ENGINEERING, PICK APPROVED PRODUCTS LIST.
- ③ SEED AND FERTILIZE AS SPECIFIED IN PLANS.
- ④ PAY LIMITS OF STRUCTURAL EXCAVATION, EQUAL TO ANGLE OF SLOPE FACE, 45° MAXIMUM ACTUAL EXCAVATION SLOPE IS DETERMINED BY OSHA REGULATIONS AND IN-SITU SOILS. EXCAVATION BEYOND "LIMITS OF STRUCTURAL EXCAVATION" AT CONTRACTOR'S EXPENSE.
- ⑤ PRIMARY SOIL REINFORCEMENT TYPES I, II, AND III ARE FOUND ON THE APPROVED PRODUCTS LIST AT www.mtr.dot.state.mn.us/ PICK GEOTECHNICAL ENGINEERING SECTION, PICK FOUNDATIONS UNIT.

REINFORCED SOIL SLOPES									
CASE 1A - 45° MAXIMUM SLOPE ANGLE, GRANULAR BORROW REINFORCED SOIL FILL									
MAX. SLOPE ANGLE (DEGREES)	REINFORCED SOIL FILL FRICTION ANGLE (DEGREES)	MINIMUM REINFORCEMENT LENGTH, L (FT)	PRIMARY SOIL REINFORCEMENT ⑤		MAXIMUM SLOPE HEIGHT H (FT)	ZONE 1		ZONE 2	
			TYPE	LONG TERM STRENGTH (T ₀₁) (PLF)		H1 (FT)	S1 _{MAX} (IN)	H2 (FT)	S2 _{MAX} (IN)
45	30	1.1 H	TYPE I	700	26.2	11.5	40	14.7	20
					26.2	26.2	24	-	-
			TYPE II	1050	26.2	21.3	40	4.9	20
					26.2	17.7	48	8.5	24
		TYPE III	1400	26.2	26.2	48	-	-	
CASE 1B - 45° MAXIMUM SLOPE ANGLE, MODIFIED SELECT GRANULAR BORROW REINFORCED SOIL FILL									
MAX. SLOPE ANGLE (DEGREES)	REINFORCED SOIL FILL FRICTION ANGLE (DEGREES)	MINIMUM REINFORCEMENT LENGTH, L (FT)	PRIMARY SOIL REINFORCEMENT ⑤		MAXIMUM SLOPE HEIGHT H (FT)	ZONE 1		ZONE 2	
			TYPE	LONG TERM STRENGTH (T ₀₁) (PLF)		H1 (FT)	S1 _{MAX} (IN)	H2 (FT)	S2 _{MAX} (IN)
45	35	0.8 H	TYPE I	700	26.2	26.2	40	-	-
					26.2	17.7	48	8.5	24
			TYPE II	1050	26.2	26.2	48	-	-
					26.2	26.2	48	-	-
		TYPE III	1400	26.2	26.2	48	-	-	
NOTES: SECONDARY REINFORCEMENT SHALL HAVE A MINIMUM LONG TERM STRENGTH OF 400 PLF.									

NOTES:

SECONDARY REINFORCEMENT SHALL HAVE A MINIMUM LONG TERM STRENGTH OF 400 PLF.

Figure 8-13. Example of standard RSS design (Mn/DOT, 2008).

CHAPTER 9

DESIGN OF REINFORCED SOIL SLOPES

9.1 INTRODUCTION

This chapter provides step-by-step procedures for the design of reinforced soil slopes. Design and analysis of existing design using a computer program is also presented. The design approach principally assumes that the slope is to be constructed on a stable foundation. Recommendations for deep seated failure analysis are included. The user is referred to standard soil mechanics texts and FHWA *Geosynthetics Design and Construction Guidelines* (Holtz et al., 2008) in cases where the stability of the foundation is at issue.

As indicated in Chapter 8, there are several approaches to the design of reinforced steepened slopes. The method presented in this chapter uses the classic rotational, limit equilibrium slope stability method as shown in Figure 8-2. As for the unreinforced case, a circular arc failure surface (not location) is assumed for the reinforced slope. This geometry provides a simple means of directly increasing the resistance to failure from the inclusion of reinforcement, is directly adaptable to most available conventional slope stability computer programs, and agrees well with experimental results.

The reinforcement is represented by a concentrated force within the soil mass that intersects the potential failure surface. By adding the failure resistance provided by this force to the resistance already provided by the soil, a factor of safety equal to the rotational stability safety factor is inherently applied to the reinforcement. The tensile capacity of a reinforcement layer is taken as the minimum of its allowable pullout resistance behind the potential failure surface or its long-term allowable design strength. The slope stability factor of safety is taken from the critical surface requiring the maximum amount of reinforcement. Final design is performed by distributing the reinforcement over the height of the slope and evaluating the external stability of the reinforced section.

The suitability of this design approach has been verified through extensive experimental evaluation by the FHWA including numerical analysis, centrifuge models, and full scale instrumented structures and found to be somewhat conservative. A chart solution developed for simplistic structures is provided as a check for the results. The method for evaluating a given reinforced soil profile is also presented. The flow chart in Figure 9-1 shows the steps required for design of reinforced soil slopes.

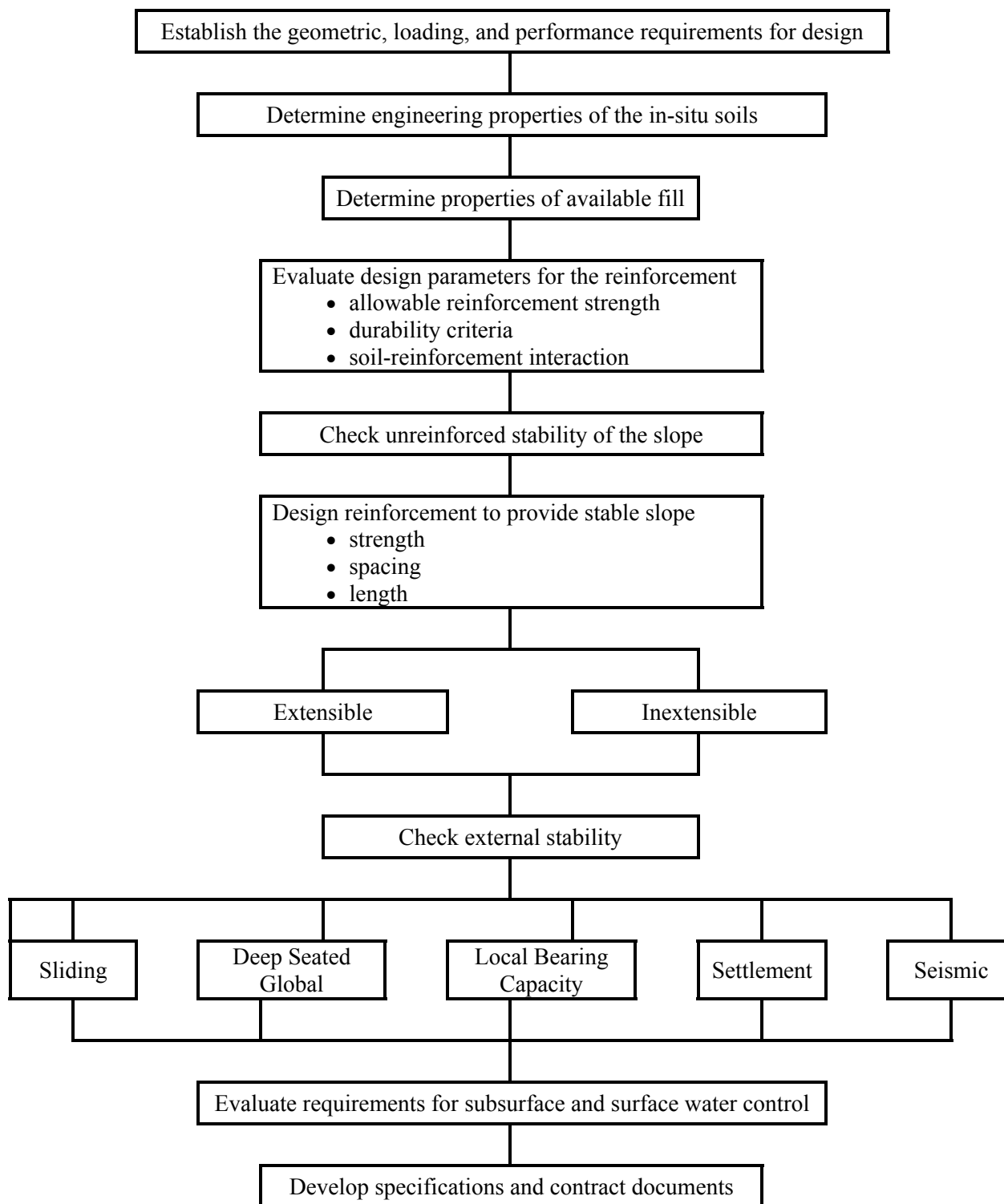


Figure 9-1. Flow chart of steps for reinforced soil slope design.

9.2 REINFORCED SLOPE DESIGN GUIDELINES

The design steps outlined in the flow chart are as follows:

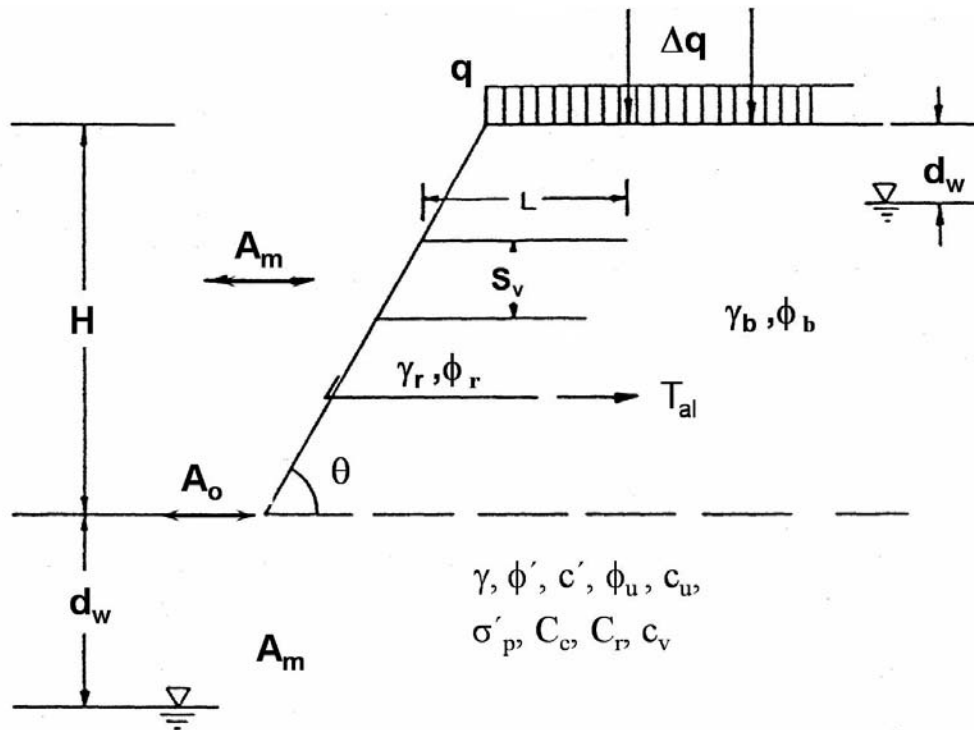
9.2.1 Step 1. Establish the geometric, loading, and performance requirements for design.

- a. Geometric and loading requirements (see Figure 9-2).
 - Slope height, H
 - Slope angle, θ
 - External (surcharge) loads:
 - Surcharge load, q
 - Temporary live load, Δq
 - Design seismic acceleration, A_m (See Division 1A, AASHTO *Standard Specifications for Highway Bridges* {AASHTO, 2002} or AASHTO LRFD Bridge Design Specifications {AASHTO, 2007}).
 - Traffic Barrier
 - See article 2.7 of AASHTO *Standard Specifications for Highway Bridges* (AASHTO, 2002), Section 13 of AASHTO LRFD Bridge Design Specifications (AASHTO, 2007), and AASHTO *Roadside Design Guide* (AASHTO, 2006).
- b. Performance requirements.
 - External stability and settlement
 - Sliding: $F.S. \geq 1.3$
 - Deep seated (overall stability): $F.S. \geq 1.3$
 - Local bearing failure (lateral squeeze): $F.S. \geq 1.3$
 - Dynamic loading: $F.S. \geq 1.1$
 - Settlement-post construction magnitude and time rate based on project requirements
 - Compound failure: $F.S. \geq 1.3$
 - Internal slope stability: $F.S. \geq 1.3$

9.2.2 Step 2. Determine the engineering properties of the in-situ soils.

(see recommendations in Chapter 2, Section 2.6 and Chapter 3, Section 3.2.2)

- The foundation and retained soil (i.e., soil beneath and behind reinforced zone) profiles.
- Strength parameters c_u and ϕ_u , or c' and ϕ' for each soil layer.
- Unit weights γ_{wet} and γ_{dry} .
- Consolidation parameters (C_c , C_r , c_v and σ'_p).
- Location of the groundwater table d_w , and piezometric surfaces.
- For failure repair, identify location of previous failure surface and cause of failure.



Notations:

H = slope height

θ = slope angle

T_{al} = allowable strength of reinforcement

L = length of reinforcement

S_v = vertical spacing of reinforcement

q = surcharge load

Δq = temporary live load

A_o = ground acceleration coefficient

A_m = design seismic acceleration

d_w = depth to ground water table in slope

d_{wf} = depth to ground water table in foundation

c_u and ϕ_u or c' and ϕ' = strength parameters for each soil layer

γ_{wet} and γ_{dry} = unit weights for each soil layer

C_c , C_r , c_v and σ'_p = consolidation parameters for each soil layer

Figure 9-2. Requirements for design of reinforced soil slopes.

9.2.3 Step 3. Determine the properties of reinforced fill and, if different, the retained backfill. (see recommendations in Chapter 3, Section 3.2)

- Gradation and plasticity index
- Compaction characteristics based on 95% AASHTO T-99, γ_d and $\pm 2\%$ of optimum moisture, w_{opt}
- Compacted lift thickness
- Shear strength parameters, c_u , ϕ_u or c' , and ϕ'
- Electro chemical properties of reinforced fill
 - For geosynthetic reinforcement: pH
 - For steel reinforcement: pH, resistivity, chlorides, sulfates, and organic content

9.2.4 Step 4. Evaluate design parameters for the reinforcement. (see recommendations in Chapter 3, Section 3.5.)

- Allowable geosynthetic strength (Eq. 3-12), $T_{al} = \text{ultimate strength } (T_{ULT}) \div \text{reduction factor (RF) for creep, installation damage and durability:}$

For granular reinforced fill meeting the recommended gradation in Chapter 3, and electrochemical properties in Chapter 3, $RF = 7$ may be conservatively used for preliminary design and for routine, noncritical structures where the minimum test requirements outlined in Table 3-12 are satisfied. **Remember, there is a significant cost advantage in obtaining lower RF from test data supplied by the manufacture and/or from agency evaluation!**

- Allowable steel strength (Eq. 3-11), $T_{al} = F_y A_c / b$, where A_c is the area of the steel adjusted for corrosion. **Note: Soils with higher fines are often more corrosive and Table 3-3 property requirements must be carefully checked for the reinforced fill.**
- Pullout Resistance: (See recommendations in Chapter 3 and Appendix B)
 - Use F.S. = 1.5 for granular soils
 - Use F.S. = 2 for cohesive soils
 - Minimum anchorage length, L_e , = 3 ft (1 m)

9.2.5 Step 5. Check unreinforced stability.
(see discussion in Chapter 8)

- a. Evaluate unreinforced stability to determine: if reinforcement is required; critical nature of the design (i.e., unreinforced F.S. $\leq$ or $\geq$ 1); potential deep-seated failure problems;

and the extent of the reinforced zone. Perform a stability analysis using conventional stability methods (see FHWA *Soils and Foundations Workshop Reference Manual* {Samtani and Nowatzki, 2006}) to determine safety factors and driving moments for potential failure surfaces.

- Use both circular-arc and sliding-wedge methods, and consider failure through the toe, through the face (at several elevations), and deep-seated below the toe.

(A number of stability analysis computer programs are available for rapid evaluation, e.g., FHWA sponsored programs including ReSSA and the STABL family of programs developed at Purdue University including the current version, STABL4M. In all cases, a few calculations should be made by hand to be sure the computer program is giving reasonable results.)

- b. Determine the size of the critical zone to be reinforced.
 - Examine the full range of potential failure surfaces found to have:
Unreinforced safety factor, $FS_U \leq$ Required safety factor, FS_R .
 - Plot all of these surfaces on the cross-section of the slope.
 - The surfaces that just meet the required safety factor roughly envelope the limits of the critical zone to be reinforced as shown in Figure 9-3.

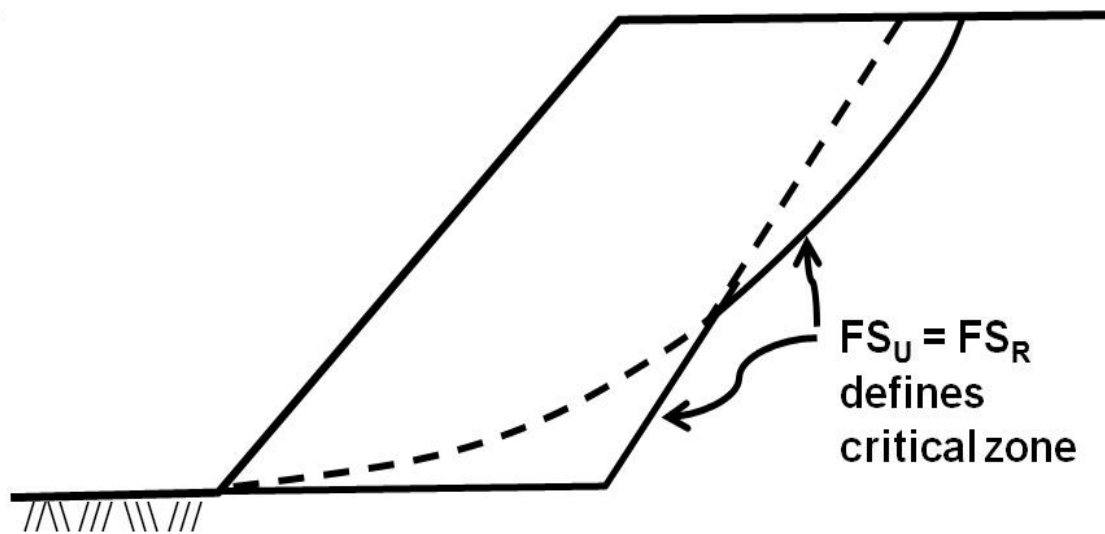


Figure 9-3. Critical zone defined by rotational and sliding surfaces that meets the required safety factor.

- c. Critical failure surfaces extending below the toe of the slope are indications of deep foundation and edge bearing capacity problems that must be addressed prior to completing the design. Where foundation problems are indicated, a more extensive foundation analysis is warranted, and foundation improvement measures should be considered as discussed in Chapter 8.

9.2.6 Step 6. Design reinforcement to provide a stable slope.

(see Figure 9-4, and discussion in Chapter 8.)

- a. Calculate the total reinforcement tension per unit width of slope T_S required to obtain the required factor of safety FS_R for each potential failure surface inside the critical zone in step 5 that extends through or below the toe of the slope using the following:

$$T_S = (FS_R - FS_U) \frac{M_D}{D} \quad (9-1)$$

where:

T_S = the sum of the required tensile force per unit width of reinforcement (considering rupture and pullout) in all reinforcement layers intersecting the failure surface

M_D = driving moment about the center of the failure circle

D = the moment arm of T_S about the center of failure circle

= radius of circle R for **continuous, sheet type extensible reinforcement** (i.e., assumed to act tangentially to the circle)

= radius of circle R for **continuous, sheet type inextensible reinforcement** (e.g., wire mesh reinforcement) to account for normal stress increase on adjacent soil

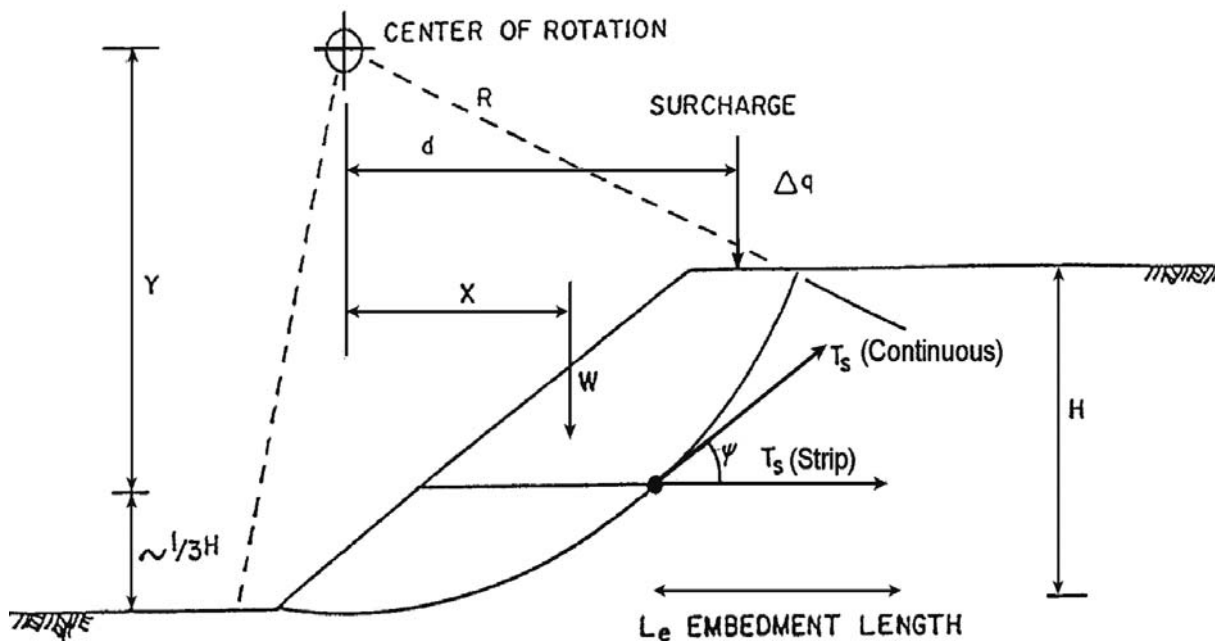
= vertical distance, Y , to the centroid of T_S for **discrete element, strip type reinforcement**. Assume $H/3$ above slope base for preliminary calculations (i.e. assumed to act in a horizontal plane intersecting the failure surface at $H/3$ above the slope base)

FS_U = unreinforced slope safety factor

FS_R = target minimum slope factor of safety which is applied to both the soil and reinforcement

T_{S-MAX} = the largest T_S calculated and establishes the total design tension

- **Note: the minimum unreinforced safety factor usually does not control the location of T_{S-MAX} ; the most critical surface is the surface requiring the greatest amount of reinforcement strength.**



Factor of safety of unreinforced slope:

$$F.S._u = \frac{\text{Resisting Moment } (M_R)}{\text{Driving Moment } (M_D)} = \frac{\int_0^{L_{sf}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)}$$

where: W = weight of sliding earth mass
 L_{sf} = length of slip plane
 Δq = surcharge
 τ_f = shear strength of soil

Factor of safety of reinforced slope

$$FS_R = FS_u + \frac{T_s + D}{M_D}$$

where: T_s = sum of available tensile force per width of reinforcement for all reinforcement layers
 D = moment arm of T_s about the center of rotation
 $= R$ for continuous extensible and inextensible reinforcement
 $= Y$ for discrete (e.g., strips) reinforcement

Figure 9-4. Rotational shear approach to determine required strength of reinforcement.

- b. Determine the total design tension per unit width of slope, T_{S-MAX} , using the charts in Figure 9-5 and compare with T_{S-MAX} from step 6a. If significantly different, check the validity of the charts based on the limiting assumptions listed in the figure and recheck calculations in steps 5 and equation 9-1.

- Figure 9-5 is provided for a quick check of computer-generated results. The figure presents a simplified method based on a two-part wedge type failure surface and is limited by the assumptions noted on the figure.

Note that Figure 9-5 is not intended to be a single design tool. Other design charts available from the literature could also be used (e.g., Ruegger, 1986; Leshchinsky and Boedeker, 1989; Jewell, 1990). As indicated in Chapter 8, several computer programs are also available for analyzing a slope with given reinforcement and can be used as a check. Judgment in selection of other appropriate design methods (i.e., most conservative or experience) is required.

- c. Determine the distribution of reinforcement:

- For low slopes ($H \leq 20$ ft {6 m}) assume a uniform reinforcement distribution and use T_{S-MAX} to determine spacing or the required tension T_{MAX} requirements for each reinforcement layer.
- For high slopes ($H > 20$ ft {6 m}), either a uniform reinforcement distribution may be used (preferable) or the slope may be divided into two (top and bottom) or three (top, middle, and bottom) reinforcement zones of equal height using a factored T_{S-MAX} in each zone for spacing or design tension requirements (see Figure 9-6). The total required tension in each zone is found from:

For 1 zone:

Use T_{S-MAX}

For 2 zones:

$$T_{Bottom} = 3/4 T_{S-MAX} \quad (9-2)$$

$$T_{Top} = 1/4 T_{S-MAX} \quad (9-3)$$

For 3 zones:

$$T_{Bottom} = 1/2 T_{S-MAX} \quad (9-4)$$

$$T_{Middle} = 1/3 T_{S-MAX} \quad (9-5)$$

$$T_{Top} = 1/6 T_{S-MAX} \quad (9-6)$$

The force is assumed to be uniformly distributed over the entire zone.

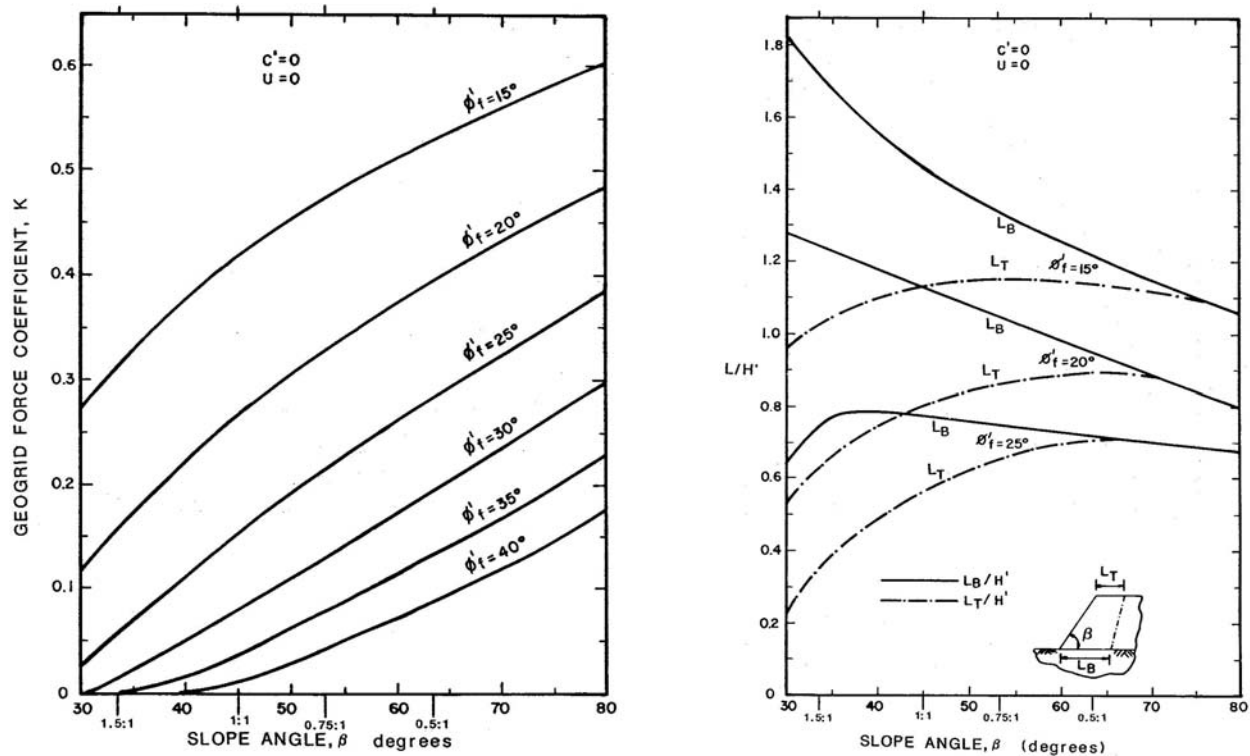


CHART PROCEDURE:

- 1) Determine force coefficient K from figure above, where ϕ_r = friction angle of reinforced fill:

$$\phi_r = \tan^{-1} \left(\frac{\tan \phi_f}{FR_R} \right)$$

- 2) Determine:

$$T_{S-MAX} = 0.5 K \gamma_r (H')^2$$

where: $H' = H + q/\gamma_r$

q = a uniform load

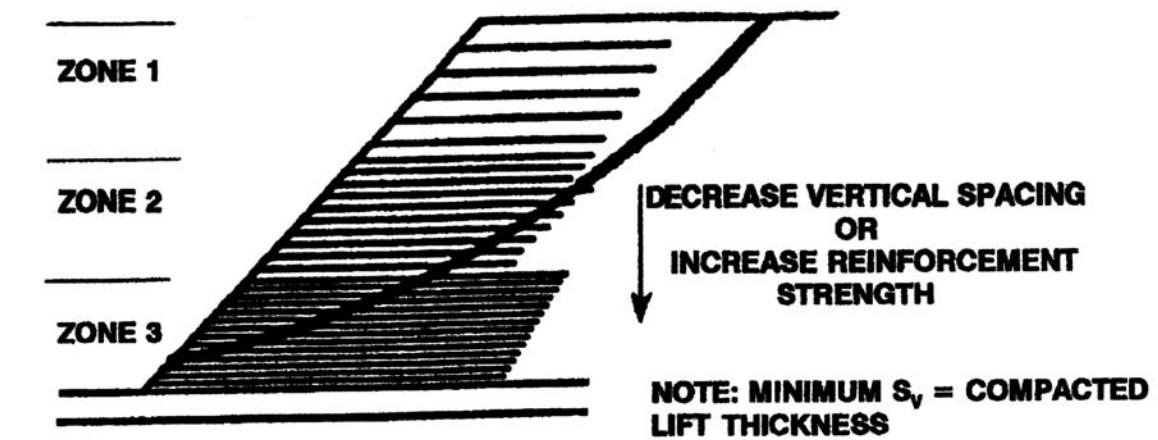
- 3) Determine the required reinforcement length at the top L_T and bottom L_B of the slope from the figure above.

LIMITING ASSUMPTIONS

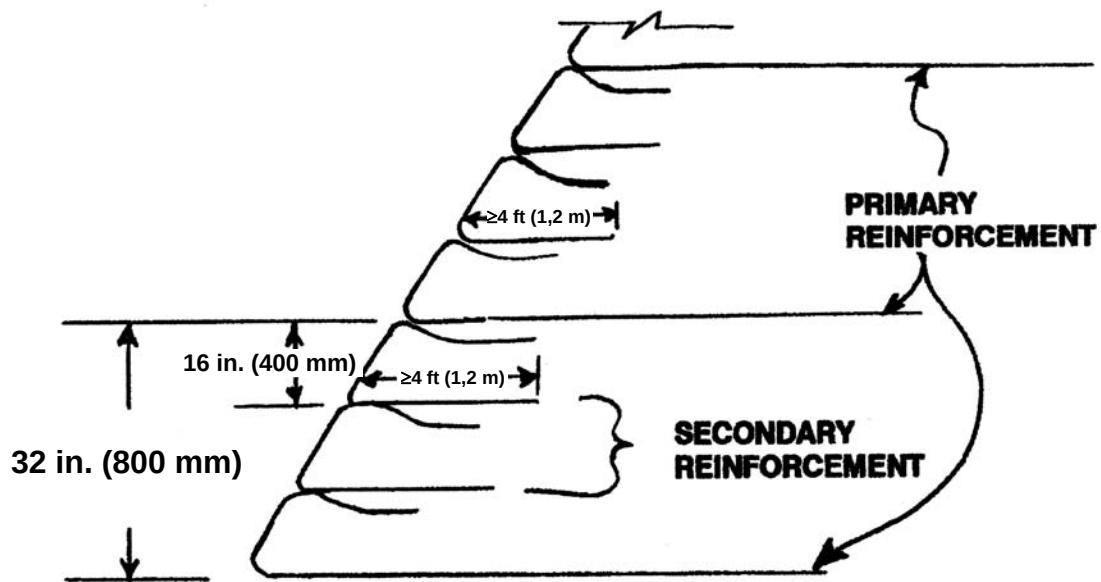
- Extensible reinforcement
- Slopes constructed with uniform, cohesionless soil, $c = 0$
- No pore pressures within slope
- Competent, level foundation soils
- No seismic forces
- Uniform surcharge not greater than $0.2 \gamma_r H$
- Relatively high soil/reinforcement interface friction angle, $\phi_{sg} = 0.9 \phi_r$ (may not be appropriate for some geosynthetics)

Figure 9-5. Chart solution for determining the reinforcement strength requirements (after Schmertmann et. al., 1987).

NOTE: Charts © The Tensar Corporation



A) SPACING VERSUS REINFORCEMENT STRENGTH



B) PRIMARY AND SECONDARY REINFORCEMENT APPROACH

Figure 9-6. Reinforcement spacing considerations for high slopes.

d. Determine reinforcement vertical spacing S_v or the maximum design tension T_{MAX} requirements for each reinforcement layer.

- For each zone, calculate T_{MAX} for each reinforcing layer in that zone based on an assumed S_v or, if the allowable reinforcement strength is known, calculate the minimum vertical spacing and number of reinforcing layers N required for each zone based on:

$$T_{max} = \frac{T_{zone} S_v}{H_{zone}} = \frac{T_{zone}}{N} \leq T_{al} R_c \quad (9-7)$$

where:

R_c = coverage ratio of the reinforcement which equals the width of the reinforcement b divided by the horizontal spacing S_h

S_v = vertical spacing of reinforcement in meters; multiples of compacted layer thickness for ease of construction

T_{zone} = maximum reinforcement tension required for each zone

= T_{S-MAX} for low slopes ($H < 6m$)

T_{al} = T_{ult} / RF (see Chapter 3 and equation 3-12)

H_{zone} = height of zone

= T_{top} , T_{middle} , and T_{Bottom} for high slopes ($H > 20 \text{ ft } \{6 \text{ m}\}$)

N = number of reinforcement layers

- Use short 4 to 6.5 ft (1.2 to 2 m) lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 16 in. (400 mm) or less for face stability and compaction quality (see Figure 9-6b).
 - For slopes flatter than 1H:1V, closer spaced reinforcements (i.e., every lift or every other lift, but no greater than 16 in. {400 mm}) preclude having to wrap the face in well graded soils (e.g., sandy gravel and silty and clayey sands). Wrapped faces are required for steeper slopes and uniformly graded soils to prevent face sloughing. Alternative vertical spacings could be used to prevent face sloughing, but in these cases a face stability analysis should be performed either using the method presented in this chapter or by evaluating the face as an infinite slope using (Collin, 1996):

$$F.S. = \frac{c'H + (\gamma_g - \gamma_w)Hz \cos^2\beta \tan\phi' + F_g (\cos\beta \sin\beta + \sin^2\beta \tan\phi')}{\gamma_g H z \cos\beta \sin\beta} \quad (9-8)$$

where: c' = effective cohesion
 ϕ' = effective friction angle
 γ_g = saturated unit weight of soil
 γ_w = unit weight of water
 z = vertical depth to failure plane defined by the depth of saturation
 H = vertical slope height
 β = slope angle
 F_g = summation of geosynthetic resisting force

- Intermediate reinforcement should be placed in continuous layers and needs not be as strong as the primary reinforcement, but it must be strong enough to survive construction (e.g. minimum survivability requirements for geotextiles in road stabilization applications in AASHTO M-288, 2006) and provide localized tensile reinforcement to the surficial soils.
 - If the interface friction angle of the intermediate reinforcement ρ_{sr} (from ASTM D 5321 or estimated as discussed in Chapter 3, Section 3.4.3) is less than that of the primary reinforcement ρ_r , then ρ_{sr} should be used in the analysis for the portion of the failure surface intersecting the reinforced soil zone.
- e. To ensure that the rule-of-thumb reinforcement force distribution is adequate for critical or complex structures, recalculate T_S using equation 9-1 to determine potential failure above each layer of primary reinforcement.
- f. Determine the reinforcement lengths required:
- The embedment length L_e of each reinforcement layer beyond the most critical sliding surface (i.e., circle found for T_{S-MAX}) must be sufficient to provide adequate pullout resistance based on:

$$L_e = \frac{T_{max} FS}{F^* \alpha \sigma'_v R_c C} \quad (9-9)$$

where F^* , α , R_c , C and σ'_v are defined in Chapter 3, Section 3.4.

- Minimum value of L_e is 3 ft (1 m). For cohesive soils, check L_e for both short- and long-term pullout conditions, when using the semi empirical equations in Chapter 3 to obtain F^* .

For long-term design, use ϕ'_r with $c'_r = 0$

For short-term evaluation, conservatively use ϕ_r with $c_r = 0$ from consolidated undrained triaxial or direct shear tests or run pullout tests

- Plot the reinforcement lengths as obtained from the pullout evaluation on a slope cross section containing the rough limits of the critical zone determined in step 5 (see Figure 9-7).
 - The length required for sliding stability at the base will generally control the length of the lower reinforcement levels.
 - Lower layer lengths must extend at least to the limits of the critical zone as shown in Figure 9-7. Longer reinforcements may be required to resolve deep seated failure problems (see step 7).
 - Upper levels of reinforcement may not be required to extend to the limits of the critical zone, provided sufficient reinforcement exists in the lower levels to provide the FS_R for all circles within the critical zone as shown in Figure 9-7.
- Check that the sum of the reinforcement forces passing through each failure surface is greater than T_s required for that surface.
 - Only count reinforcement that extends 3 ft (1 m) beyond the surface to account for pullout resistance.
 - If the available reinforcement force is not sufficient, increase the length of reinforcement not passing through the surface or increase the strength of lower-level reinforcement.
- Simplify the layout by lengthening some reinforcement layers to create two or three sections of equal reinforcement length for ease of construction and inspection.
- Reinforcement layers do not generally need to extend to the limits of the critical zone, except for the lowest levels of each reinforcement section.

- Check the length obtained using chart b in Figure 9-5. Note: L_e is already included in the total length, L_t and L_B from chart B.
- g. Check design lengths of complex designs.
- When checking a design that has zones of different reinforcement length, lower zones may be over reinforced to provide reduced lengths of upper reinforcement levels.
 - In evaluating the length requirements for such cases, the pullout stability for the reinforcement must be carefully checked in each zone for the critical surfaces exiting at the base of each length zone.

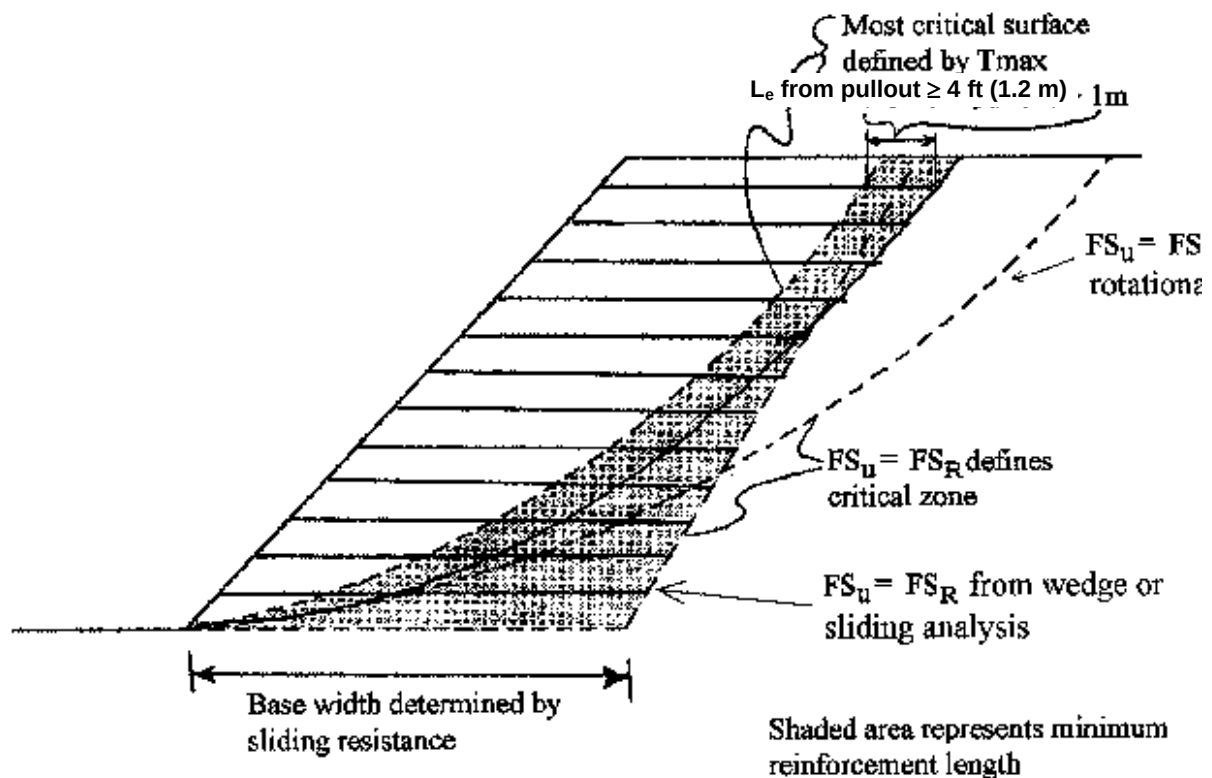


Figure 9-7. Developing reinforcement lengths.

9.2.7 Step 7. Check external stability. (see discussion in Chapter 8.)

- Sliding resistance (Figure 9-8)
 - Evaluate the width of the reinforced soil zone **at any level** to resist sliding along the reinforcement. Use a two-part wedge type failure surface defined by the limits of the reinforcement (the length of the reinforcement at the depth of evaluation defined in step 5). The analysis can best be performed using a computerized method which takes into account all soil strata and interface friction values. If the computer program does not account for the presents of reinforcement, the back of the failure surface should be angled at $45 + \phi/2$ or parallel to the back of the reinforced zone, whichever is flatter (i.e., the wedge should not pass through layers of reinforcement to avoid an overly conservative analysis). The frictional resistance provided by the weakest layer, either the reinforced soil, the foundation soil or the soil-reinforcement interface, should be used in the analysis.

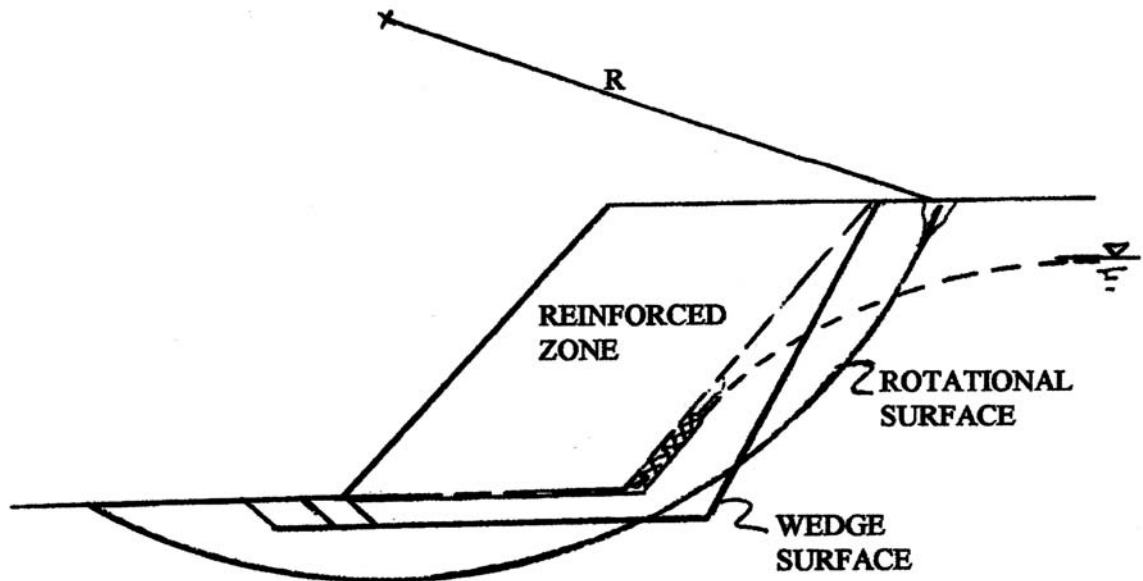
A simple analysis using Figure 9-5b can be performed as a quick check, but should not be used for the primary analysis due to the limiting assumptions noted on the figure. The method also assumes that the reinforcement layers are truncated along a plane parallel to the slope face, which may or may not be the case. The analysis was based on a two-part wedge model to predict L_B assuming that the reinforcement interface is the weakest plane. A reduction is applied to the interface friction angle, $\phi_{sg} = 0.9 \phi_r$, which may not be appropriate for some geosynthetics. The frictional resistance provided by the weakest layer in contact with the geosynthetic, either the reinforced soil or the foundation soil, should be used in the analysis.

- Deep seated global stability (Figure 9-8a).
 - Evaluate potential deep-seated failure surfaces behind the reinforced soil zone to provide:

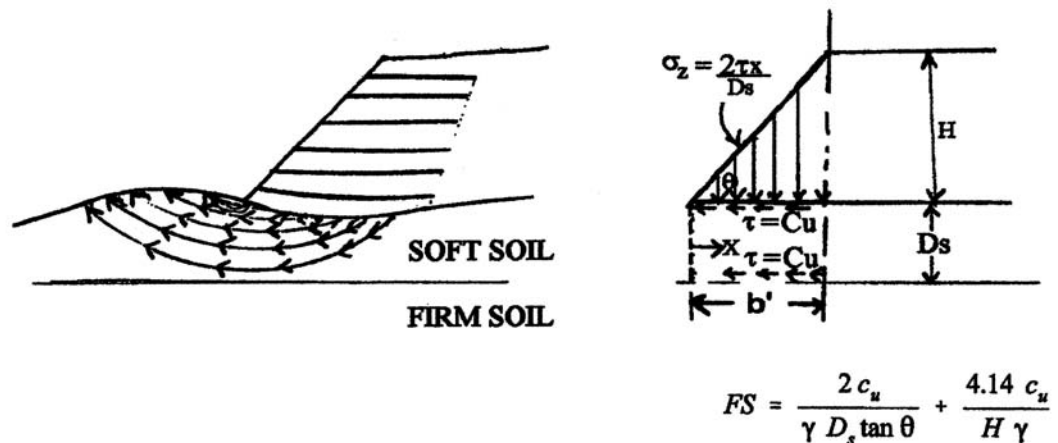
$$F.S. = \frac{M_R}{M_D} \geq 1.3 \text{ minimum} \quad (9-14)$$

Note: $F.S. \geq 1.3$ is recommended as a minimum and that value should be increased based on the criticality of the slope (e.g., slopes beneath bridge abutments and major roadways) and/or confidence in geotechnical conditions (e.g., soil properties and location of groundwater).

The analysis performed in step 5 should provide the factor of safety for failure surfaces behind the reinforced soil zone. However, as a check, classical rotational slope stability methods such as simplified Bishop, Morgenstern and Price, Spencer, or others may be used (see FHWA's Ground Improvement Manuals, FHWA NHI-06-019 and FHWA NHI-06-020 {Elias et al., 2006}). Appropriate computer programs also may be used.



a) Deep seated (global) stability analysis.



b) Local bearing failure (lateral squeeze)

Figure 9-8. Failure through the foundation.

- Local bearing failure at the toe (lateral squeeze) (Figure 9-8b).
 - If a weak soil layer exists beneath the embankment to a limited depth D_s which is less than the width of the slope b' , the factor of safety against failure by squeezing may be calculated from (Silvestri, 1983):

$$FS_{\text{squeezing}} = \frac{2c_u}{\gamma D_s \tan \theta} + \frac{4.14c_u}{H \gamma} \geq 1.3 \quad (9-15)$$

where:

- θ = angle of slope
- γ = unit weight of soil in slope
- D_s = depth of soft soil beneath slope base of the embankment
- H = height of slope
- c_u = undrained shear strength of soft soil beneath slope

Caution is advised and rigorous analysis (e.g., numerical modeling) should be performed when $FS < 2$. This approach is somewhat conservative as it does not provide any influence from the reinforcement. When the depth of the soft layer, D_s , is greater than the slope base width, b' , general slope stability will govern design.

- Foundation settlement.
 - Determine the magnitude and rate of total and differential foundation settlements using classical geotechnical engineering procedures (see FHWA *Soils and Foundations Workshop Reference Manual*, {Samtani and Nowatzki, 2006}).

9.2.8 Step 8. Seismic stability.

- Dynamic stability (Figure 9-9).
 - Perform a pseudo-static type analysis using a seismic ground coefficient A , obtained from local building code and a design seismic acceleration A_m equal to $A_m = A/2$. Reinforced soil slopes are clearly yielding type structures, more so than walls. As such, A_m can be taken as $A/2$ as allowed by AASHTO in Division 1A-Seismic Design, 6.4.3 Abutments (AASHTO, 2002) and Appendix A11.1.1.2 (AASHTO, 2007)

$$F.S. \text{ dynamic} \geq 1.1$$

In the pseudo-static method, seismic stability is determined by adding a horizontal and/or vertical force at the centroid of each slice to the moment equilibrium equation (see Figure 9-9). The additional force is equal to the seismic coefficient times the total weight of the sliding mass. It is assumed that this force has no influence on the normal force and resisting moment, so that only the driving moment is affected. The liquefaction potential of the foundation soil should also be evaluated.

9.2.9 Step 9. Evaluate requirements for subsurface and surface water runoff control.

- Subsurface water control.
 - Design of subsurface water drainage features should address flow rate, filtration, placement, and outlet details.
 - Drains are typically placed at the rear of the reinforced zone as shown in Figure 9-10. Geocomposite drainage systems or conventional granular blanket and trench drains could be used (see Chapter 5).

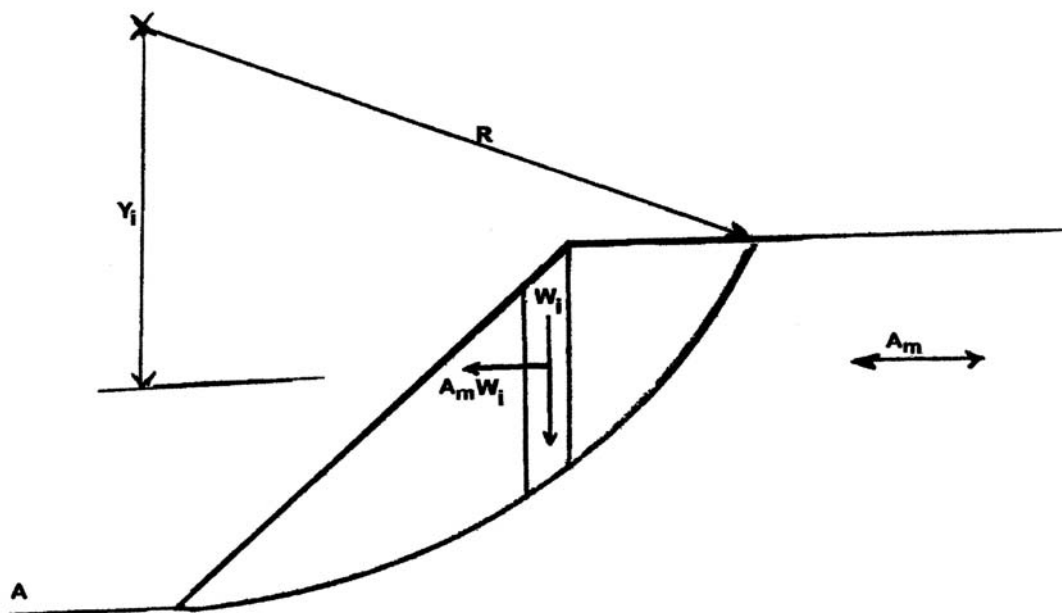
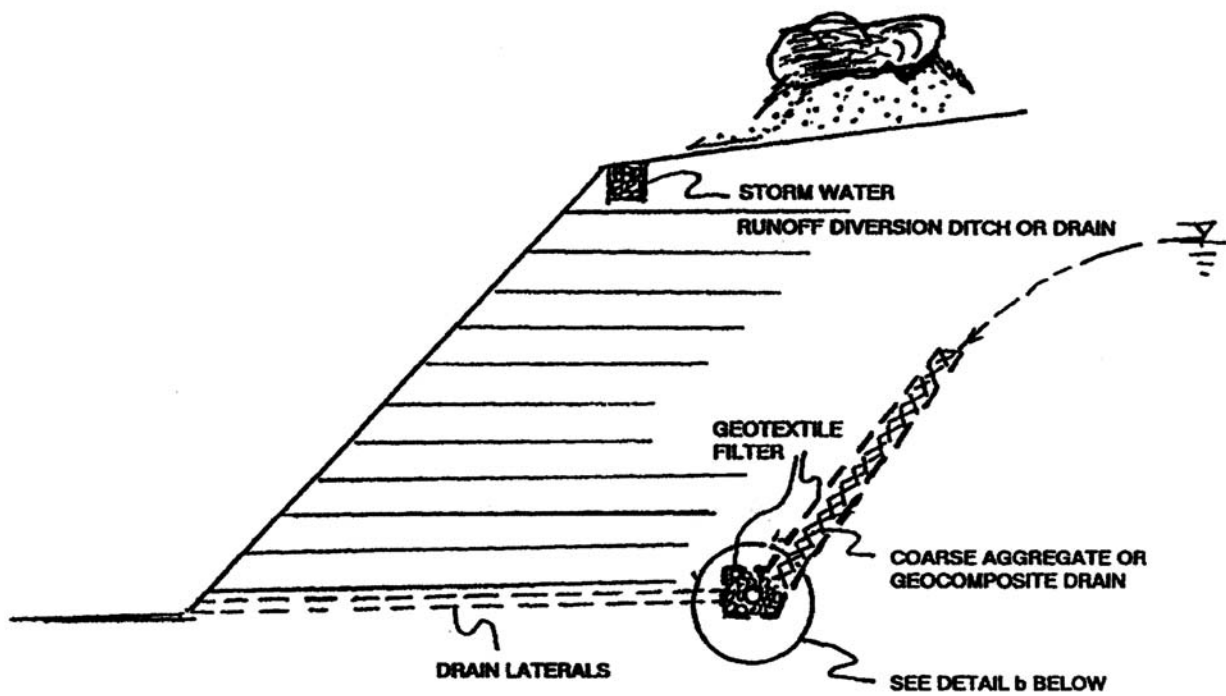
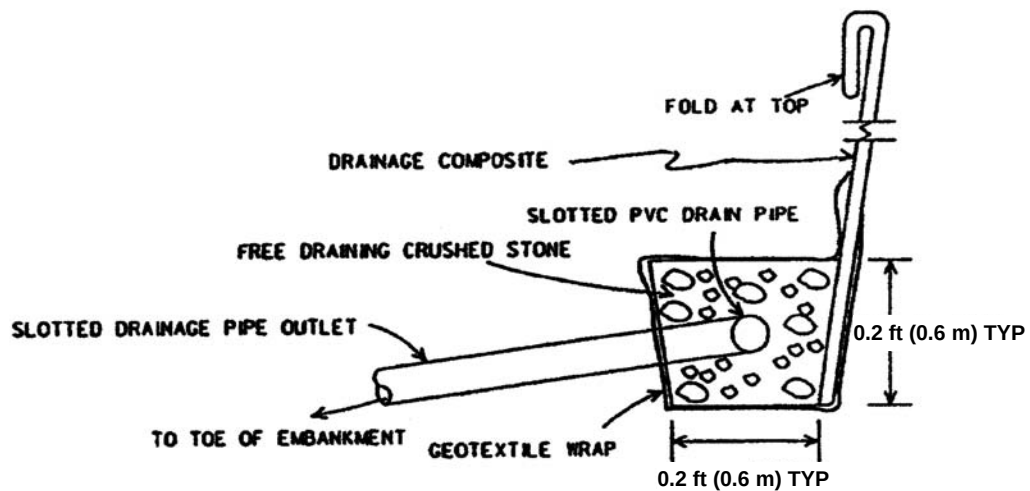


Figure 9-9. Seismic stability analysis.



a) GROUND WATER AND SURFACE DRAINAGE



b) TYPICAL DRAIN DETAILS

Figure 9-10. Subsurface drainage considerations.

- Lateral spacing of outlets is dictated by site geometry, estimated flow, and existing agency standards. Outlet design should address long-term performance and maintenance requirements.
- Geosynthetic drainage composites can be used in subsurface water drainage design. Drainage composites should be designed with consideration of:
 - o Geotextile filtration/clogging
 - o Long-term compressive strength of polymeric core
 - o Reduction of flow capacity due to intrusion of geotextile into the core
 - o Long-term inflow/outflow capacity

Procedures for checking geotextile permeability and filtration/clogging criteria were presented in FHWA *Geosynthetic Design and Construction Guidelines* (Holtz et al., 2008). Long-term compressive stress and eccentric loadings on the core of a geocomposite should be considered during design and selection. Though not yet addressed in standardized test methods or standards of practice, the following criteria are suggested for addressing core compression. The design pressure on a geocomposite core should be limited to either:

- o the maximum pressure sustained on the core in a test of 10,000 hours minimum duration
- o the crushing pressure of a core, as defined with a quick loading test, divided by a factor of safety of 5

Note that crushing pressure can only be defined for some core types. For cases where a crushing pressure cannot be defined, suitability should be based on the maximum load resulting in a residual thickness of the core adequate to provide the required flow after 10,000 hours, or the maximum load resulting in a residual thickness of the core adequate to provide the required flow as defined with the quick loading test divided by a factor of safety of 5.

Intrusion of the geotextiles into the core and long-term outflow capacity should be measured with a sustained transmissivity test. The ASTM D4716 test procedure *Constant Head Hydraulic Transmissivity of Geotextiles and Geotextile Related Products*, should be followed. The test procedure should be modified for sustained testing and for use of sand sub-stratum and super-stratum

in lieu of closed cell foam rubber. Load should be maintained for 100 hours or until equilibrium is reached, whichever is greater.

- Slope stability analyses should account for interface shear strength along a geocomposite drain. The geocomposite/soil interface will most likely have a friction value that is lower than that of the soil. Thus, a potential failure surface may be induced along the interface.
- Geotextile reinforcements (primary and intermediate layers) must be more permeable than the reinforced fill material to prevent a hydraulic build up above the geotextile layers during precipitation.

Special emphasis on the design and construction of subsurface drainage features is recommended for structures where drainage is critical for maintaining slope stability. Redundancy in the drainage system is also recommended for these cases.

- Surface water runoff.
 - Surface water runoff should be collected above the reinforced slope and channeled or piped below the base of the slope. Standard Agency drainage details should be utilized.
 - Wrapped faces and/or intermediate layers of secondary reinforcement may be required at the face of reinforced slopes to prevent local sloughing. Guidance is provided in Chapter 8 and Table 8-1. Intermediate layers of reinforcement help achieve compaction at the face, thus increasing soil shear strength and erosion resistance. These layers also act as reinforcement against shallow or sloughing types of slope failures. Intermediate reinforcement is typically placed on each or every other soil lift, except at lifts where primary structural reinforcement is placed. Intermediate reinforcement also is placed horizontally, adjacent to primary reinforcement, and at the same elevation as the primary reinforcement when primary reinforcement is placed at less than 100 percent coverage in plan view. The intermediate reinforcement should extend 4 to 6.5 ft (1.2 to 2 m) back into the fill from the face.
 - Select a long-term facing system to prevent or minimize erosion due to rainfall and runoff on the face.

- Calculate flow-induced tractive shear stress on the face of the reinforced slope by:

$$\lambda = d \cdot \gamma_w \cdot s \quad (9-16)$$

where:

- λ = tractive shear stress, psf (kPa)
- d = depth of water flow, ft (m)
- γ_w = unit weight of water, lbs/ft³ (kN/m³)
- s = the vertical to horizontal angle of slope face, ft/ft (m/m)

For $\lambda < 2$ psf (100 Pa), consider vegetation with temporary or permanent erosion control mat

For $\lambda > 2$ psf (100 Pa), consider vegetation with permanent erosion control mat or other armor type systems (e.g., riprap, gunite, prefab modular units, fabric-formed concrete, etc.)

- Select vegetation based on local horticultural and agronomic considerations and maintenance.
- Select a synthetic (permanent) erosion control mat that is stabilized against ultra-violet light and is inert to naturally occurring soil-born chemicals and bacteria. Erosion control mats and blankets vary widely in type, cost, and, more importantly, applicability to project conditions. **Slope protection should not be left to the construction contractor or vendor's discretion.** Guideline material specifications for synthetic permanent erosion control mats are provided in Chapter 10.

9.3 COMPUTER ASSISTED DESIGN

An alternative to reinforcement design (step 6 in the previous section) is to develop a trial layout of reinforcement and analyze the reinforced slope with a computer program such as the FHWA ReSSA program. Layout includes number, length, design strength, and vertical distribution of the geosynthetic reinforcement. The charts presented in Figure 9-5 provide a method for generating a preliminary layout. Note that these charts were developed with the specific assumptions noted on the figure.

Analyze the reinforced soil slope with the trial geosynthetic reinforcement layouts. **The most economical reinforcement layout must provide the minimum required stability safety factors for internal, external, and compound failure planes.** A contour plot of lowest safety factor values about the trial failure circle centroids is recommended to map and locate the minimum safety factor values for the three modes of failure.

The method of analysis in section 9.2 assumes that the reinforcing force contributed to the resisting moment and thus inherently applies the required factor of safety to the reinforcement. However, some computer programs (and design charts) are based on the assumption that the reinforcement force reduces the driving moment with the stability factor of safety FS calculated as:

$$FS = \frac{M_R}{M_D - T_s D} \quad (9-17)$$

With this assumption, the stability factory of safety is not applied to the reinforcement. For such computations and any other methods not applying a factor of safety to the reinforcement, the allowable strength of the reinforcement T_{al} must be divided by a required minimum factor of safety $FS_R = 1.3$ to provide an equivalent material uncertainty.

External stability analysis as was previously shown in step 7 will include an evaluation of local bearing capacity, foundation settlement, and dynamic stability.

9.4 PROJECT COST ESTIMATES

Cost estimates for reinforced slope systems are generally per square foot of vertical face. Table 9-1 can be used to develop a cost estimate. As an example, the following provides a cost estimate for the 6.5 ft (5 m) high reinforced slope design Example E.8. Considering the 12 layers of reinforcement at a length of 16 ft (4.9 m), the reinforced section would require a total reinforcement of 192 ft² per ft (60 m² per meter) length of embankment or 12 ft² per vertical ft of height (12 m² per vertical meter of height). Adding 10 to 15 percent for overlaps and overages results in an anticipated reinforcement quantity of 13.5 ft² per ft (13.5 m² per meter embankment height). Based on the cost information from suppliers, reinforcement with an allowable strength $T_a \geq 280$ lb/ft (4.14 kN/m) would cost on the order of \$0.10 to 0.15/ft² (\$1.00 to \$1.50/m²). Assuming \$0.05 ft² (\$0.50 m²) for handling and placement, the in-place cost of reinforcement would be approximately \$2.50/ft² (\$25/m²) of vertical embankment face. Approximately 24.6 yd³ (18.8 m³) of additional fill would be required for the reinforced section per foot (per meter) of embankment length. Using a typical in-place cost for locally

available fill with some hauling of 6 yd^3 ($\$8/\text{m}^3$) (about $\$4$ per 1000 kg), $\$2.80/\text{ft}^2$ ($\$30/\text{m}^2$) will be added to the cost. In addition, overexcavation and backfill of existing embankment material will be required to allow for placement of the reinforcement. Assuming $\$1.50/\text{yd}^3$ ($\$2/\text{m}^3$) for overexcavation and replacement will add approximately $\$0.40/\text{ft}^2$ ($\$4/\text{m}^2$) of vertical face. The erosion protection for the face would also add a cost of $\$0.50/\text{ft}^2$ ($\$5/\text{m}^2$) of vertical face plus seeding and mulching. Thus, the total estimated cost for this option would be on the order of $\$6/\text{ft}^2$ ($\$64/\text{m}^2$) of vertical embankment face. Alternative facing systems such as soil bioengineered treatment and/or the use of wire baskets for face would each add approximately $\$2$ to $\$3/\text{ft}^2$ ($\$20$ to $\$30/\text{m}^2$) to the construction costs, but reduction in long-term maintenance will most likely offset these costs.

Table 9-1. Estimated Project Costs.

Item	Total Volume	Unit Cost	Extension	per Vertical square foot (meter)
Reinforced fill (in place)	$\text{yd}^3 (\text{m}^3)$			
Overexcavation	$\text{yd}^3 (\text{m}^3)$			
Reinforcement (in place)	$\text{yd}^2 (\text{m}^2)$			
Facing system	$\text{yd}^2 (\text{m}^2)$			
Support				
Vegetation				
Permanent erosion control mat				
Alternate facing systems	$\text{ft}^2 (\text{m}^2)$			
Groundwater control system	$\text{ft}^2 (\text{m}^2)$			
Guardrail	$\text{ft} (\text{m})$			
Total	-----	-----		
Unit cost per vertical $\text{ft}^2 (\text{m}^2)$				

Note Slope Dimensions:

Height H =

Length L =

Face Surface Area, A

Reinforcement Area = $L_{\text{reinforcement}} \times \text{Number of Layers}$

CHAPTER 10

CONTRACTING METHODS AND SPECIFICATIONS FOR MSE WALLS AND SLOPES

From its introduction in the early 1970s, it is estimated that the total construction value of MSE walls is in excess of \$2 billion. This estimate does not include reinforced slope construction, for which estimates are not available.

Since the early 1980s, hundreds of millions of dollars have been saved on our Nation's highways by bidding alternates for selection of earth retaining structures. During that time, the number of available MSE systems or components have increased, and some design and construction problem areas that have been identified. These include misapplication of wall technology; poor specifications; lack of specification enforcement; inequitable bidding procedures; poor construction techniques; inadequate inspection; and inconsistent selection, review, and acceptance practices on the part of public agencies. Although the actual causes of each particular problem are unique, Agency procedures that address the design and construction of earth retaining systems Can, when well formulated and enforced, minimize such problems; or when not well formulated or enforced, can contribute to such problems.

MSE wall and RSS systems are contracted using two different approaches:

- Agency or material supplier designs with system components, drainage details, erosion measures, and construction execution explicitly specified in the contracting documents; or
- Performance or end-result approach using approved or generic systems or components, with lines and grades noted on the drawings and geometric and design criteria specified. In this case, a project-specific design review and detail plan submittal occurs in conjunction with working drawing submittal.

Some user agencies prefer one approach to the other or a mixed use of approaches developed based upon criticality of a particular structure. Both contracting approaches are valid if properly implemented. Each approach has advantages and disadvantages.

This chapter will outline the necessary elements of each contracting procedure, the approval process, and current material and construction specifications.

While this chapter specifically addresses the need for formal policy and procedures for MSE and RSS structures, the recommendations and need for uniformity of practice applies to all types of retaining structures.

10.1 POLICY DEVELOPMENT

It is desirable that each Agency develop a formal policy with respect to design and contracting of MSE wall and RSS systems.

The general objectives of such a policy are to:

- Obtain uniformity within the Agency.
- Establish standard policies and procedures for design technical review and acceptance of MSEW and RSS systems or components.
- Establish a policy for the review/acceptance of new retaining wall and reinforced slope systems and or components.
- Delineate responsibility in house for the preparation of plans, design review and construction control.
- Delineate design responsibility for plans prepared by consultants and material suppliers.
- Develop design and performance criteria standards to be used on all projects.
- Develop and or update material and construction specifications to be used on all projects.
- Establish contracting procedures by weighing the advantages/disadvantages of prescriptive or end-result methods.

10.2 SYSTEM OR COMPONENT APPROVALS

The recent expiration of most process or material patents associated with MSE systems has led to introduction by numerous suppliers of a variety of complete systems or components that are applicable for use. Alternatively, it opens the possibility of Agency-generic designs that may incorporate proprietary and generic elements.

Approval of systems or components is a highly desirable feature of any policy for reinforced soil systems prior to their inclusion during the design phase, or as part of a value engineering alternate.

For the purpose of prior approval, it is desirable that the supplier submit data that satisfactorily addresses the following items as a minimum:

- System development or component and year it was commercialized.
- Systems or component supplier organizational structure, specifically engineering and construction support staff.
- Limitations and disadvantages of system or component.
- Prior list of users including contact persons, addresses and telephone numbers.

- Sample material and construction control specifications showing material type, quality, certifications, field testing, acceptance and rejection criteria and placement procedures.
- A documented field construction manual describing in detail, with illustrations as necessary, the step-by-step construction sequence and the contractor's quality control plan.
- Detailed design calculations for typical applications in conformance with current practice or AASHTO, whenever applicable.
- Typical unit costs, supported by data from actual projects.
- Independent performance evaluations of a typical project by a professional engineer.

The development, submittal, and approval of such a technical package provides a complete bench-mark for comparison with systems that have been in successful use and a standard when checking project-specific designs.

Some vendor wall systems have been reviewed, and others are currently being reviewed, under the HITEC program (see Section 1.2). The HITEC program is still available within the American Society of Civil Engineers (ASCE) organization. Wall system suppliers are encouraged to conduct an independent review of newly developed components and/or systems related to materials, design, construction, performance, and quality assurance for use by DOTs in their system approval process.

For the purpose of review and approval of geosynthetics (systems or components) used for reinforcement applications, the manufacturer/supplier submittal must satisfactorily address the following items that are related to the establishment of a long-term allowable tensile strength used in design:

- Laboratory test results documenting creep performance over a range of load levels for minimum duration of 10,000 hr. in accordance with ASTM D5262.
- Laboratory test results and methodology for extrapolation of creep data for 75- and 100-year design life as described in Appendix D.
- Laboratory test results documenting ultimate strength in accordance with ASTM D4595 for geotextiles or ASTM D6637 for geogrids. Tests to be conducted at a strain rate of 10 percent per minute.
- Laboratory test results and extrapolation techniques, documenting the hydrolysis resistance of polyester (PET), oxidative resistance of polypropylene (PP) and high density polyethylene (HDPE), and stress cracking resistance of HDPE for all components of geosynthetic and values for partial factor of safety for aging degradation calculated for a 75- and 100-year design life. Recommended methods are outlined in FHWA RD 97-144 (Elias et al., 1999).

- Field and laboratory test results along with literature review documenting reduction factor values for installation damage as a function of backfill gradation.
- For projects where a potential for biological degradation exists, laboratory test results and extrapolation techniques, documenting biological resistance of all material components of the geosynthetic and values for a reduction factor for biological degradation.
- Laboratory test results documenting joint (seams and connections) strength (ASTM D4884 and GRI:GG2).
- Laboratory tests documenting pullout interaction coefficients for various soil types or site-specific soils in accordance with ASTM D6706.
- Laboratory tests documenting direct sliding coefficients for various soil types or project specific soils in accordance with ASTM D5321.
- Manufacturing quality control program and data indicating minimum test requirements, test methods, test frequency, and lot size for each product. Further minimum conformance requirements as proscribed by the manufacturer shall be indicated. The following is a minimum list of conformance criteria required for approval:

<u>Test</u>	<u>Test Procedure</u>	<u>Minimum Conformance Requirement</u>
Wide Width Tensile (geotextiles)	ASTM D4595	<i>To be provided by material supplier or specialty company</i>
Specific Gravity (HDPE only)	ASTM D1505	
Melt Flow index (PP & HDPE)	ASTM D1238	
Intrinsic Viscosity (PET only)	ASTM D4603	
Carboxyl End Group (PET only)	ASTM D2455	
Single Rib Tensile (geogrids)	ASTM D6637	

- The primary resin used in manufacturing shall be identified as to its ASTM type, class, grade, and category.

For HDPE resin type, class, grade and category in accordance with ASTM D1248 shall be identified. For example type III, class A, grade E5, category 5.

For PP resins, group, class and grade in accordance with ASTM D4101 shall be identified. For example group 1, class 1, grade 4.

For PET resins minimum production intrinsic viscosity (ASTM4603) and maximum carboxyl end groups (ASTM D2455) shall be identified.

For all products the minimum UV resistance as measured by ASTM D4355 shall be identified.

Prior approval should be based on Agency evaluations with respect on the following:

- The conformance of the design method and construction specifications to current Agency requirements for MSE walls and RSS slopes and any deviations to current engineering practice. For reinforced slope systems, conformance to current geotechnical practice.
- Past experience in construction and performance of the proposed system.
- The adequacy of the data in support of nominal long-term strength (T_{al}) for geosynthetic reinforcements.
- The adequacy of the QA/QC plan for the manufacture of geosynthetic reinforcements.

10.3 DESIGN AND PERFORMANCE CRITERIA

It is highly desirable that each Agency formalize its design and performance criteria as part of a design manual that may be incorporated in the *Bridge Design Manual* under *Retaining Structures for MSE walls* and/or a *Highway Design Manual* for reinforced slope structures. This would ensure that all designs whether Agency/Consultant or Supplier prepared, are based on equal, sound principles.

The design manual may adopt current AASHTO LRFD Bridge Design Specifications (2007) Section 11.10 *Mechanically Stabilized Earth Walls*, or methods outlined in this manual as a primary basis for design and performance criteria and list under appropriate sections any deviations, additions and clarification to this practice that are relevant to each particular Agency, based on its experience. Construction material specifications for MSE walls may be modeled on Section 7 of current AASHTO LRFD Bridge Construction Specifications (2004), *Earth Retaining Systems*, or the complete specifications contained in this chapter.

With respect to reinforced slope design, the performance criteria should be developed based on data outlined in Chapter 9. Material and construction specifications for RSS are provided in this chapter as well as for drainage and erosion control materials usually required for such construction.

10.4 AGENCY OR SUPPLIER DESIGN

This contracting approach includes the development of a detailed set of MSE wall or RSS slope plans and material specifications in the bidding documents.

The advantage of this approach is that the complete design, details, and material specifications can be developed and reviewed over a much longer design period. This approach further empowers Agency engineers to examine more options during design but requires an engineering staff trained in MSE and RSS technology. This trained staff is also a valuable asset during construction, when questions arise or design modifications are required.

The disadvantage is that for alternate bids, additional sets of designs and plans must be processed, although only one will be constructed. A further disadvantage is that newer systems or components may not be considered during the design stage.

The fully detailed plans shall include but not be limited to, the following items:

10.4.1 Plan and Elevation Sheets

- Plan view to reflect the horizontal alignment and offset from the horizontal control line to the face of wall or slope. Beginning and end stations for the reinforced soil construction and transition areas, and all utilities, signs, lights, etc. that affect the construction should be shown.
- For MBW unit faced walls, the plan view should show alignment baseline, limits of bottom of wall alignment and limits of top of wall alignment, as alignments vary with the batter of MBW system actually supplied.
- Elevation views indicating elevations at top and bottom of walls or slopes, beginning and end stations, horizontal and vertical break points, location and elevation of copings and barriers, and whole station points. Location and elevation of final ground line shall be indicated.
- Length, size, and type of soil reinforcement and where changes in length or type occur shall be shown.
- Panel and MBW unit layout and the designation of the type or module, the elevation of the top of leveling pad and footings, the distance along the face of the wall to all steps in the footings and leveling pads.
- Internal drainage alignment, elevation, and method of passing reinforcements around such structures.
- Any general notes required for construction.
- Cross sections showing limits of construction, fill requirements, and excavation limits. Mean high water level, design high water level, and drawdown conditions shall be shown where applicable.
- Limits and extent of reinforced soil volume.

- All construction constraints, such as staged construction, vertical clearance, right-of-way limits, etc.
- Payment limits and quantities.

10.4.2 Facing/Panel Details

- Facing details for erosion control for reinforced slopes and all details for facing modules, showing all dimensions necessary to construct the element, reinforcing steel, and the location of reinforcing attachment devices embedded in the panels.
- All details of the architectural treatment or surface finishes.

10.4.3 Drainage Facilities/Special Details

- All details for construction around drainage facilities, overhead sign footings, and abutments.
- All details for connection to traffic barriers, copings, parapets, noise walls, and attached lighting.
- All details for temporary support including slope face support where warranted.
- All details for wall initiation and termination, and any transitions.

10.4.4 Design Computations

The plans shall be supported by detailed computations for internal and external stability and life expectancy for the reinforcement.

For plans prepared by material suppliers, the Owner and/or their consultant normally determine deep seated global stability. The Owner must define responsibility for compound stability analysis, when applicable.

10.4.5 Geotechnical Report

The plans shall be prepared based on a geotechnical report that details the following:

- Engineering properties of the foundation soils including shear strength and consolidation parameters used to establish settlement and stability potential for the proposed construction. Maximum bearing pressures must be established for MSE wall construction.
- Engineering properties of the reinforced soil including shear strength parameters (ϕ , c) compaction criteria, gradation, and electrochemical limits.

- Engineering properties of the fill or in-situ soil behind the reinforced soil mass, including shear strength parameters (ϕ , c) and compaction criteria.
- Groundwater or free water conditions and required drainage schemes if required.

10.4.6 Construction Specifications

Construction and material specifications for the applicable system or component as detailed later in this chapter, which include testing requirements for all materials used.

10.5 END RESULT DESIGN APPROACH

Under this approach, often referred as "line and grade" or "two line drawing," the Agency prepares drawings of the geometric requirements for the structure or reinforced slope and material specifications for the components or systems that may be used. The components or systems that are permitted are specified or are from a pre-approved list maintained by the Agency, from its prequalification process.

The end-result approach, with sound specifications and prequalification of suppliers and materials, offers several benefits. Trained and experienced staff performs design of the MSE structure. The prequalified material components (facing, reinforcement, and miscellaneous) have been successfully and routinely used together, which may not be the case for in-house design with generic specifications for components. Also, the system specification approach lessens engineering costs and manpower for an Agency and transfers some of the project's design cost to construction.

The disadvantage is that Agency engineers may not fully understand the technology at first and, therefore may not be fully qualified to review and approve construction modifications. Newer systems may not be considered due to the lack of confidence of Agency personnel to review and accept these systems. In addition, complex phasing and special details are not addressed until after the contract has been awarded.

The bid quantities are obtained from specified pay limits denoted on the "line and grade" drawings and can be bid on a lump-sum or unit-price basis. The basis for detailed designs to be submitted after contract award are contained either as complete special provisions or by reference to AASHTO or Agency manuals, as a special provision.

Plans, furnished as part of the contract documents, contain the geometric, geotechnical and design-specific information listed below.

10.5.1 Geometric Requirements

- Plan and elevation of the areas to be retained, including beginning and end stations.
- For MBW unit faced walls, the plan view should show alignment baseline, limits of bottom of wall alignment and limits of top of wall alignment, as alignments vary with the batter of MBW system actually supplied.
- Typical cross section that indicates face batter, pay limits, drainage requirements, excavation limits, etc.
- Elevation view of each structure showing original ground line, minimum foundation level, finished grade at ground surface, and top of wall or slope line.
- Location of utilities, signs, etc., and the loads imposed by each such appurtenance, if any.
- Construction constraints such as staged construction, right-of-way, construction easements, etc.
- Mean high water level, design high water level, and drawdown conditions where applicable.

10.5.2 Geotechnical Requirements

They are the same as in Section 8.4 except that the design responsibility is clearly delineated as to areas of contractor/supplier and Agency responsibility.

Typically, the Agency would assume design responsibility for developing global stability, bearing resistance and settlement analyses, as they would be the same regardless of the system used. The contractor/supplier would assume responsibility for both internal and local external stability for the designed structures.

10.5.3 Structural and Design Requirements

- Reference to specific governing sections of the Agency design manual (materials, structural, hydraulic and geotechnical), construction specifications and special provisions. If none is available for MSE walls, refer to current AASHTO, both Division I, Design and Division II, Specifications.
- Magnitude, location, and direction of external loads due to bridges, overhead signs and lights, and traffic surcharges.
- Limits and requirements of drainage features beneath, behind, above, or through the reinforced soil structure.
- Slope erosion protection requirements for reinforced slopes.
- Size and architectural treatment of concrete panels for MSE walls.

10.5.4 Performance Requirements

- Tolerable movement of the structure both horizontal and vertical.
- Tolerable face panel movement.
- Monitoring and measurement requirements.

10.6 STANDARD DESIGNS

The development and use of standard MSEW and RSS designs are discussed in Sections 4.8 and 8.8, respectively. With standard design, the Agency has certain responsibilities in preparation of the project plans and the vendor has certain responsibilities. For the example standard designs (Berg, 2000), the following Agency responsibilities are noted on the standard plans.

10.6.1 MSEW Standard Designs

Agency Responsibilities:

In addition to the standard sheets, plan and front elevation views of the modular block retaining walls shall be included in the plans. The plan view must show alignment baseline, limits of bottom of wall alignment, and limits of top of wall alignment as alignments vary with batter of wall system supplied. The front elevation must identify bottom and top of wall elevations, existing grades, and finished grades.

If the wall is curved, show the radius at the bottom and the top of each wall segment and the P.C. and P.T. station points off of baseline and limits of bottom and top of wall alignment. Reference adjacent pavement elevations (including superelevations, as applicable).

Reference standard plates and provide details for traffic barriers, curb and gutter, handrails and fencing as required by project conditions. See AASHTO and Agency design manuals, standard plates and details for requirements.

Surface drainage patterns shall be shown in the plan view. Provide dimensions for width and depth of the drainage swale as well as the type of impervious liner material. Surface water runoff should be collected above and diverted around wall face.

Detail lines and grades of the internal drainage collection pipe. Detail or note the destination of internal wall drains as well as the method of termination (daylight end of pipe or connection into hydraulic structure).

Soft soils and/or high water conditions may not be suitable for application of standard designs and may require a project specific design.

Standard design charts are not applicable to:

- Project/sites where foundation soils shear strength and/or bearing capacity do not meet or exceed values used in the development of standard design charts.
- Projects with a large (Agency defined) quantity of face area where project specific designs are recommended.
- Where slopes in front of wall are steeper than 1:3.
- Where maximum wall height exceeds 32 ft (7.0 m).
- Where walls are tiered.
- Walls with soundwalls.

Contractor Responsibilities:

Approved combinations of modular block unit and soil reinforcement products list with MBW reinforcement class noted are held and maintained by the Agency. Only approved product combinations may be used in standard designs.

Provide detailed drawings for construction containing:

- Elevation view with reinforcement placement requirements, wall facing layout, and geometric information. Top of wall may extend up to 4 inches (100 mm) above plan top of wall elevation.
- Plan view with bottom and top of wall alignment, and plan limits of wall alignment.
- Cross sections detailing batter, reinforcement, vertical spacing. Reinforcement lengths. Subsurface drainage, surface drainage, and water runoff collection above wall.
- Reinforcement layouts reinforcement shall be placed at 100% coverage ratio. Reinforcement elevations shall be consistent across length of wall structure.
- Note block, reinforcement, and fill placement methods and requirements.
- Detail all wall fill penetrations and wall face penetrations. Detail reinforcement and/or wall facing unit placement around penetrations.
- Details that are specific to vendor products and their interaction with other project components.
- List information on approved combination of MBW unit and geosynthetic reinforcement, including Agency classification code, nominal block width, properties for field identification, and installation instructions.
- Details of cap units and installation/fastening instructions for the caps. Cap units shall be set in a bed of adhesive designed to withstand moisture and temperature extremes, remain flexible, and shall be specifically formulated for bonding masonry to masonry.

- Certification by professional engineer that the construction layout meets the requirements of plans and Agency MSEW standards. Deviation from standard design tables by value engineering submittal only.

10.6.2 RSS Standard Designs

Agency Responsibilities:

Review by Turf and Erosion Prevention Unit and the Office of Environmental Services (or similar), shall be performed for all RSS applications. Turf establishment and maintenance items, hydroseeding over erosion control blanket, use of turf reinforcement mat in channelized flow areas, modification of seed mix, turf maintenance contract items, in addition to the details contained on these drawings, should be evaluated on a project basis.

In addition to the standard sheets, typical cross sections of the soil slopes shall be included in the plans as well as including soil slopes on the project cross sections.

Detail transition of RSS to adjacent slopes or structures.

Reference standard plates and provide details for traffic barriers, curb and cutter, handrails and fencing as required by project conditions. See AASHTO and Agency design manuals, standard plates, and details for requirements.

Detail lines and grades of the internal drainage collection pipe. Detail or note the destination of internal drains as well as the method of termination (daylight end of pipe or connection into adjacent hydraulic structure).

Surface drainage patterns shall be shown in the plan view. Surface water runoff should be collected above and diverted around slope face.

Define reinforced soil slope angle and define construction limits on the plan view based on this angle. Standard slope angles are 45 and 70 degrees.

Soft soils and/or high water conditions (defined as groundwater within a depth equal to the slope height H) may not be suitable for application of standard designs and requires special consideration by the Agency.

Standard designs are not applicable for projects with large quantity (Agency defined) of vertical face area where project specific designs are recommended.

Designs based on level backfill, zero toe slope and traffic surcharge. Slopes above or below the oversteepened reinforced slope are not suitable for application of standard designs and require special consideration by the Agency.

Refer to Case 1A and 1B for soil slopes between 1:2 (26.5°) and 45° maximum. Use Case 2 for soil slopes greater than 45° and up to 70° maximum.

Geotechnical investigation shall be performed for all RSS applications.

Agency Responsibilities:

Approved soil reinforcement products list, with type noted, and approved erosion control products list, are held and maintained by the Agency. Only approved products may be used in standard designs.

Provide detailed drawings for construction, containing:

- Elevation view with reinforcement placement requirements, soil slope layout and geometric information.
- Cross sections detailing slope face angle, reinforcement vertical spacing, reinforcement lengths, subsurface drainage, surface drainage, and slope face erosion protection.
- Detail all reinforced fill penetrations and face penetrations. Detail reinforcement and erosion protection placement around penetrations.
- List information on approved geosynthetic reinforcement, including Agency classification code, properties for field identification and installation directions. List product and installation information on welded wire mesh facing forms if utilized.
- Certification by Professional Engineer that construction layout meets the requirements of plans and Agency RSS standards. Deviation from standard design tables by value engineering submittal only.

10.7 REVIEW AND APPROVALS

Where Agency design is based on a supplier's plans, it should be approved for incorporation in the contract documents following a rigorous evaluation by Agency structural and geotechnical engineers. The following is a checklist of items requiring review:

- Conformance to the project line and grade.
- Conformance of the design calculations to Agency standards or codes such as current AASHTO with respect to design methods, allowable bearing capacity, allowable tensile strength, connection design, pullout parameters, surcharge loads, and factors of safety.

- Development of design details at obstructions such as drainage structures or other appurtenances, traffic barriers, cast-in-place junctions, etc.
- Facing details and architectural treatment.

For end result contracting methods, the special provisions should contain a requirement that complete design drawings and calculations be submitted within 60 days of contract award for Agency review.

The review process should be similar to the supplier design outlined above and be conducted by the Agency's structural and geotechnical engineers.

10.8 CONSTRUCTION SPECIFICATIONS AND SPECIAL PROVISIONS FOR MSE WALL AND RSS CONSTRUCTION

A successful reinforced soil project will require sound, well-prepared material and construction specifications to communicate project requirements as well as construction guidance to both the contractor and inspection personnel. Poorly prepared specifications often result in disputes between the contractor and owner representatives.

A frequently occurring problem with MSE systems is the application of different or unequal construction specifications for similar MSE systems. Users are encouraged to utilize a single unified specification that applies to all systems, regardless of the contracting method used. The construction and material requirements for MSE systems are sufficiently well developed and understood to allow for unified material specifications and common construction methods.

Guide construction and material specifications are presented in this chapter for the following types of construction:

- Section 10.9 – Example specification for MSE walls with segmental precast concrete, WWM, or MBW facings and steel (grid or strip) or geosynthetic reinforcements.
- Section 10.10 – Example specifications for RSS systems.

These guide specifications should serve as the technical basis for Agency developed standard specifications for these items. **Local experience and practice should be incorporated as applicable.** EDIT NOTE: Some key items that may be edited based upon local experience and/or practice are noted with a text box insert and discussion. The contractor should be required to submit a quality control plan detailing measurements and documentation that will

be maintained during construction to assure consistency in meeting specification requirement.

10.9 EXAMPLE SPECIFICATION FOR MECHANICALLY STABILIZED EARTH (MSE) WALLS

The following specification addresses MSE walls reinforced with galvanized steel or geosynthetics, and faced with precast segmental panels, welded wire mesh (WWM), or masonry modular block wall (MBW) units. This example specification has been modified from the Arizona DOT (2009) LRFD MSE Wall specification. It is consistent with the design checklist in Chapter 4 and with recommendations with in this manual, but in some cases may extend beyond these recommendations. Sections to be filled in are shown as: or with an example value noted, e.g. 30 days.

MECHANICALLY STABILIZED EARTH (MSE) WALLS

1 Description:

1.01 General:

The work under this section consists of designing, furnishing all materials and constructing Mechanically Stabilized Earth (MSE) retaining walls in accordance with these specifications and in compliance with the lines and grades, dimensions and details shown on the project plans and as directed by the Engineer.

The contractor shall provide the MSE wall designer with a complete set of project plans and specifications and shall ensure that the wall design is compatible with all other project features that can impact the design and construction of the wall. The following terms are used in this specification for identification of various entities for which the contractor shall be fully responsible:

<u>Term</u>	<u>Entity</u>
Wall Manufacturer	The entity contractually retained by the contractor to provide materials and construction services for an accepted MSE wall system as identified in Subsection 1.03.
Wall Designer	The entity contractually retained by the contractor to provide design of an accepted MSE wall system as identified in Subsection 1.03. The wall designer may be a representative of the wall manufacturer.

1.02 Certifications:

(A) Certification of Design Parameters:

See Subsection 2.01 herein specified.

(B) Certification of Materials:

See Subsections 3.04, 3.07 and 3.10 herein specified.

1.03 Accepted Systems:

The contractor shall select one of the appropriate DOT pre-approved earth retaining systems to be constructed for the MSE walls designated on the plans.

Pre-approved systems are listed under category “Proprietary Retaining Walls” in the Approved Products List (APL). Copies of the most current version of the APL are available on the Internet at [REDACTED].

The features of the system furnished, including design and configuration of precast elements, fasteners, connections, soil reinforcements, joint fillers, geotextile filter, and other necessary components, shall be those that have been pre-approved.

Heights and lengths of earth retaining walls may vary from, but shall not be less than, those shown on the plans. The height and length to be used for any system shall be the minimum for that system that will effectively retain the earth behind the wall for the loading conditions and the contours, profile, or slope lines shown on the plans, or on the approved working drawings, and in accordance with all relevant internal and external stability design criteria, but not more than the pre-approved height for the particular MSE wall system selected.

1.04 Manufacturer's Field Representative:

The manufacturer's field representative performing the work described in this specification shall have, in the past three years, successfully installed at least four MSE retaining walls of heights, lengths and complexity similar to those shown on the plans and meeting the tolerances specified herein. The manufacturer's field representative may make field changes subject to the approval of the Engineer. Any such changes shall be documented in writing within 24 hours of the approved changes. This written document shall be sealed by the manufacturer's design engineer, who is registered as a Civil Engineer in the State .

1.05 MSE Pre-Activity Meeting:

A pre-activity meeting will be scheduled prior to commencement of MSE wall construction activity. As a minimum, this meeting shall be attended by the Engineer, contractor (including wall construction crew chiefs), the MSE wall sub-contractor, MSE wall manufacturer's and MSE Wall designer's representatives. No wall construction activity shall be performed until the contractor's final submittals have been approved as having satisfactorily resolved all review comments and the pre-activity meeting has been held.

1.06 Wall Aesthetics:

Wall aesthetics shall be as specified in the project plans and special provisions.

2 Submittals (Working Drawings and Design):

2.01 Submittals:

The submittals required shall include working drawings, construction procedures, supporting design calculations, verification of experience, and a transmittal letter. The transmittal letter shall only list

the documents included in the submittal. No technical information shall be included in the transmittal letter.

Working drawings and calculations shall be sealed by an engineer, who is registered as a Civil Engineer in the State. The MSE wall designer/supplier shall document on the working drawings all assumptions made in the design. The following statement shall be included near the P.E. seal on the first sheet of the working drawings: "All design assumptions are validated through notes or details on these drawings."

Six complete sets of working drawings, design calculations and MSE supplier's construction procedures modified as necessary by the contractor and Wall Designer for site-specific conditions shall be submitted to the Engineer for review. The Engineer shall have 30 calendar days after receiving the six complete sets to finish a review. The revised package shall be resubmitted to the Engineer for review. The Engineer shall have 15 calendar days to complete this review. This review process shall be repeated until the entire submittal is accepted by the Engineer.

The Department assumes no responsibility for errors or omissions in the working drawings. Acceptance of the final working drawings submitted by the contractor shall not relieve the contractor of any responsibility under the contract for the successful completion of the work.

Construction of the wall shall not commence until the contractor receives a written Notification To Proceed (NTP) from the Engineer. The NTP will be issued once the complete wall package (drawings, calculations and construction procedures) is approved. Fabrication of any of the wall components before the NTP shall be at the sole risk of the contractor.

2.02 Working Drawings:

The contractor shall submit complete working drawings and specifications for each installation of the system in accordance with the requirements of Subsection as modified herein.

Working drawings shall include the following at a minimum:

- (1) Layout of the wall including plan and elevation views;
- (2) All design parameters and assumptions including design life;
- (3) Existing ground elevations and utilities impacted by the wall, and those that should be field verified by the contractor, for each location;
- (4) Complete details of all elements and component parts required for the proper construction of the system at each location and any required accommodations for drainage systems, foundation subgrades or other facilities shown on the contract documents;
- (5) The working drawing submittal shall clearly detail any special design requirements. These special design requirements may include, but are not limited to; structural frames to place reinforcements around obstructions such as deep foundations and storm drain crossings, drainage systems, placement sequence of drainage and unit core fill with respect to reinforced (structure) fill behind a wall face using modular block facing units, guardrail post installation, scour protection, foundation subgrade modification, all corner details (acute, obtuse and 90 degrees), slip joints, joint details of MSE walls with other cast-in-place structures, wedges,

shims and other devices such as clamps and bracing to establish and maintain vertical and horizontal wall facing alignments;

(6) A complete listing of components and materials specifications; and

(7) Other site-specific or project specific information required by the contract.

2.03 MSE Wall Design:

(A) General:

The working drawings shall be supplemented with all design calculations for the particular installation as required herein. Installations that deviate from the pre-approved design shall be accompanied by supporting stability (internal; external; and global/overall and/or compound if required in the project documents) calculations of the proposed structure as well as supporting calculations for all special details not contained in the pre-approved design. The MSE wall designer/supplier shall note all deviations of the proposed wall design from the pre-approved design.

The proposed design shall satisfy the design parameters shown on the project plans and listed in these specifications, and comply with the design requirements of the following document:

- FHWA NHI-10-024 Vol I and NHI-10-025 Vol II, “Design of MSE Walls and Reinforced Slopes,” (Berg et al., 2009).
- AASHTO (2007), “AASHTO LRFD Bridge Design Specifications,” 4th Edition, including 2008 and 2009 Interims.

All references to AASHTO (2007) shall mean to include the latest interims.

Maximum reinforcement loads shall be calculated using the “Simplified Method” as presented in AASHTO (2007) and as per the requirements specified herein. No other design method will be allowed. EDIT NOTE: The 2008 Interims states that the *Simplified Method* or the *Coherent Gravity Method* may be used. Agencies should specify what method(s) are acceptable and not leave as a contractor option.

Sample analyses and hand-calculations shall be submitted to verify the output from software used by the MSE wall designer. Sample analyses and hand-calculations shall be required for complex walls having geometries and loading conditions that are not readily amenable to computer analysis. Failure modes, including circular, non-circular, and multi-part wedge, shall be analyzed for deep-seated global stability and compound stability to verify the most critical failure case. EDIT NOTE: Agency must specify who – the Agency or the contractor/wall vendor – is responsible for global and for compound stability analyses. If the contractor/wall supplier is responsible, subsurface data in sufficient detail to perform the analyses must be provided by the Agency to the contractor/wall supplier. See Chapter 4 for additional discussion.

Unless otherwise specified in the contract, all structures shall be designed to conform to the requirements shown in Table 1 and other requirements specified herein.

TABLE 1			
DESIGN PARAMETERS			
Description	Limit State	Value	Note*
1. Design Life	All limit states	<u>75 Years</u>	
2. Effective (Drained) Friction Angle			
a. Retained Backfill	All limit states	<u>32° min</u>	
b. Reinforced Backfill	All limit states	<u>34° to max 40°</u>	1
3. Length of soil reinforcement, B	All limit states	0.7H min or 8-ft whichever is more	2
4. Limiting eccentricity	Strength (all)	B/4 (soil), 3/8B (rock)	
	Service I	B/6 (soil), B/4 (rock)	
5. Coefficient of Sliding Friction	Strength (all)	$\tan[\min(\phi_r, \phi_f, \phi_i)]$	3
6. Resistance factors			
a. Sliding	Strength (all)	1.0	4
b. Bearing	Strength (all)	0.65	5
c. Overall (slope) stability			
I. Deep Seated Stability	Service I	<u>0.65</u>	6
II. Compound Stability	Service I	<u>0.65</u>	6
d. Pullout resistance			
I. Static	Strength (all)	0.90	7
II. Combined static/earthquake	Strength (all)	1.20	7
e. Tensile resistance of metallic reinforcements and connectors			
I. Static			
- Strip reinforcement	Strength (all)	0.75	8
- Grid reinforcement	Strength (all)	0.65	8,9
II. Combined static/earthquake			
- Strip reinforcement	Strength (all)	1.00	8
- Grid reinforcement	Strength (all)	0.85	8,9
f. Tensile resistance of geosynthetic reinforcements and connectors			
I. Static	Strength (all)	0.90	
II. Combined static/earthquake	Strength (all)	1.20	
* Refer to Table 1.1 for notes.			

TABLE 1.1	
NOTES FOR TABLE 1	
#	Note
1	A minimum friction angle of 34 degrees shall be substantiated by laboratory tests discussed in Subsection 3.05(D). If the measured friction angle in laboratory tests as per Subsection 3.05(D) is greater than 40-degrees then the friction angle in the analysis shall be limited to 40-degrees.
2	H is the design height of the wall and is defined as the difference in elevation between from the finished grade at the tope of wall and the top of leveling pad. The top of the leveling pad shall always be below the minimum embedment reference line as indicated on the plans for that location. The length of the soil reinforcement, B, is measured from the backface of the wall facing unit. In case of grid type reinforcements the length of the soil reinforcement is measured from the backface of the wall to the last full transverse member. For modular block facing units, the total length of the reinforcement, B _T , as measured from the front face of the wall is the length B as defined above plus the width of the modular block unit (the horizontal dimension of the block unit measured perpendicular to the wall face).
3	ϕ_r = friction angle of reinforced wall fill; ϕ_f = friction angle of foundation soil; ϕ_i = friction angle of the interface between reinforcement and soil for cases of sheet reinforcement such as geotextiles. All friction angles are effective (drained) friction angles. Refer to Geotechnical Report for friction angle of foundation soil.
4	Passive resistance shall not be considered in evaluation of sliding resistance.
5	For all limit states, the design loading for the MSE retaining wall system shall not exceed the factored general and local bearing resistances specified in the Geotechnical Report(s).
6	For earthquake loading condition, a resistance factor of 0.90 shall be used.
7	Live load due to vehicular traffic shall be included in the computations to determine the maximum tensile forces in reinforcement layers, but shall be neglected in the computations for pullout resistance. EDIT NOTE: Agency should specify whether or not to include live load in tensile force calculations for pullout check, see Chapter 4 for discussion. Intensity of live load shall be considered as a uniform surcharge using the equivalent height of soil in accordance with Section Article 3.11.6.4 of AASHTO (2007).
8	Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 of AASHTO (2007) and apply to net section less sacrificial area.
9	Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.
10	Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating an extreme event limit state.

(B) Subsurface Drainage Systems:

Walls shall be provided with subsurface drainage measures as shown on the project plans and specifications. As a minimum, an underdrain system shall be provided for leading subsurface and surface water away from the backfill and outside the limits of the wall. Geocomposite drains, if used for subsurface drainage, shall be in accordance with Subsection [] and [] of the specifications.

(C) Obstructions in Backfill:

(1) General:

Where obstructions, such as deep foundations or storm drains crossings, are located in the reinforced backfill zone, cutting of reinforcements to avoid obstructions shall not be permitted. A minimum offset of one diameter but not less than three (3) feet shall be maintained between the face of any pipe crossings and the back face of retaining wall panels. A minimum clearance of three (3) feet shall be maintained between the face of any other obstruction and the back face of retaining wall panels.

(2) Horizontal Deflection of Reinforcements:

In the horizontal plane at a reinforcing level, a deviation up to fifteen (15) degrees from the normal to the face of the wall may be allowed for strip reinforcement and bolted connection. This deviation is herein referred to as the splay angle. Grid reinforcements may not be splayed, unless connection has been specifically fabricated to accommodate a splay and connection detail has been approved by the Agency. If used, the splay in grid reinforcement is limited to fifteen (15) degrees. For obstructions that cannot be accommodated with splayed reinforcement, structural frames and connections shall be required, and shall be designed in accordance with Section 10 ("Steel Structures") of AASHTO (2007) for the maximum tension in the reinforcements. The structural frame design shall be such that bending moments are not generated in the soil reinforcement or the connection at the wall face. The design, along with supporting calculations, shall be included in the working drawings.

(3) Vertical Deflection of Reinforcements:

Vertical deflection of the reinforcement to avoid obstructions such as utilities along the wall face shall be limited to a maximum of 15 degrees from normal to face of wall. Bends in the reinforcement shall be smooth and gradual to ensure that galvanization remains intact.

(D) Hydrostatic Pressures:

As determined by the Engineer and/or as noted on the plans, for walls potentially subject to inundation, such as those located adjacent to rivers, canals, detention basins or retention basins, a minimum hydrostatic pressure equal to three (3) feet shall be applied at the high-water level for the design flood event. Effective unit weights shall be used in the calculations for internal and external stability beginning at levels just below the equivalent surface of the pressure head line. Where the wall is influenced by water fluctuations, the wall shall be designed for rapid drawdown conditions which could result in differential hydrostatic pressure greater than three (3) feet. As an alternative to designing for rapid drawdown conditions, Size 57 coarse aggregate, as specified in AASHTO M 43, shall be provided as reinforced wall fill for the full length of the wall and to the maximum height of submergence of the wall. Separation geotextile, as specified in Subsection , shall be provided at the interface of the Size 57 coarse aggregate and reinforced wall fill above it, and at the interface of the retained backfill behind it. Adjoining sections of separation geotextile shall be overlapped by a minimum of 12 inches.

(E) Acute Angle Corners:

Wall corners with an included angle of less than 70 degrees shall be designed for bin-type lateral pressures for the extent of the wall where the full length of the reinforcement cannot be installed without encountering a wall face. Acute angle corner structures shall not be stand-alone separate structures. Computations shall be provided that demonstrate deformation compatibility between the acute angle corner structure and the rest of the MSE wall. Full-height vertical slip joints shall be provided at the acute angle corner and after the last column of panels where full length of the reinforcements can be placed. The soil reinforcement attached to the slip joints shall be oriented perpendicular to the slip joint panels and shall be the full design length. Special connection and compaction details shall be provided on the working drawings.

(F) Spacing of Metallic Reinforcement for Flexible Face Wall Systems:

For permanent walls, vertical and horizontal spacing of metallic reinforcements for flexible face (welded wire or similar) wall systems shall not exceed 18 inches. The stiffness of the facing and spacing of reinforcements shall be such that the maximum local deformation between soil reinforcement layers shall be limited to less than 1½ inches. EDIT NOTE: Recommended limitation range, see Chapter 3, is 1 to 2 inches. Agency should specify specific value. Facing elements shall not yield in bending and tension.

For temporary walls, i.e., walls with up to 36 months service life, the contractor may adjust the stiffness of the facing and spacing of the reinforcements such that the local deformation between the reinforcement is within the elastic range in bending and tension, and the overall geometry meets the line and grade requirements for the temporary walls.

(G) Soil Reinforcement for Modular Block Wall Systems:

The soil reinforcement lengths and percent coverage at a given reinforcement level shall be in accordance with the plans. All soil reinforcement shall be positively connected to the modular block facing units that is capable of resisting 100% of the maximum tension in the soil reinforcements at any level within the wall. Detailed documentation for connection strength shall be submitted as noted in Subsection 3.10. The vertical spacing of the soil reinforcement for walls with modular block facing units shall be as follows:

1. The first (bottom) layer of soil reinforcement shall be no further than 16 inches above the top of the leveling pad.
2. The last (top) layer of soil reinforcement shall be no further than 20 inches on the average below the top of the uppermost MBW unit.
3. The maximum vertical spacing between layers of adjacent soil reinforcement shall not exceed 32 inches. For walls deriving any part of their connection capacity by friction the maximum vertical spacing of the reinforcement should be limited to two times the block depth (front face to back face) to assure construction and long-term stability. The top row of reinforcement should be one-half the vertical spacing.

(H) Initial Batter of Wall:

The initial batter of the wall, both during construction and upon completion, shall be within the vertical and horizontal alignment tolerances included in this specification. The initial batter of the wall at the start of construction and the means and methods necessary to achieve the batter shall be provided on the working drawings. Subject to Engineer's approval, the initial batter may be modified

at the start of construction by the manufacturer's field representative based on the evaluation of the backfill material selected by the contractor. Any such changes shall be documented in writing within 24 hours of the approved changes. This written document shall be sealed by the manufacturer's design engineer who is registered as a Civil Engineer in the state. Details of the wedges or shims or other devices, such as clamps and external bracing used to achieve or maintain the wall batter, shall be as shown on the working drawings and/or accompanying construction manual. Permanent shims shall comply with the design life criteria, and shall maintain the design stress levels required for the walls.

3 Material Requirements:

3.01 Precast Concrete Elements:

Precast concrete elements shall conform to the requirements for precast minor structures in Sections and . The concrete shall be Class with minimum design strength of 4,000 pounds per square inch. The mix design shall conform to the requirements of Subsection 3.02.

Prior to casting, all embedded components shall be set in place to the dimensions and tolerances designated in the plans and specifications. Rustication for wall aesthetics shall be in accordance with project plans, special provisions, and applicable requirements of Sections , , and .

(A) Concrete Testing and Inspection:

Precast concrete elements shall be subjected to compressive strength testing in accordance with Subsection , and inspected for dimensional tolerances and surface conditions in accordance with Subsections and respectively. Panels delivered to the site without the Agency acceptance stamp will be rejected.

(B) Casting:

Precast concrete face panels shall be cast on a horizontal surface with the front face of the panel at the bottom of the form. Connection hardware shall be set in the rear face. The concrete in each precast concrete panel shall be placed without interruption and shall be consolidated by deploying an approved vibrator, supplemented by such hand tamping as may be necessary to force the concrete into the corner of the forms, and to eliminate the formation of stone pockets or cleavage planes. Form release agents as specified in Subsection shall be used on all form faces for all casting operations.

The contractor shall advise the Engineer of the starting date for concrete panel casting at least 14 calendar days prior to beginning the operation if the casting operation is within the State, or 21 calendar days if the casting operation is outside the State.

(C) Finish:

(1) Non-Exposed Surfaces:

Rear faces of precast concrete panels shall receive a Class 1 finish in accordance with Subsection .

(2) Exposed Surfaces:

The type of finish required on exposed surfaces shall be as shown in the plans.

(a) Exposed Aggregate Finish:

- (1) Prior to placing concrete, a set retardant shall be applied to the casting forms in accordance with the manufacturer's instructions.
- (2) After removal from the forms and after the concrete has set sufficiently to prevent its dislodging, the aggregate shall be exposed by a combination of brushing and washing with clear water. The depth of exposure shall be between $\frac{3}{8}$ inch and $\frac{1}{2}$ inch.
- (3) An acrylic resin sealer consisting of 80 percent thinner and 20 percent acrylic solids by weight shall be applied to the exposed aggregate surface at a rate of one (1) gallon per 250 square feet.

(b) Concrete Panel Finish:

Concrete panel finish shall be in accordance with Subsection .

(D) Tolerances:

Precast concrete elements shall comply with Subsection and . Connection device placement shall be within ± 1 inch of the dimensions shown on the drawings. Panel squareness as determined by the difference between the two diagonals shall not exceed $\frac{1}{2}$ inch.

(E) Identification and Markings:

The date of manufacture, the production lot number, and the piece mark shall be inscribed on a non-exposed surface of each element.

(F) Handling, Storage and Shipping:

All panels shall be handled, stored, and shipped in such a manner to eliminate the dangers of chipping, discoloration, cracks, fractures, and excessive bending stresses. Panels in storage shall be supported in firm blocking to protect panel connection devices and the exposed exterior finish. Storing and shipping shall be in accordance with the manufacturer's recommendations.

(G) Compressive Strength:

Precast concrete elements shall not be shipped or placed in the wall until a compressive strength of 3,400 pounds per square inch has been attained. The facing elements shall be cast on a flat and level area and shall be fully supported until a compressive strength of 1,000 pounds per square inch has been attained.

(H) Precast Concrete Panel Joints:

(1) General:

Where the wall wraps around an inside corner, a corner block panel shall be provided with flange extensions that will allow for differential movement without exposing the panel joints. The back face of vertical and horizontal joints shall be covered with geotextile filter. Joint filler, bearing pads, and geotextile filter shall be as recommended by the wall manufacturer and shall meet the requirements shown on the approved working drawings.

If required, as indicated on the plans, flexible open-cell polyurethane foam strips shall be used for filler for vertical joints between panels, and in horizontal joints where pads are used.

All joints between panels on the back side of the wall shall be covered with a geotextile meeting the requirements for filtration applications as specified by AASHTO M 288. The minimum width shall be one (1) foot.

(2) Bearing Pads:

All horizontal and diagonal joints between panels shall include bearing pads. Bearing pads shall meet or exceed the following material requirements:

- Preformed EPDM (Ethylene Propylene Diene Monomer) rubber pads conforming to ASTM D 2000 Grade 2, Type A, Class A with a Durometer Hardness of 70.
- Preformed HDPE (High Density Polyethylene) pads with a minimum density of 0.946 grams per cubic centimeter in accordance with ASTM D 1505.

The stiffness (axial and lateral), size, and number of bearing pads shall be determined such that the final joint opening shall be $\frac{3}{4}+1/8$ inch unless otherwise shown on the plans. The MSE wall designer shall submit substantiating calculations verifying the stiffness (axial and lateral), size, and number of bearing pads assuming, as a minimum, a vertical loading at a given joint equal to 2 times the weight of facing panels directly above that level. As part of the substantiating calculations, the MSE wall designer shall submit results of certified laboratory tests in the form of vertical load-vertical strain and vertical load-lateral strain curves for the specific bearing pads proposed by the MSE wall designer. The vertical load-vertical strain curve should extend beyond the first yield point of the proposed bearing pad.

3.02 Steel Components:

Steel components shall conform to the applicable requirements of Sections and .

(A) Galvanization:

Soil reinforcement steel shall be hot-dip galvanized in accordance with AASHTO M 111 (ASTM A123). Connection hardware steel can be galvanized by hot-dipping or other means, provided the method satisfies the requirements of AASHTO M 111 (ASTM A123). A minimum galvanization coating of 2.0 oz/ft² (605 g/m²) or 3.4 mils (85 µm) thickness is required. Soil reinforcement steel shall be adequately supported while lifting and placing such that the galvanization remains intact.

Steel members with damaged (peeled) galvanization shall be repaired according to ASTM A780 and as specified in approved working drawings, at no additional cost to the Agency.

(B) Metallic Reinforcing Strips and Tie Strips:

Reinforcing strips shall be hot-rolled from bars to the required shape and dimensions. The strips' physical and mechanical properties shall conform to the requirements of ASTM A572, Grade 65 minimum.

Tie strips shall be shop fabricated of hot-rolled steel conforming to the requirements of ASTM A1101, Grade 50 minimum. The minimum bending radius of the tie strips shall be $\frac{3}{8}$ inch. Galvanization shall be applied after the strips are fabricated, inclusive of punch holes for bolts as shown on approved drawings.

(C) Metallic Reinforcing Mesh:

Reinforcing mesh shall be shop fabricated of cold-drawn steel wire conforming to the requirements of AASHTO M 32, and shall be welded into the finished mesh fabric in accordance with AASHTO M 55. Galvanization shall be applied after the mesh is fabricated. A minimum galvanization coating of 2.0 oz/ft² (605 g/m²) or 3.4 mils (85 µm) thickness is required.

(D) Connector Pins:

Connector pins and mat bars shall be fabricated and connected to the soil reinforcement mats as shown in the approved working drawings. Connector bars shall be fabricated of cold drawn steel wire conforming to the requirements of AASHTO M 32.

(E) Welded Wire Fabric:

All welded wire fabric shall conform to the requirements of AASHTO M 32, AASHTO M 55, and the approved working drawings. Welded wire fabric shall be galvanized in conformance with the requirements of ASTM A123.

(F) Fasteners:

Connection hardware shall conform to the requirements shown in the approved working drawings. Connection hardware shall be cast in the precast concrete panels such that all connectors are in alignment and able to transfer full and even load to the soil reinforcement. Once the reinforcement is connected to the panel, the amount of slack shall not exceed $\frac{1}{8}$ inch between the connector and the reinforcement during field installation. Fasteners shall be galvanized and conform to the requirements of AASHTO M 164 or equivalent.

3.03 Geosynthetic Reinforcement:

Geosynthetic soil reinforcement shall be limited to geogrids listed on the Agency's Approved Products List (APL). The geogrid shall be a regular network of integrally connected polymer tensile elements, with aperture geometry sufficient to permit significant mechanical interlock with the surrounding soil. Geogrid structure shall be dimensionally stable and able to retain its geometry under manufacture, transport and installation.

The nominal long-term tensile design strength (T_{al}) of specific geosynthetic material shall meet or exceed the Agency's APL.

3.04 Certificate of Analysis for Soil Reinforcements:

The contractor shall furnish the Engineer with a Certificate of Analysis conforming to the requirements of Subsection for all materials.

For geosynthetics, the Certificate of Analysis shall verify that the supplied geosynthetic is the type approved by the Engineer and as measured in full accordance with all test methods and standards specified herein. The manufacturer's certificate shall state that the furnished geosynthetic meets the requirements of the specifications, as evaluated by the manufacturer's quality control program. In case of dispute over validity of values, the Engineer can require the contractor to supply test data from an Agency-approved laboratory to support the certified values submitted, at no additional cost to the Department.

For metallic wall reinforcement, a mill test report containing the ultimate tensile strength for the soil reinforcement shall be included in the certification. For metallic wall reinforcement, a mill test report containing the galvanization coverage shall be included in the certification. For metallic mesh wall reinforcement, a mill test report containing the ultimate weld strength for the soil reinforcement shall be included in the certification.

3.05 Reinforced Wall Fill Material:

(A) General:

Reinforced wall fill material shall be free of shale, organic matter, mica, gypsum, smectite, montmorillonite, or other soft poor durability particles. No salvaged material, such as asphaltic concrete millings or Portland Cement Concrete rubble, etc., will be allowed.

(B) Soundness:

The reinforced backfill material shall have a soundness loss of 30 percent or less when tested in accordance with AASHTO T 104 using a magnesium sulfate solution with a test duration of four cycles. Alternatively, the material shall have a soundness loss of 15 percent or less when tested in accordance with AASHTO T 104 using a sodium sulfate solution with a test duration of five cycles.

(C) Gradation and Plasticity Index:

Gradations will be determined per AASTHO T 27 and shall be in accordance with Table 2, unless otherwise specified. The reinforced backfill shall be well-graded in accordance with the Unified Soil Classification System (USCS) in ASTM D2487. Furthermore, the reinforced wall fill shall not be gap-graded.

Plasticity Index (PI), as determined in accordance with AASHTO T 90, shall not exceed six.

Table 2 BACKFILL GRADATION REQUIREMENTS	
Sieve Size	Percent Passing
4 inch (Note 1)	100
No. 40	0-60
No. 200	0-15
Note 1: Maximum particle size shall be limited to $\frac{3}{4}$ inch for geosynthetics and epoxy- or PVC-coated reinforcements unless the contractor provides tests, acceptable to the Engineer, that have evaluated the extent of construction damage anticipated for the specific fill material and reinforcement combination. Construction damage testing shall be performed in accordance with the requirements of Chapter 5 of Publication No. FHWA NHI-09-087, dated 2009 ("Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes.")	

(D) Internal Friction Angle Requirement:

The reinforced wall fill material shall exhibit an effective (drained) angle of internal friction of not less than 34 degrees, as determined in accordance with AASHTO T 236.

The test shall be run on the portion finer than the No. 10 sieve. The sample shall be compacted at optimum moisture content to 95 percent of the maximum dry density, as determined in accordance with the requirements of AASHTO T 99. The sample shall be tested at the compacted condition without addition of water. No direct shear testing will be required when 80 percent or more of the material is larger than $\frac{3}{4}$ inch.

(E) Electrochemical Requirements:

The reinforced backfill material shall meet the electrochemical requirements of Table 3 when metallic soil reinforcement is used and Table 4 when geosynthetic soil reinforcement is used. For all soil reinforcements, the organic content of backfill shall be less than one (1) percent, determined in accordance with AASHTO T-267.

Table 3 ELECTROCHEMICAL REQUIREMENTS FOR METALLIC REINFORCEMENTS		
Characteristic	Requirement	Test Method
pH	5.0 to 10.0	AASHTO T-289
Resistivity, min.	3,000 ohm-cm	AASHTO T-288
Chlorides, max.	100 ppm	ASTM D4327
Sulfates, max.	200 ppm	ASTM D4327
* If the resistivity is greater or equal to 5,000 ohm-cm, the chloride and sulfate requirements may be waived.		

Table 4 ELECTROCHEMICAL REQUIREMENTS FOR GEOSYNTHETIC REINFORCEMENTS			
Base Polymer	Property	Requirement	Test Method
Polyolefin (PP and HDPE)*	pH	> 3	AASHTO T-289
Polyester	pH	> 3 and < 9	AASHTO T-289
* PP: Polypropylene and HDPE: High Density Polyethylene.			

(F) Rock Reinforced Wall Fill:

Material that is composed primarily of rock fragments (material having less than 25 percent passing a ¾-inch sieve) shall be considered to be a rock fill. The maximum particle size shall not exceed the limits listed in Table 2. Such material shall meet all the other requirements of Subsection 3.05(B) and Subsection 3.05(E). When such material is used, a very high survivability separation geotextile, meeting the minimum requirements for filtration applications specified in AASHTO M 288 and Subsection [REDACTED], shall encapsulate the rock backfill to within three (3) feet below the wall coping. Adjoining sections of separation fabric shall be overlapped by a minimum of 12 inches. Additionally, the upper three (3) feet of backfill shall contain no stones greater than three (3) inches in their greatest dimension, and shall be composed of material not considered to be rock backfill, as defined herein.

(G) Limits of Reinforced Wall Fill:

For all walls, except back-to-back walls, the reinforced backfill shall extend to at least one (1) foot beyond the free end of the reinforcement. EDIT to Agency practice/requirements. For back-to-back walls wherein the free ends of the reinforcement of the two walls are spaced apart less than or equal to one-half the design height of the taller wall, reinforced wall fill shall be used for the space between the free ends of the reinforcements as well. The design height of the wall is defined as the difference in elevation between finished grade at top of wall and the top of leveling pad. The top of the leveling pad shall always be below the minimum embedment reference line as indicated on the plans for the location under consideration.

3.06 Retained Backfill Material:

(A) General:

Backfill behind the limits of the reinforced backfill shall be considered as retained backfill for a distance equal to 50 percent of the design height of the MSE wall or as shown on the plans, except for back-to-back MSE walls as described in Subsection 3.05(G) above. The retained backfill shall be free of shale, mica, gypsum, smectite, montmorillonite or other soft particles of poor durability. The retained backfill shall meet the soundness criteria as described in Subsection 3.05(B).

The percent fines (the fraction passing No. 200 sieve) shall be less than 50 as determined in accordance with [REDACTED] Test Method, and the Liquid Limit (LL) and the Plasticity Index (PI) shall be less than 40 and 20, respectively, as determined in accordance with AASHTO T-90.

Material that is composed primarily of rock fragments (material having less than 25 percent passing a ¾-inch sieve), shall be considered to be a rock backfill and the requirements of Subsection 3.05(F) shall apply.

(B) Internal Friction Angle Requirement:

Unless otherwise noted on the plans, the retained backfill material shall exhibit an effective (drained) angle of internal friction of not less than [REDACTED] degrees as determined by AASHTO T 236. EDIT insert Agency value consistent with material specification.

The test shall be run on the portion finer than the No. 10 sieve. The sample shall be compacted at optimum moisture content and to 95 percent of maximum dry density, as determined in accordance with AASHTO T 99 (Proctor) test OR AASHTO T 180 (Modified Proctor) test. EDIT NOTE:

Specify test method consistent with compaction specification. The sample shall be tested at the compacted condition without addition of water.

No direct shear testing will be required when 80 percent or more of the material is larger than $\frac{3}{4}$ inch.

3.07 Certificate of Analysis for Reinforced Wall Fill and Retained Backfill Materials

At least three weeks prior to construction of the MSE wall, the contractor shall furnish the Engineer with an 80-pound representative sample of each of the backfill material and a Certificate of Analysis conforming to the requirements of Subsection 106.05 certifying that the backfill materials comply with the requirements specified herein. During construction the reinforced and retained backfill shall be sampled and tested by the Contractor for acceptance and quality control testing in accordance with the requirements stated in Table 929-5 and Table 929-6, respectively. A new sample and Certificate of Analysis shall be provided any time the reinforced and retained backfill material changes as noted in Table 929-5 and 929-6, respectively.

Table 5 Sampling Frequency for Reinforced Backfill Material	
Test	Frequency
Gradation (AASHTO T 26), Plasticity Index (AASHTO T 90)	One per <u>2,000</u> CY At job site
Resistivity, pH, Organic Content, Chlorides, Sulfates (Table 929-3)	One per <u>2,000</u> CY At job site
Internal friction angle (AASHTO T 236) Proctor density and Optimum Moisture by AASHTO T 99 OR AASHTO T 180 EDIT NOTE: Specify one, consistent with compaction specification. Test pad section (Subsection 4.06(B))	One per material change and change in source*
* The gradation and plasticity tests performed at the frequency noted in Table 5 shall be used to determine the Unified Soil Classification System (USCS) designation as per ASTM D 2487. New tests shall be required with each change in USCS designation including change in dual symbol designations (example: SW-SM, SW-SC, etc.). All requirements of Subsection 3.05 shall be satisfied. New tests shall also be required for each new source regardless of whether the USCS designation changes or not.	

Table 6 Sampling Frequency for Retained Backfill Material	
Test	Frequency
Gradation (AASHTO T 27), Plasticity Index (AASHTO T 90)	One per <u>5,000</u> CY At job site
Internal friction angle (AASHTO T 236) Proctor density and Optimum Moisture by AASHTO T 99 OR AASHTO T 180 EDIT NOTE: Specify one, consistent with compaction specification.	One per material change and change in source*
* The gradation and plasticity tests performed at the frequency noted in Table 6 shall be used to determine the Unified Soil Classification System (USCS) designation as per ASTM D2487. New tests shall be required with each change in USCS designation including change in dual symbol designations (example: SW-SM, SW-SC, etc.). All requirements of Subsection 3.06 shall be satisfied. New tests shall also be required for each new source regardless of whether the USCS designation changes or not.	

3.08 Cast-in-Place Concrete:

Cast-in-place concrete shall conform to the requirements of Sections and . Unless otherwise approved, all cast-in-place concrete shall be Class with a minimum compressive strength of 4,000 pounds per square inch.

3.09 Modular Block (Segmental) Facing Units:

This section covers dry-cast hollow and solid concrete masonry structural retaining wall units, machine made from Portland cement, water, and suitable mineral aggregates. The units are intended for use as facing units in the construction of mortarless, modular block walls (MBW) also known as segmental retaining walls (SRW). Metallic or geosynthetic reinforcement specified in Section 3.02 and 3.03, respectively, may be used as soil reinforcement in the reinforced (structure) wall fill zone.

(A) Casting:

Cementitious material in the modular block facing unit shall be Portland cement conforming to the requirements of ASTM C 150. If fly ash is used it shall not exceed 20% by weight of the total cement content and shall conform to ASTM C 618. Aggregates used in concrete blocks shall conform to ASTM C 33 for normal weight concrete aggregate. Efflorescence control agent shall be used in concrete mix design to prevent efflorescence on the block.

The contractor shall advise the Engineer of the starting date for concrete panel casting at least 14 calendar days prior to beginning the operation if the casting operation is within the State, or 21 calendar days if the casting operation is outside the State.

(B) Physical Requirements:

At the time of delivery to the work site, the modular block facing units shall conform to the following physical requirements:

- 1) Minimum required compressive strength of 4,000 psi (average 3 coupons)
- 2) Minimum required compressive strength of 3,500 psi (individual coupon)
- 3) Minimum oven dry unit weight of 125 pcf
- 4) Maximum water absorption of 5 % after 24 hours
- 5) Maximum number of blocks per lot of 2,000. Tests on blocks shall be submitted at the frequency of one set per lot.

Acceptance of the concrete block, with respect to compressive strength, water absorption and unit weight, will be determined on a lot basis. The lot shall be randomly sampled and tested in accordance with ASTM C140. As no additional expense to the Department, the manufacturer shall perform the tests at an Agency approved laboratory and submit the results to the Engineer for approval. Compressive strength test specimens shall be cored or shall conform to the saw-cut coupon provisions of ASTM C 140. Block lots represented by test coupons that do not reach an average compressive strength of 4,000 psi will be rejected.

(C) Freeze-Thaw Durability:

In areas where repeated freezing and thawing under saturated conditions occur, the units shall be tested to demonstrate freeze-thaw durability in accordance with Test Method ASTM C1262. Freeze-thaw durability shall be based on tests from five specimens made with the same materials, concrete

mix design, manufacturing process, and curing method, conducted not more than 18 months prior to delivery. Specimens used for absorption testing shall not subsequently be used for freeze-thaw testing. Specimens shall comply with either or both of the following acceptance criteria depending on the severity of the project location as determined by the Department:

- 1) The weight loss of four out of five specimens at the conclusion of 150 cycles shall not exceed 1% of its initial weight when tested in water.
- 2) The weight loss of each of four out of the five test specimens at the conclusion of 50 cycles shall not exceed 1.5% of its initial mass when tested in a saline (3% sodium chloride by weight) solution.

(D) Tolerances for Modular Block Dimensions:

Modular blocks shall be manufactured within the following tolerances:

- 1) The length and width of each individual block shall be within $\pm \frac{1}{8}$ inch of the specified dimension. Hollow units shall have a minimum wall thickness of $1\frac{1}{4}$ inches.
- 2) The height of each individual block shall be within $\pm 1/16$ inch of the specified dimension.
- 3) When a broken (split) face finish is required, the dimension of the front face shall be within ± 1.0 inch of the theoretical dimension of the unit.

(E) Finish and Appearance:

Units that indicate imperfect molding, honeycomb or open texture concrete and color variation on front face of block due to excess form oil or other reasons shall be rejected. All units shall be visually efflorescence free. All units shall be sound and free of cracks or other defects that would interfere with the proper placing of the unit or significantly impair the strength or permanence of the construction. Minor cracks (e.g. no greater than $1/50$ inch in width and no longer than 25% of the unit height) incidental to the usual method of manufacture or minor chipping resulting from shipment and delivery, are not grounds for rejection.

The exposed faces shall be free of chips, cracks or other imperfections when viewed from a distance of 30 feet under diffused lighting. Up to five (5) percent of a shipment may contain slight cracks or small chips not larger than 1.0 inch.

Color and finish shall be as shown on the plans and shall be erected with a running bond configuration.

(F) Pins:

If pins are required to align modular block facing units, they shall consist of a non-degrading polymer or hot-dipped galvanized steel and be made for the express use with the modular block units supplied. Connecting pins shall be capable of holding the geogrid in the proper design position during backfilling.

(G) Cap Units and Adhesive:

The cap unit connection to the block unit immediately under it shall be of a positive interlocking type and not frictional. Cap units shall be cast to or attached to the top of modular block facing units in strict accordance with the requirements of the manufacturer of the blocks and the adhesive. The surface of the block units under the cap units shall be clear of all debris and standing water before the

approved adhesive is placed. Contractor shall provide a written 10-year warranty, acceptable to Owner, that the integrity of the materials used to attach the cap blocks will preclude separation and displacement of the cap blocks for the warranty period.

(H) Unit (Core) Fill:

Unit (core) fill is defined as free-draining, coarse grained material that is placed within the empty cores of the modular block facing units. Unit (core) fill shall be a well graded crushed stone or granular fill meeting the gradation shown in Table 7. Gradation for unit fill shall be tested at the frequency of 1 test per 50 yd³ at the job site and for every change in the material source.

Table 7
Gradation for Unit (Core) Fill

U.S. Sieve Size	Percent Passing
1½-inch	<u>100</u>
1-inch	75-100
¾-inch	50-75
No. 4	0-60
No. 40	0-50
No. 200	0-5

(I) Gravel Fill:

A minimum width of 1-ft of gravel fill should be provided behind solid (non-hollow) modular block units. A minimum volume of 1-ft³/ft² of drainage fill shall be provided. Gravel fill shall meet the requirements of the unit (core) fill. A suitable geotextile fabric between the gravel fill and reinforced wall fill shall be used to meet the filtration requirements if the gravel fill does not meet the filtration criteria. The selection of a suitable geotextile for filtration purposes shall be supported by design computations taking in to account the actual gradations of the gravel fill and the reinforced wall fill to be used on the project. Gradation for gravel fill shall be tested at the frequency of 1 test per 50 yd³ at the job site and for every change in the material source.

3.10 Certificate of Analysis for Modular Block Connection

For modular block facing units, a certification shall be provided with detailed calculations according to AASHTO (2007) and the results of laboratory test results performed in accordance with Section C.3 in Appendix B of FHWA NHI-10-025, dated 2009 (“Mechanically Stabilized Earth Walls and Reinforced Soil Slopes – Volume II”). Such certification shall demonstrate that all connections, including block-to-reinforcement and block-to-block connections, and all related components meet or exceed the current AASHTO 75 year design life requirements and are capable of resisting 100% of the maximum tension in the soil reinforcements at any level within the wall. Long-term connection testing for extensible reinforcements is also required. The effect of wall batter and normal pressures representative of the full range of wall configurations and heights shall be incorporated in the tests.

4 Construction Requirements:

4.01 Excavation:

The contractor shall ensure that temporary slopes are safe during the period of wall construction, and shall adhere to all applicable local, state and federal regulations. During construction of the MSE walls, the contractor shall design, construct, maintain and, when called for, remove temporary excavation support systems (shoring). Temporary excavation support systems may be left in place if approved by the Engineer. The back slope of the excavation shall be benched. Where shoring is required, the contractor shall submit the shoring design, and a plan outlining construction and removal procedures, to the Engineer for review and approval prior to proceeding with the work. Shoring plans shall be prepared and submitted as part of the working drawings, as specified in Subsection [REDACTED] and shall bear the seal and signature of a licensed Professional Civil or Structural Engineer, registered in the State. All shoring design shall include appropriate input and review by a geotechnical engineer.

4.02 Foundation Preparation:

(A) General:

In the absence of specific ground improvement requirements in the plans and special provisions, the following applies:

The foundation for the reinforced wall fill and retained backfill shall be graded level for the entire area of the base of such backfills, plus an additional 12 inches on all sides, or to the limits shown in the plans.

If soil reinforcement components are to be positioned on native soil, the top one (1) foot of native soil shall meet the requirements of the reinforced backfill material specified in Subsection 3.05.

If soil reinforcement components are to be positioned on native rock mass, the rock mass shall be classified as at least Class II rock mass in accordance with Section 10 of 4th Edition of AASHTO (2007) Bridge Specifications. Otherwise the top foot of native rock mass on which the MSE structure is to be constructed shall be scarified and compacted to a dry density not less than 100 percent of maximum dry density as determined in accordance with AASHTO T 99 OR AASHTO T 180 . EDIT NOTE: Specify one method, consistent with compaction specification.

(B) Proof-Rolling:

The contractor shall perform proof-rolling to evaluate the stability and uniformity of the subgrades on which the MSE structure will be constructed. Proof rolling shall be performed on the entire areas at the following locations:

1. At the bottom of the overexcavation and recompaction zones, if specified on the plans.
2. At the bottom of the overexcavation and replacement zones, if specified on the plans.
3. At the base of all walls.
4. At the top of native soil layers that have been scarified, moisture-conditioned, and recompacted (if different from the bottom of the overexcavation and recompaction zones, or overexcavation and replacement zones).

Proof-rolling shall be done immediately after subgrade compaction while the moisture content of the subgrade soil is near optimum, or at the moisture content that was used to achieve the required compaction.

If proof-rolling is performed after installation of pipe underdrains, the proof-roller shall not be used within 1½ feet of the underdrains.

Proof-rolling shall be performed with a pneumatic-tired tandem axle roller with at least three wheels on each axle, a gross weight of 25 tons (50 kips), a minimum tire pressure of 75 pounds per square inch, and a minimum rolling width of 75 inches. A Caterpillar PS-300B (or PF-300B), Ingersoll-Rand PT-240R, BOMAG BW24R, Dynapac CP271, or equipment with equivalent capabilities shall be used for proof-rolling.

Proof-rolling equipment shall be operated at a speed between 1.5 and 3 miles per hour, or slower as required by the Engineer to permit measurements of the deformations, ruts and/or pumping.

Proof-rolling shall be carried out in two directions at right angles to each other with no more than 24 inches between tire tracks of adjacent passes. The contractor shall operate the proof-roller in a pattern that readily allows for the recording of deformation data and complete coverage of the subgrade.

The following actions shall be taken based on the results of the proof-rolling activity:

1. Rutting less than ¼-inch – The grade is acceptable.
2. Rutting greater than ¼-inch and less than 1½ inches – The grade shall be scarified and re-compacted.
3. Rutting greater than 1½ inches – The compacted area shall be removed and reconstructed.
4. Pumping (deformation that rebounds, or materials that are squeezed out of a wheel's path) greater than one(1) inch – The area shall be remediated as directed by the Engineer.

The contractor shall be responsible for maintaining the condition of the approved proof-rolled soils throughout the duration of the retaining wall construction. Wall construction shall not commence until the foundation has been approved by the Engineer.

4.03 Concrete Leveling Pad:

Leveling pads shall be constructed of unreinforced concrete as shown on the working drawings. Gravel leveling pads shall not be allowed. As a minimum, the concrete for leveling pads shall meet the requirements of Section . The elevation of the top of leveling pad shall be within ⅛ inch from the design elevation when measured by a straightedge over any 10-foot run of the leveling pad.

The minimum width of the leveling pad shall be the width of the facing unit plus 8-inches. The centerline of the leveling pad shall be within 1 inch from design location. When the facing units are centered on the leveling pad, the leveling pad shall extend approximately 4-inches beyond the limits of the facing unit as measured in the direction perpendicular to the face of the wall.

Cast-in-place leveling pads shall be cured for a minimum of 24 hours before placement of wall facing units. A geotextile shall be applied over the back of the area of any openings between the facing units and leveling pad steps. The geotextile shall extend a minimum of six (6) inches beyond the edges of the opening. The opening shall be filled with concrete, conforming to Section , or shall be concurrently backfilled on both sides with soil.

4.04 Subsurface Drainage:

Prior to wall erection, the contractor shall install a subsurface drainage system as shown on the working drawings.

4.05 Wall Erection:

(A) General:

Walls shall be erected in accordance with the manufacturer's written instructions. The contractor shall be responsible for ensuring that a field representative from the manufacturer is available at the site during construction of the initial 10-foot height of the full length of wall, and as called upon thereafter by the Engineer, to assist the contractor and Engineer at no additional cost to the Agency. All temporary construction aids (e.g., wedges, clamps, etc.) shall be in accordance with the manufacturer's recommendations.

(B) Placement Tolerances for Walls with Precast Facing:

For walls with rigid facing, such as precast concrete panels, the panels shall be placed such that their final position is vertical or battered as shown on the working drawings. As wall fill material is placed, the panels shall be maintained in the correct vertical alignment by means of temporary wedges, clamps, or bracing as recommended by the manufacturer. A minimum of two, but not more than three, rows of panel wedges shall remain in place at all times during wall erection. Wedges shall be removed from lower rows as panel erection progresses, so as to prevent chipping or cracking of concrete panels. The contractor shall repair any damage to erected concrete panels as directed by the Engineer and to the Engineer's satisfaction. No external wedges in front of the wall shall remain in place when the wall is complete.

Erection of walls with panel facing shall be in accordance with the following tolerances:

- Vertical and horizontal alignment of the wall face shall not vary by more than $\frac{3}{4}$ inch when measured along a 10-foot straightedge.
- The overall vertical tolerance (plumbness) of the finished wall shall not exceed $\frac{1}{2}$ inch per 10 feet of wall height. Negative (outward leaning) batter is not acceptable.
- The maximum permissible out of plane offset at any panel joint shall not exceed $\frac{3}{8}$ inch.
- The final horizontal and vertical joint gaps between adjacent facing panel units shall be within $\frac{1}{8}$ inch and $\frac{1}{4}$ inch, respectively, of the design final joint opening per the approved calculations required in Subsection 3.01(H).

Wall sections not conforming to these tolerances shall be reconstructed at no additional cost to the Department.

(C) Placement Tolerances for Permanent Walls with Flexible Facing:

Erection of permanent walls with flexible facing (such as welded wire mesh) shall be in accordance with the following tolerances:

- Vertical and horizontal alignment of the wall face shall not vary by more than two (2) inches when measured along a 10-foot straightedge, or as shown in the plans and specifications.

- The overall vertical tolerance (plumbness) of the wall shall not exceed one (1) inch per 10 feet of wall height. Negative (outward leaning) batter is not acceptable.
- The offset limit between consecutive rows of facing shall not exceed one (1) inch from planned offset.

Wall sections not conforming to these tolerances shall be reconstructed at no additional cost to the Department.

(D) Placement Tolerances for Modular Block Units:

Erection of walls with Modular Block Units shall be as per the following requirements:

- Vertical and horizontal alignment of the wall face shall not vary by more than 3/4-inch when measured along a 10-foot straightedge.
- Overall vertical tolerance (plumbness) of the wall shall not exceed 1 1/4-inch per 10-ft of wall height from the final wall batter. Negative (outward leaning) batter is not acceptable.
- The first row of units shall be level from unit-to-unit and from front-to-back. Use the tail of the units for alignment and measurement.
- All units shall be laid snugly together and parallel to the straight or curved line of the wall face.
- Unless otherwise noted, all blocks shall be dry-stacked and placed with each block evenly spanning the joint in the row below (running bond). Shimming or grinding shall control the elevations of any two adjacent blocks within 1/16 inch.
- The top of blocks shall be checked with a minimum length of 3-feet long straight edge bubble level. Any high points identified by the straight edge shall be ground flat. Block front to back tilting shall be checked frequently, however correction by shimming shall be done no later than 3 completed courses.

Wall sections not conforming to these tolerances shall be reconstructed at no additional cost to the Department.

(E) Placement of Metallic Reinforcement Elements:

Metallic reinforcement elements shall be placed normal (perpendicular) to the face of the wall, unless otherwise shown on the approved plans. All reinforcement shall be structurally connected to the wall face.

At each level of the soil reinforcement, the reinforced wall fill material shall be roughly leveled and compacted before placing the next layer of reinforcement. The reinforcement shall bear uniformly on the compacted reinforced soil from the connection to the wall to the free end of the reinforcing elements. The reinforcement placement elevation shall be at the connection elevation to two (2) inches higher than the connection elevation.

Where overlapping of reinforcing may occur, such as at corners, reinforcing connections to panels shall be adjusted to maintain at least three (3) inches of vertical separation between overlapping reinforcement.

(F) Placement of Geotextile:

All joints between precast concrete panels shall be covered with geotextile on the backside of the wall. Adhesive shall be applied to panels only. Adhesive shall not be applied to geotextile fabric or within two (2) inches of a joint. The contractor shall provide geotextile having a minimum width of 12 inches, and shall overlap fabric a minimum of four (4) inches. For modular block walls, the placement of the geotextile fabric shall be in accordance with the plans.

(G) Joint Pads and Fillers:

The contractor shall install joint pads and fillers as shown on the working drawings.

(H) Placement of Geosynthetic Reinforcement:

Geosynthetic reinforcement shall be installed in accordance with the manufacturer's site-specific wall erection instructions.

Geosynthetic reinforcement shall be placed in continuous longitudinal rolls in the direction of the main reinforcement. Joints parallel to the wall shall not be permitted, except as shown on the working drawings.

Reinforcement coverage shall be 100 percent of embedment area unless otherwise shown in the working drawings. Adjacent sections of geosynthetic reinforcement need not be overlapped except when exposed in a wrap-around face system, at which time the reinforcement rolls shall be overlapped or mechanically connected per the manufacturer's requirements.

Geosynthetic reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geosynthetic reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geosynthetic reinforcement in position until the subsequent soil layer can be placed.

During construction, the surface of the fill shall be kept approximately horizontal. Geosynthetic reinforcement shall be placed directly on the compacted horizontal fill surface. The reinforcement shall bear uniformly on the compacted reinforced soil from the connection to the wall to the free end of the reinforcing elements. The reinforcement placement elevation shall be at the connection elevation to two (2) inches higher than the connection elevation.

4.06 Reinforced Wall Fill Placement:

(A) General:

Reinforced wall fill placement shall closely follow erection of each course of facing panels. Backfill shall be placed in such a manner to avoid damage or disturbance of the wall materials, misalignment of facing panels, or damage to soil reinforcement or facing members. The contractor shall place backfill to the level of the connection and in such a manner as to ensure that no voids exist directly beneath reinforcing elements.

For walls with modular block facing units, the backfill shall not be advanced more than the height of a modular block unit until the drainage fill, core fill and all fill in all openings within the blocks at

that level have been placed. The filled units shall be swept clean of all debris before installing the next level of units and/or placing the geogrid materials.

For walls with flexible facing with gabion style facing, the rock near the wall face shall be hand-placed in accordance with the recommendations of the wall manufacturer.

The maximum lift thickness before compaction shall not exceed ten (10) inches. EDIT NOTE: Insert Agency maximum lift height. The contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

For geosynthetic reinforcements, the fill shall be spread by moving the machinery parallel to or away from the wall facing and in such a manner that the geogrid remains taut. Construction equipment shall not operate directly on the geogrid. A minimum fill thickness of six (6) inches over the geogrid shall be required prior to operation of vehicles. Sudden braking and sharp turning shall be avoided.

For metallic reinforcements, the fill shall be spread by moving the machinery parallel to or away from the wall facing and in such a manner that the steel reinforcement remains normal to the face of the wall. Construction equipment shall not operate directly on the steel reinforcement. A minimum fill thickness of three (3) inches over the steel reinforcement shall be required prior to operation of vehicles. Sudden braking and sharp turning shall be avoided.

Wall materials which are damaged during backfill placement shall be removed and replaced by the contractor, at no additional cost to the Department. The contractor may submit alternative corrective procedures to the Engineer for consideration. Proposed alternative corrective procedures shall have the concurrence of the MSE wall supplier and designer, in writing, prior to submission to the Engineer for consideration. All corrective actions shall be at no additional cost to the Department.

(B) Compaction:

Reinforced wall fill shall be compacted to 95 percent of the maximum dry density as determined in accordance with the requirements of AASHTO T 99 OR AASHTO T 180. EDIT NOTE: Specify one method, consistent with compaction specification.

Retained backfill shall be compacted to 95 percent of the maximum dry density as determined in accordance with the requirements of AASHTO T 99 (Standard Proctor) OR AASHTO T 180 (Modified Proctor). EDIT NOTE: Specify one method, consistent with compaction specification.

Backfill shall be compacted using a static-weighted or vibratory roller. Sheeps-foot or grid-type rollers shall not be used for compacting material within the limits of the soil reinforcement. The contractor shall take soil density tests, in accordance with _____, to ensure compliance with the specified compaction requirements. Soil density tests shall be taken at intervals of not less than one for every 2000 cubic yards, with a minimum of one test per lift. Compaction tests shall be taken at locations determined by the Engineer.

The backfill density requirement within three (3) feet of the wall facing shall be 90 percent of maximum dry density as determined by AASHTO T 99 (Standard Proctor) OR AASHTO T 180 (Modified Proctor). EDIT NOTE: Specify one method, consistent with compaction specification.. Compaction within three (3) feet of the wall shall be achieved by a minimum number of passes of a lightweight mechanical tamper or roller system. The minimum number of passes and rolling pattern shall be determined, prior to construction of the wall, by constructing a test pad section. The

minimum dimensions of the test pad shall be five (5) feet wide, 15 feet long, and three (3) feet final depth.

Compaction in the test pad section shall be performed as follows:

- Maximum lift thickness before compaction shall be eight (8) inches.
- Minimum one density test per lift.

Only those methods used to establish compaction compliance in the test pad section shall be used for production work. Any change in the material as per Table 5 or the approved equipment shall require the contractor to conduct a new test pad section and obtain re-approval by the Engineer of the minimum number of passes and rolling pattern. No measurement or payment will be made for test pad sections.

(C) Moisture Control:

The moisture content of the backfill material prior to and during compaction shall be uniformly dispersed throughout each layer. Backfill materials shall have a placement moisture content three (3) percent less than or equal to optimum moisture content, as determined in accordance with the requirements of AASHTO T 99 (Standard Proctor) OR AASHTO T 180 (Modified Proctor). **EDIT NOTE:** Specify one method, consistent with compaction specification, for the reinforced wall fill, and AASHTO T 99 (Standard Proctor) OR AASHTO T 180 for (Modified Proctor). **EDIT NOTE:** Specify one method, consistent with compaction specification, the retained backfill. Backfill material with a placement moisture content in excess of optimum shall be removed and reworked until the moisture content is uniform and acceptable throughout the entire lift.

(D) Protection of the Work:

The contractor shall not allow surface runoff from adjacent areas to enter the wall construction site at any time during construction operations. In addition, at the end of each day's operation, the contractor shall slope the last lift of backfill away from the wall facing so that runoff is directed away from the structure. If the subgrade is damaged due to water or otherwise, such that it does not meet the requirements of Subsection 4.02, then as directed by the Engineer, the contractor shall rework and repair the damaged subgrade at no additional expense to the Department. The criteria in Subsection 4.02 shall be used to judge the adequacy of the repair. Rework and repair shall extend to a depth where undamaged work is encountered.

5 Method of Measurement:

Mechanically Stabilized Earth (MSE) retaining walls will be measured by the square foot of completed wall. The vertical height will be taken as the difference in elevation measured from the top of wall to the top of the leveling pad. OR The pay area will be taken as the wall panel area supplied. **EDIT NOTE:** Specify one or the other option.

6 Basis of Payment:

The accepted quantities of Mechanically Stabilized Earth (MSE) retaining walls, measured as provided above, will be paid for at the contract unit price per square foot of wall, complete in place. Such price shall include full compensation for furnishing all designs, design revisions, associated working drawings, engineering calculations, labor, materials **EDIT NOTE:** may or may not include reinforced wall fill, see below, tools, equipment, and incidentals. Such price shall also include

provision of manufacturer's field representative, and all work involved in constructing the retaining walls, including foundation preparation, proof-rolling, footings, drainage features, wall facing, slip joints, concrete or shotcrete caps and aprons, rustication, paint or stain, grout, tendons, cables, anchors, fabric, and all hardware and reinforcing steel, complete in place as shown on the plans and as specified herein.

No separate measurement or payment will be made for excavation, reinforced wall fill, and retained backfill associated with retaining walls, the cost of such work being considered as included in the price paid for the MSE retaining wall. **EDIT NOTE:** Reinforced wall fill is a separate pay item for some Agencies, and is listed as such.

No separate measurement or payment will be made for the design, construction, or removal of temporary excavation support systems (shoring), or associated geotechnical review, the cost of such work being considered as included in the price paid for the MSE retaining wall.

10.10 CONSTRUCTION SPECIFICATIONS FOR REINFORCED SLOPE SYSTEMS

The availability of many different geosynthetic reinforcement materials as well as drainage and erosion control products requires consideration of different alternatives prior to preparation of contract documents so contractors are given an opportunity to bid using feasible, cost-effective materials. Any proprietary material should undergo an Agency review prior to inclusion as either an alternate offered during design (in-house) or construction (value engineering or end result) phase.

It is highly recommended that each Agency develop documented procedures for:

- Review and approval of geosynthetic soil reinforcing materials.
- Review and approval of drainage composite materials.
- Review and approval of erosion control materials.
- Review and approval of geosynthetic reinforced slope systems and suppliers.
- In-house design and performance criteria for reinforced slopes.

The following guidelines are recommended as the basis for specifications or special provisions for the furnishing and construction of reinforced soil slopes on the basis of pre approved reinforcement materials. Specification guidelines are presented for each of the following topics:

1. Specification Guidelines for RSS Construction (Agency design).
2. Specifications for Erosion Control Mat or Blanket.
3. Specifications for Geosynthetic Drainage Composite.
4. Specification Guidelines for Proprietary Geosynthetic RSS Systems.

10.10.1 Specification Guidelines For RSS Construction (Agency Design)

Description

Work shall consist of furnishing and placing geosynthetic soil reinforcement for construction of reinforced soil slopes.

Geosynthetic Reinforcement Material

The specific geosynthetic reinforcement material and supplier shall be pre approved by the Agency as outlined in the Agency's reinforced soil slope policy.

The geosynthetic reinforcement shall consist of a geogrid or a geotextile that can develop sufficient mechanical interlock with the surrounding soil or rock. The geosynthetic reinforcement structure shall be dimensionally stable and able to retain its geometry under construction stresses and shall have high resistance to damage during construction, ultraviolet degradation, and all forms of chemical and biological degradation encountered in the soil being reinforced.

The geosynthetics shall have a Nominal Long-Term Strength (T_{al}) and Pullout Resistance, for the soil type(s) indicated, as listed in Table S1 for geotextiles and/or Table S2 for geogrids.

The Contractor shall submit a manufacturer's certification that the geosynthetics supplied meet the respective index criteria set when the geosynthetic was approved by the Agency, measured in full accordance with all test methods and standards specified. In case of dispute over validity of values, the Engineer can require the Contractor to supply test data from an Agency approved laboratory to support the certified values submitted, the Contractor's cost.

Quality Assurance/Index Properties: Testing procedures for measuring design properties require elaborate equipment, tedious set up procedures and long durations for testing. These tests are inappropriate for quality assurance (QA) testing of geosynthetic reinforcements received on site. In lieu of these tests for design properties, a series of index criteria may be established for QA testing. These index criteria include mechanical and geometric properties that directly impact the design strength and soil interaction behavior of geosynthetics. **It is likely each family of products will have varying index properties and QC/QA test procedures.** QA testing should measure the respective index criteria set when the geosynthetic was approved by the Agency. Minimum average roll values, per ASTM D 4759, shall be used for conformance.

Table S-1. Required Geotextile Reinforcement Properties.

Geotextile ⁽¹⁾	Ultimate Strength (T_{ULT}) ASTM D4595 ⁽²⁾	Nominal Long-Term Strength ⁽³⁾ (T_{al})	For use with these Fills ⁽⁴⁾
A			GW-GM
A			SW-SM-SC
B			GW-GM
B			SW-SM-SC

NOTES:

- For geotextiles, minimum permeability $\geq$ ___ cm/s $\geq$ reinforced soil permeability. Minimum survivability properties – Class 1 per AASHTO M-288 specification.
- Based on minimum average roll values (MARV) (lb/ft {kN/m}).
- Nominal long-term strength (T_{al}) based on (lb/ft {kN/m})

$$T_{al} = \frac{T_{ULT}}{RF_D \times RF_{ID} \times RF_{CR}}$$
 where RF_{CR} is developed from creep tests performed in accordance with ASTM D5262, RF_{ID} obtained from site installation damage testing and RF_D from hydrolysis or oxidative degradation testing extrapolated to 75 or 100 year design life. For default reduction factors, include the durability requirements in Table 3-12 as additional reinforcement property requirements.
- Unified Soil Classification.

Table S-2. Required Geogrid Properties.

Geosynthetic ⁽¹⁾	Ultimate Strength (T_{ULT}) ASTM D6637 ⁽²⁾	Nominal Long-Term Strength ⁽³⁾ (T_{al})	For use with these Fills ⁽⁴⁾
A			GW-GM
A			SW-SM-SC
B			GW-GM
B			SW-SM-SC

NOTES:

- For geotextiles, minimum permeability $\geq$ ___ cm/s $\geq$ reinforced soil permeability. Minimum survivability properties – Class 1 per AASHTO M-288 specification.
- Based on minimum average roll values (MARV) (kN/m). Use D6637 for geogrids.
- Nominal long-term strength (T_{al}) based on (lb/ft {kN/m})

$$T_{al} = \frac{T_{ULT}}{RF_D \times RF_{ID} \times RF_{CR}}$$
 where RF_{CR} is developed from creep tests performed in accordance with ASTM D5262, RF_{ID} obtained from site installation damage testing and RF_D from hydrolysis or oxidative degradation testing extrapolated to 75 or 100 year design life. For default reduction factors, include the durability requirements in Table 3-12 as additional reinforcement property requirements.
- Unified Soil Classification.

Construction

Delivery, Storage, and Handling - Follow requirements set forth under materials specifications for geosynthetic reinforcement, drainage composite, and geosynthetic erosion mat.

Site Excavation - All areas immediately beneath the installation area for the geosynthetic reinforcement shall be properly prepared as detailed on the plans, specified elsewhere within the specifications, or directed by the Engineer. Subgrade surface shall be level, free from deleterious materials, loose, or otherwise unsuitable soils. Prior to placement of geosynthetic reinforcement, subgrade shall be proof-rolled to provide a uniform and firm surface. Any soft areas, as determined by the Owner's Engineer, shall be excavated and replaced with suitable compacted soils. The foundation surface shall be inspected and approved by the Owner's Geotechnical Engineer prior to fill placement. Benching the backcut into competent soil shall be performed as shown on the plans or as directed, in a manner that ensures stability.

Geosynthetic Placement - The geosynthetic reinforcement shall be installed in accordance with the manufacturer's recommendations, unless otherwise modified by these specifications. The geosynthetic reinforcement shall be placed within the layers of the compacted soil as shown on the plans or as directed.

- The geosynthetic reinforcement shall be placed in continuous longitudinal strips in the direction of main reinforcement. Joints in the design strength direction (perpendicular to the slope) shall not be permitted with geotextile or geogrid, except as indicated on the drawings.
- Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings. In the case of 100% coverage in plan view adjacent strips need not be overlapped.
- Adjacent rolls of geosynthetic reinforcement shall be overlapped or mechanically connected where exposed in a wrap-around face system, as applicable.
- Place only that amount of geosynthetic reinforcement required for immediately pending work to prevent undue damage. After a layer of geosynthetic reinforcement has been placed, the next succeeding layer of soil shall be placed and compacted as appropriate. After the specified soil layer has been placed, the next geosynthetic reinforcement layer shall be installed. The process shall be repeated for each subsequent layer of geosynthetic reinforcement and soil.
- Geosynthetic reinforcement shall be placed to lay flat and pulled tight prior to backfilling. After a layer of geosynthetic reinforcement has been placed, suitable means, such as pins or small piles of soil, shall be used to hold the geosynthetic reinforcement in position until the subsequent soil layer can be placed. Under no circumstances shall a track-type vehicle be allowed on the geosynthetic reinforcement before at least 6 in. (150 mm) of soil has been placed. Sudden braking and sharp turning – sufficient to displace fill – shall be avoided.
- During construction, the surface of the fill should be kept approximately horizontal. Geosynthetic reinforcement shall be placed directly on the compacted horizontal fill surface. Geosynthetic reinforcements are to be placed within 3 in. (75 mm) of the design elevations and extend the length as shown on the elevation view unless otherwise directed by the

Owner's Engineer. Correct orientation of the geosynthetic reinforcement shall be verified by the Contractor.

Fill Placement - Fill shall be compacted as specified by project specifications or to at least 95 percent of the maximum density determined in accordance with AASHTO T-99, whichever is greater.

- Density testing shall be made every 500 yd³ (420 m³) of soil placement or as otherwise specified by the Owner's Engineer or contract documents.
- Backfill shall be placed, spread, and compacted in such a manner to minimize the development of wrinkles and/or displacement of the geosynthetic reinforcement.
- Fill shall be placed in 12-inch (300 mm) maximum lift thickness where heavy compaction equipment is to be used, and 6-inch (150 mm) maximum uncompacted lift thickness where hand operated equipment is used.
- Backfill shall be graded away from the slope crest and rolled at the end of each work day to prevent ponding of water on surface of the reinforced soil mass.
- Tracked construction equipment shall not be operated directly upon the geosynthetic reinforcement. A minimum fill thickness of 6-in. (150 mm) is required prior to operation of tracked vehicles over the geosynthetic reinforcement. Turning of tracked vehicles should be kept to a minimum to prevent tracks from displacing the fill and the geosynthetic reinforcement.
- If approved by the Engineer, rubber-tired equipment may pass over the geosynthetic reinforcement at speeds of less than 25 mph (16 km/h). Sudden braking and sharp turning shall be avoided.

Erosion Control Material Installation. See *Erosion Control Material Specification* for installation notes.

Geosynthetic Drainage Composite. See *Geocomposite Drainage Composite Material Specification* for installation notes.

Final Slope Geometry Verification. Contractor shall confirm that as-built slope geometries conform to approximate geometries shown on construction drawings.

Method of Measurement

Measurement of geosynthetic reinforcement is on a square yard (meter) basis and will be computed on the total area of geosynthetic reinforcement shown on the construction drawings, exclusive of the area of geosynthetics used in any overlaps. Overlaps are an incidental item.

Basis of Payment

The accepted quantities of geosynthetic reinforcement by Type will be paid for per square yard (meter) in-place.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
Geogrid Soil Reinforcement – Type A	Square yard (meter)
Geogrid Soil Reinforcement – Type B	square yard (meter)
Or	
Geotextile Soil Reinforcement – Type A	square yard (meter)
Geotextile Soil Reinforcement – Type B	square yard (meter)

10.10.2 Specification for Erosion Control Mat or Blanket

Description

Work shall consist of furnishing and placing a synthetic erosion control mat and/or degradable erosion control blanket for slope face protection and lining of runoff channels for use in construction of reinforced soil slopes as shown on the plans or as specified by the Engineer.

Materials

(1) Erosion Control

The specific erosion control material and supplier shall be prequalified by the Agency prior to use.

Prequalification procedures and a current list of prequalified materials may be obtained by writing to the Agency. A 1 ft by 1 ft (0.3 m by 0.3 m) sample of the material may be required by the Engineer in order to verify prequalification.

The soil erosion control mat shall be a Class __ material and be one (1) of the following types as shown on the plans:

- (i) Type __. Long-term duration (Longer than 2 Years)
Shear Stress (t_d) > 2 psf (95 Pa) to < 5 psf (240 Pa)

Prequalified Type __ products are:

- (ii) Type __. Long-term duration (Longer than 2 Years)
Shear Stress (t_d) greater than or equal to 5 psf (240 Pa)

Prequalified Type __ products are:

_____	_____
_____	_____

Certification. The Contractor shall submit a manufacturer's certification that the erosion mat/blanket supplied meets the property criteria specified when the material was approved by the Agency. The manufacturer's certification shall include a submittal package of documented test results that confirm the property values. In case of dispute over validity of property values, the Engineer can require the Contractor to supply property test data from an approved laboratory to support the certified values submitted. Minimum average roll values, per ASTM D4759, shall be used for conformance.

(2) Staples.

Staples for anchoring the soil erosion control mat shall be U-shaped, made of 1/8 in. (3 mm) or large diameter steel wire, or other approved material, have a width of 1 to 2 in. (25 to 50 mm), and a length of not less than 18 in. (450 mm) for the face of RSS, and not less than 12 in. (300 mm) for runoff channels.

Construction Methods

(1) General.

The soil erosion control mat shall conform to the class and type shown on the plans. The Contractor has the option of selecting an approved soil erosion control mat conforming to the class and type shown on the plans, and according to the current approved material list.

(2) Installation.

The soil erosion control mat, whether installed as slope protection or as flexible channel liner in accordance with the approved materials list, shall be placed within 24 hours after seeding or sodding operations have been completed, or as approved by the Engineer. Prior to placing the mat, the area to be covered shall be relatively free of all rocks or clods over 1-½ inches (38 mm) in maximum dimension and all sticks or other foreign material which will prevent the close contact of the mat with the soil. The area shall be smooth and free of ruts or depressions exist for any reason, the Contractor shall be required to rework the soil until it is smooth and to reseed or resod the area at the Contractor's expense.

Installation and anchorage of the soil erosion control mat shall be in accordance with the project construction drawings unless otherwise specified in the contract or directed by the Engineer.

The erosion control material shall be placed and anchored on a smooth graded, firm surface approved by the Engineer. Anchoring terminal ends of the erosion control material shall be accomplished through use of key trenches. The material in the trenches shall be anchored to the soil with staples on maximum 20 in. (0.5 m) centers.

Erosion control material shall be anchored, overlapped, and otherwise constructed to ensure performance until vegetation is well established. Pins shall be as designated on the construction drawings, with a maximum spacing of 4 ft. (1.25 m) recommended.

Soil Filling. If noted on the construction drawings, the erosion control mat shall be filled with a fine grained topsoil, as recommended by the manufacturer. Soil shall be lightly raked or brushed on/into the mat to fill mat thickness or to a maximum depth of 1 in. (25 mm).

Method of Measurement

Measurement of erosion mat and erosion blanket material is on a square meter basis and will be computed on the projected slope face area from defined plan lines, exclusive of the area of material used in any overlaps, or from payment lines established in writing by the Engineer. Overlaps, anchors, checks, terminals or junction slots, and wire staples or wood stakes are incidental items.

Quantities of erosion control material as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions. Such variations in quantity will not be considered as alterations in the details of construction or a change in the character of work.

Basis of Payment

The accepted quantities of erosion control material will be paid for per square meter in place.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
Geosynthetic (Permanent) Erosion Control Mat	square yard (meter)
and/or	
Degradable (Temporary) Erosion Control Blanket	square yard (meter)

10.10.3 Specification for Geosynthetic Drainage Composite

Description

Work shall consist of furnishing and placing a geosynthetic drainage system as a subsurface drainage media for reinforced soil slopes.

Drainage Composite Materials

The specific drainage composite material and supplier shall be preapproved by the Agency.

The geocomposite drain shall be:

[insert approved materials that meet the project requirements. Geocomposites should be designed on a project specific basis. Design criteria for flow capacity, filtration, and permeability are summarized in the FHWA Geosynthetic, Design and Construction Guidelines (Holtz et al., 2008).]

OR

The geocomposite drain shall be a composite construction consisting of a supporting structure or drainage core material surrounded by a geotextile. The geotextile shall encapsulate the drainage core and prevent random soil intrusion into the drainage structure. The drainage core material shall consist of a three dimensional polymeric material with a structure that permits flow along the core laterally. The core structure shall also be constructed to permit flow regardless of the water inlet surface. The drainage core shall provide support to the geotextile. The core and fabric shall meet the minimum property requirements listed in Table S3.

A geotextile flap shall be provided along all drainage core edges. This flap shall be of sufficient width for sealing the geotextile to the adjacent drainage structure edge to prevent soil intrusion into the structure during and after installation. The geotextile shall cover the full length of the core.

The geocomposite core shall be furnished with an approved method of constructing and connecting with outlet pipes or weepholes as shown on the plans. Any fittings shall allow entry of water from the core but prevent intrusion of backfill material into the core material.

Certification and Acceptance. The Contractor shall submit a manufacturer's certification that the geosynthetic drainage composite supplied meets the design properties and respective index criteria measured in full accordance with all test methods and standards specified. The manufacturer's certification shall include a submittal package of documented test results that confirm the design values. In case of dispute over validity of design values, the Engineer can require the Contractor to supply design property test data from an approved laboratory, to support the certified values submitted. Minimum average roll values, per ASTM D4759, shall be used for conformance.

Table S3. Minimum Physical Property Criteria For Geosynthetic Drainage Composites In Reinforced Soil Slopes		
PROPERTY	TEST METHOD	VALUE ¹
<u>Composite:</u>	ASTM D4716	ft ² /s/ unit width (min)
Flow Capacity ²		
<u>Geotextile:</u>		
AOS ³	ASTM D4751	____ Max. Diameter (mm)
Permeability ⁴	ASTM D4491 ⁵	_____ cm/s
Trapezoidal Tear CLASS 2 ⁶ CLASS 3 ⁷	ASTM D4533	56 lb (250 N) 40 lb (180 N)
Grab Strength CLASS 2 ⁶ CLASS 3 ⁷	ASTM D4632	160 lb (700 N) 110 lb (500 N)
Puncture CLASS 2 ⁶ CLASS 3 ⁷	ASTM D6241	310 lb (1375 N) 40 lb (180 N)
Notes: 1. Values are minimum unless noted otherwise. Use value in weaker principal direction, as applicable. All numeric values represent minimum average roll values. 2. The flow capacity requirements for the project shall be determined with consideration of design flow rate, compressive load on the drainage material, and slope of drainage composite installation. 3. Both a maximum and a minimum AOS may be specified. Sometimes a minimum diameter is used as a criterion for improved clogging resistance. See FHWA Geosynthetic Design and Construction Guidelines (Holtz et al., 2008) for further information. 4. Permeability is project specific. A nominal coefficient of permeability may be determined by multiplying permittivity value by nominal thickness. The k value of the geotextile should be greater than the k value of the soil. 5. Standard Test Methods for Water Permeability (hydraulic conductivity) of Geotextiles by Permittivity. 6. CLASS 2 geotextiles are recommended where construction conditions are unknown or where sharp angular aggregate is used and a heavy degree of compaction (95% AASHTO T99) is specified. 7. CLASS 3 geotextiles (from AASHTO M-288) may be used with smooth graded surfaces having no sharp angular projections, no sharp aggregate is used, and compaction requirements are light (<95% AASHTO T99).		

Construction

Delivery, Storage, and Handling. The Contractor shall check the geosynthetic drainage composite upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the geosynthetic drainage composite shall be protected from temperatures greater than 140° F (60° C), mud, dirt, and debris. Follow manufacturer's recommendations in regards to

protection from direct sunlight. At the time of installation, the geosynthetic drainage composite shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. If approved by the Engineer, torn or punctured sections may be removed or repaired. Any geosynthetic drainage composite damaged during storage or installation shall be replaced by the Contractor at no additional cost to the Owner.

Placement. The soil surface against which the geosynthetic drainage composite is to be placed shall be free of debris and inordinate irregularities that will prevent intimate contact between the soil surface and the drain.

Seams. Edge seams shall be formed by utilizing the flap of geotextile extending from the geocomposite's edge and lapping over the top of the geotextile of the adjacent course. The geotextile flap shall be securely fastened to the adjacent fabric by means of plastic tape or non-water-soluble construction adhesive, as recommended by the supplier. Where vertical splices are necessary at the end of a geocomposite roll or panel, a 8-inch (200-mm)-wide continuous strip of geotextile may be placed, centered over the seam and continuously fastened on both sides with plastic tape or non water soluble construction adhesive. As an alternative, rolls of geocomposite drain material may be joined together by turning back the geotextile at the roll edges and interlocking the cuspidations approximately 2 in. (50 mm). For overlapping in this manner, the geotextile shall be lapped over and tightly taped beyond the seam with tape or adhesive. Interlocking of the core shall always be made with the upstream edge on top in the direction of water flow. To prevent soil intrusion, all exposed edges of the geocomposite drainage core shall be covered by tucking the geotextile flap over and behind the core edge. Alternatively, a 1 ft (300 mm) wide strip of geotextile may be used in the same manner, fastening it to the exposed fabric 8 in. (200 mm) in from the edge and fold the remaining flap over the core edge.

Repairs. Should the geocomposite be damaged during installation by tearing or puncturing, the damaged section shall be cut out and replaced completely or repaired by placing a piece of geotextile that is large enough to cover the damaged area and provide a sufficient overlap on all sides to fasten.

Soil Fill Placement. Structural backfill shall be placed immediately over the geocomposite drain. Care shall be taken during the backfill operation not to damage the geotextile surface of the drain. Care shall also be taken to avoid excessive settlement of the backfill material. The geocomposite drain, once installed, shall not be exposed for more than seven days prior to backfilling.

Method of Measurement

Measurement of geosynthetic drainage composite is on a square meter basis and will be computed on the total area of geosynthetic drainage composite shown on the construction drawings, exclusive of the area of drainage composite used in any overlaps. Overlaps, connections, and outlets are incidental items.

Quantities of drainage composite material as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions. Such variations in quantity will not be considered as alterations in the details of construction or a change in the character of work.

Basis of Payment

The accepted quantities of drainage composite material will be paid for per square meter in place.

Payment will be made under:

<u>Pay Item</u>	<u>Pay Unit</u>
Geosynthetic Drainage Composite	square yard (meter)

10.10.4 Specification Guidelines for Geosynthetic Reinforced Soil Slope Systems

Description

Work shall consist of design, furnishing materials, and construction of geosynthetic reinforced soil slope structure. Supply of geosynthetic reinforcement, drainage composite, and erosion control materials, and site assistance are all to be furnished by the slope system supplier.

Reinforced Slope System

Acceptable Suppliers - The following suppliers can provide Agency approved system:

- (1)
- (2)
- (3)

Materials. Only geosynthetic reinforcement, drainage composite, and erosion mat materials approved by the contracting Agency prior to project advertisement shall be utilized in the slope construction. Geogrid Soil Reinforcement, Geotextile Soil Reinforcement, Drainage Composite, and Geosynthetic Erosion Mat materials are specified under respective material specifications.

Design Submittal. The Contractor shall submit six sets of detailed design calculations, construction drawings, and shop drawings for approval within 30 days of authorization to proceed and at least 60 days prior to the beginning of reinforced slope construction. The calculations and drawings shall be prepared and sealed by a Professional Engineer, licensed in the State. Submittal shall conform to Agency requirements for RSS.

Material Submittals. The Contractor shall submit six sets of manufacturer's certification that indicate the geosynthetic soil reinforcement, drainage composite, and geosynthetic erosion mat meet the requirements set forth in the respective material specifications, for approval at least 60 days prior to start of RSS.

Construction

(Should follow the specifications details in this chapter)

Method of Measurement

Measurement of geosynthetic RSS Systems is on a vertical square foot basis.

Payment shall include reinforced slope design and supply and installation of geosynthetic soil reinforcement, reinforced soil fill, drainage composite, and geosynthetic erosion mat. Excavation of any unsuitable materials and replacement with select fill, as directed by the Engineer shall be paid under a separate pay item.

Quantities of reinforced soil slope system as shown on the plans may be increased or decreased at the direction of the Engineer based on construction procedures and actual site conditions.

Basis of Payment

The accepted quantities of geosynthetic RSS system will be paid for per vertical square foot (meter) in place.

Payment will be made under:

Pay Item

Pay Unit

Geosynthetic RSS System

Vertical square foot (meter)

CHAPTER 11

FIELD INSPECTION AND PERFORMANCE MONITORING

Construction of MSE and RSS systems is relatively simple and rapid. The construction sequence consists mainly of preparing the subgrade, placing and compacting backfill in normal lift operations, laying the reinforcing layer into position, and installing the facing elements (*tensioning of the reinforcement may also be required*) or outward facing for RSS slopes. Special skills or equipment are usually not required, and locally available labor can be used, however, experienced crews can provide higher production rates. Most material suppliers provide training for construction of their systems. The outline of a checklist showing general requirements for monitoring and inspecting MSE and RSS systems is provided in Table 11-1. The table should be expanded by the agency to include detailed requirements based on the agencies specifications and the specific project plans and specification requirements. Examples of detailed checklists for specific sections are provided later in this chapter.

There are some special construction considerations that the designer, construction personnel, and inspection team need to be aware of so that potential performance problems can be avoided. These considerations relate to the type of system to be constructed, to specific site conditions, the backfill material used and facing requirements. The following sections review items relating to:

- Section 11.1 - preconstruction reviews.
- Section 11.2 - prefabricated materials inspection.
- Section 11.3 - construction control.
- Section 11.4 - performance monitoring programs.

11.1 PRECONSTRUCTION REVIEWS

Prior to erection of the structure, personnel responsible for observing the field construction of the retaining structure must become thoroughly familiar with the following items:

- Plans and specifications.
- Site conditions relevant to construction requirements.
- Material requirements.
- Construction sequences for the specific reinforcement system.

Table 11-1. Outline of MSE/RSS Field Inspection Checklist Requirements.

- 1. **Read the specifications and become familiar with:**
 - material requirements
 - construction procedures
 - soil compaction procedures
 - alignment tolerances
 - acceptance/rejection criteria
- 2. **Review the construction plans and become familiar with:**
 - construction sequence
 - corrosion protection requirements
 - special placement to reduce damage
 - soil compaction restrictions
 - details for drainage requirements
 - details for utility construction
 - construction of slope face
 - contractor's documents
- 3. **Review material requirements and approval submittals.**
- 4. **Review construction sequence for the reinforcement system.**
- 4. **Check site conditions and foundation requirements. Observe:**
 - preparation of foundations
 - leveling pad construction (check level and alignment)
 - site accessibility
 - limits of excavation
 - construction dewatering
 - drainage features; seeps, adjacent streams, lakes, etc.
- 5. **On site, check reinforcements and prefabricated units. Perform inspection of prefabricated elements (i.e. casting yard) as required. Reject precast facing elements if:**
 - compressive strength < specification requirements
 - molding defects (e.g., bent molds)
 - honey-combing
 - severe cracking, chipping or spalling
 - color of finish variation
 - tolerance control
 - misaligned connections
- 6. **Check reinforcement labels to verify whether they match certification documents.**
- 7. **Observe materials in batch of reinforcements to make sure they are the same. Observe reinforcements for flaws and nonuniformity.**
- 8. **Obtain test samples according to specification requirements from randomly selected reinforcements.**
- 9. **Observe construction to see that the contractor complies with specification requirements for installation.**
- 10. **If possible, check reinforcements after aggregate or riprap placement for possible damage. This can be done either by constructing a trial installation, or by removing a small section of aggregate or riprap and observing the reinforcement after placement and compaction of the aggregate, at the beginning of the project. If damage has occurred, contact the design engineer.**
- 11. **Check all reinforcement and prefabricated facing units against the initial approved shipment and collect additional test samples.**
- 12. **Monitor facing alignment:**
 - adjacent facing panel joints
 - precast face panels
 - modular block walls
 - wrapped face walls
 - line and grade

11.1.1 Plans and Specifications

Specification requirements for MSE and RSS are reviewed in Chapter 10. The owner's field representatives should carefully read the specification requirements for the specific type of system to be constructed, with special attention given to material requirements, construction procedures, soil compaction procedures, alignment tolerances, and acceptance/rejection criteria. Plans should be reviewed. Unique and complex project details should be identified and reviewed with the designer and contractor, if possible. Special attention should be given to material handling and storage, the construction sequence, corrosion protection requirements for metallic reinforcement and UV protection for geosynthetics, special placement requirements to reduce construction damage of reinforcement, soil compaction restrictions, details for drainage requirements and utility construction, and construction of the outward slope. The contractor's documents should be checked to make sure that the latest issue of the approved plans, specifications, and contract documents are being used.

A checklist for review of MSE structures drawings is presented in Table 11-2 (FHWA NHI-08-094 and 095). A checklist for review of MSE specifications is presented in Table 11-3 (FHWA NHI-08-094/095).

11.1.2 Review of Site Conditions and Foundation Requirements

The site conditions should be reviewed to determine if there will be any special construction procedures required for preparation of the foundations, site accessibility, excavation for obtaining the required reinforcement length, and construction dewatering and other drainage features.

Foundation preparation involves the removal of unsuitable materials from the area to be occupied by the retaining structure including all organic matter, vegetation, and slide debris, if any. This is most important in the facing area to reduce facing system movements and, therefore, to aid in maintaining facing alignment along the length of the structure. The field personnel should review the borings to determine the anticipated extent of the removal required.

Where construction of reinforced fill will require a side slope cut, a temporary earth support system may be required to maintain stability. The contractor's method and design should be reviewed with respect to safety and the influence of its performance on adjacent structures. Caution is also advised for excavation of utilities or removal of temporary bracing or sheeting in front of the completed MSE structures. Loss of ground from these activities could result in settlement and lateral displacement of the retaining structure.

The groundwater level found in the site investigation should be reviewed along with levels of any nearby bodies of water that might affect drainage requirements. Slopes into which a cut is to be made should be carefully observed, especially following periods of precipitation, for any signs of seeping water (often missed in borings). Construction dewatering operations should be required for any excavations performed below the water table to prevent a reduction in shear strength due to hydrostatic water pressure.

MSE/RSS structures should be designed to permit drainage of any seepage or trapped groundwater in the retained soil. If water levels intersect the structure, it is also likely that a drainage structure behind and beneath the wall will be required. Surface water infiltration into the retained fill and reinforced fill should be minimized by providing an impermeable cap and adequate slopes to nearby surface drain pipes or paved ditches with outlets to storm sewers or to natural drains.

Internal drainage of the reinforced fill can be attained by use of a free-draining granular material that is free of fines (material passing No. 200 {0.075 mm} sieve should be less than 5 percent). Because of its high permeability, this type of fill will prevent retention of any water in the soil fill as long as a drainage outlet is available. Details are generally provided for drainage to the base of the fill as shown on Figures 5-6, 5-9 and 5-10, to avert water from exiting through the face of the wall, which could cause erosion and/or face stains. The drains will, of course, require suitable outlets for discharge of seepage away from the reinforced soil structure. Care should be taken to avoid creating planes of weakness within the structure with drainage layers.

Table 11-2. Checklist for Drawing Review. (after FHWA NHI-08-094/095)

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			<u>1.0 DOCUMENTS</u>
			1.1 Have you thoroughly reviewed the design drawings?
			1.2 Is there a set of all project drawings in the field trailer?
			1.3 Has the contractor submitted shop drawings?
			1.4 Have the shop drawings been approved by the designer and/or construction division manager?
			<u>2.0 LAYOUT</u>
			2.1 Have you located the horizontal and vertical control points?
			2.2 Do you know where the MSEW/RSS begins and ends?
			2.3 Have you identified any locations of existing utilities, signs, piles, lights that affect the proposed construction?
			2.4 Have you identified the elevations/ grade at top and at bottom of MSEWs/RSSs?
			2.5 Have you identified the existing and finished grades?
			2.6 Do you know where the construction limits are?
			2.7 Have you identified how the site will be accessed and any provisions for material storage?
			2.8 Is phased construction involved?
			<u>3.0 FOUNDATION PREPARATION</u>
			3.1 Are any special foundation treatments required?
			3.2 Is the foundation stepped?
			3.3 Is concrete leveling pad and the required elevation(s) shown on the drawings?
			3.4 Is shoring required?
			<u>4.0 DRAINAGE</u>
			4.1 Have you located the details for drainage?
			4.2 When must the drainage provisions be installed?
			4.3 Where does the drainage system outlet and does it allow for positive drainage?
			4.4 Are geotextile filters required?
			4.5 Is a drainage barrier (geomembrane) required for this project?
			<u>5.0 FACING</u>
			5.1 Have you identified the facing type, shape, size, and architectural finishing?
			5.2 Are there different types, colors, or sized facing units on the job?
			5.3 How do the facing units fit together?

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			5.4 Do you understand any corner/curve details?
			5.5 Do you understand bracing, bearing pads, wedging and shimming requirements?
			5.6 Is the facing battered?
			5.7 Are geotextile filters required for wall joints and is the placement shown on the drawings including overlaps and termination at the base and toe of the wall.
			<u>6.0 REINFORCING</u>
			6.1 What type of reinforcement is used in this project?
			6.2 Can you determine the length, location and type of reinforcement throughout the length and height of the wall or slope?
			6.3 Do you understand how the reinforcing connects to the facing?
			6.4 Have you identified any details for avoiding obstructions when placing reinforcement?
			6.5 Are cross sections showing reinforcement location? Are cross sections shown for each stationing and major elevation change?
			<u>7.0 BACKFILL</u>
			7.1 Are different types of fill required in different locations in the wall?
			<u>8.0 ANCILLARY ITEMS</u>
			8.1 Is there any coping specified in the drawings?
			8.2 Is there any traffic barrier or guard rail specified in the drawings?
			8.3 Have you considered interfaces with CIP structures?
			8.4 Do you understand the details for joints at or connections to CIP structures?
			8.5 Are any of the following involved in this project?
			8.5.1 Catch Basins/Drop Inlets
			8.5.2 Culverts/Pipes?
			8.5.3 Piles/Drilled Shafts?
			8.5.4 Utilities and other obstructions?
			8.6 Have you identified and do you understand any special detail to accommodate these obstructions?
			8.7 Do you know who is responsible for installation of each ancillary item?
			8.8 Are diversion ditches, collection ditches, or slope drains shown on the drawings?
			8.9 Is a permanent or temporary erosion control blanket required?
			8.10 Do you understand any erosion control details?

Table 11-3. Checklist for Specification Compliance. (after FHWA NHI-08-094/095)

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			<u>1.0 DOCUMENTS</u>
			1.1 Have you thoroughly reviewed the specifications?
			1.2 Is there a set of specifications in the field trailer?
			1.3 Are standard specifications or special provisions required in addition to the project specifications? Do you have a copy?
			<u>2.0 PRE-CONSTRUCTION QUALIFYING OF MATERIAL SOURCES / SUPPLIERS</u>
			2.1 Has the Contractor submitted pre-construction qualification test results (showing that it meets the gradation, density, electrochemical, and other soil-property requirements) for:
			2.1.1 Reinforced soil
			2.1.2 Retained soil
			2.1.3 Facing soil (if applicable)
			2.1.4 Drainage aggregate
			2.1.4 Graded granular filters (if applicable)
			2.2 Has the Contractor or Manufacturer submitted pre-construction qualification test results and/or Certificate of Compliance demonstrating that the facing materials comply with the applicable sections of the specifications including:
			2.2.1 Facing unit and connections
			2.2.2 Horizontal facing joint bearing pads
			2.2.3 Geotextile filter for facing joint
			2.3 Has the Contractor or Manufacturer submitted pre-construction qualification test results and/or Certificate of Compliance demonstrating that the reinforcing materials comply with the applicable sections of the specifications?
			2.4 Has the Contractor or Manufacturer submitted pre-construction qualification test results and/or Certificate of Compliance demonstrating that the drainage materials comply with the applicable sections of the specifications including:
			2.2.1 Geotextile filters (e.g., Type, AOS, permittivity, strength)
			2.2.2 Prefabricated Drains (i.e., geotextile filter and core)
			2.2.3 Drainage Pipe (material, type, ASTM designation and schedule)
			2.4 Has approval of the soil sources been officially granted for:
			2.4.1 Reinforced soil
			2.4.2 Retained soil

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			2.4.3 Facing soil
			2.4.4 Drainage aggregate
			2.5 Has approval of the facing material sources been officially granted
			2.6 Has approval of the reinforcing material sources been officially granted?
			<u>3.0 FOUNDATION PREPARATION</u>
			3.1 Has temporary shoring been designed and approved?
			<u>4.0 DRAINAGE</u>
			4.1 Is the Contractor or Manufacturer submitting QC test results <u>at the specified frequency</u> demonstrating that the drainage materials comply with the applicable sections of the specifications?
			4.2 Do the drainage materials delivered to the site correspond to the approved shop drawings?
			4.3 Do the identification labeling/markings on the geotextile filters and/or prefabricated drainage materials delivered to the site correspond to the pre-construction and QC submittals (date of manufacturing, lot number, roll numbers, etc.)?
			4.4 Have the drainage materials been inspected for damage due to transport, handling, or storage activities?
			4.5 Are the drainage materials properly stored to prevent damage, exposure to UV light, contamination?
			4.6 If any drainage materials were found damaged, have they been set aside, rejected, or repaired in accordance with the specifications?
			4.7 Has QA sampling of the drainage materials been performed at the required frequency?
			4.8 Does the QA lab know exactly which tests to run and the required test parameters?
			4.9 Do the QA test results for the drainage materials meet the specified property values?
			<u>5.0 FACING</u>
			5.1 Is the Contractor or Manufacturer submitting QC test results <u>at the specified frequency</u> demonstrating that the facing materials comply with the applicable sections of the specifications?
			5.2 Do the facing components delivered to the site correspond to the approved shop drawings including:
			5.2.1 Facing unit (shape, dimensions, reinforcement connections, overall quantity)?
			5.2.2 Horizontal facing joint bearing pads (material type, hardness, modulus)

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			5.2.3 Geotextile filter for facing joint (type, AOS, permittivity, strength)
			5.3 Do the identification labeling/markings on the facing units and components delivered to the site correspond to the pre-construction qualification and QC submittals (date of manufacturing, batch number, lot number, etc.)?
			5.4 Have the facing units and components been inspected for damage due to transport, handling, or storage activities?
			5.5 Are the facing units and components properly stored to prevent damage?
			5.6 If any facing units and components were found damaged, have they been rejected or repaired in accordance with the specifications?
			5.7 Has QA sampling of the facing units and component materials been performed at the required frequency?
			5.8 Does the QA lab know exactly which tests to run and the required test parameters?
			5.9 Do the QA test results for the facing unit and component materials meet the specified property values?
			<u>6.0 REINFORCING</u>
			6.1 Is the Contractor or Manufacturer submitting QC test results <u>at the specified frequency</u> demonstrating that the reinforcing materials comply with the applicable sections of the specifications?
			6.2 Do the reinforcing materials delivered to the site correspond to the approved shop drawings (strength, dimensions, overall quantity)?
			6.3 Do the identification labeling/markings on the reinforcing materials delivered to the site correspond to the pre-construction and QC submittals (date of manufacturing, lot number, roll numbers, etc.)?
			6.4 Have the reinforcing materials been inspected for damage due to transport, handling, or storage activities?
			6.5 Are the reinforcing materials properly stored to prevent damage, exposure to UV light, or corrosion?
			6.6 If any reinforcing materials were found damaged, have they been set aside, rejected, or repaired in accordance with the specifications?
			6.7 Has QA sampling of the reinforcing materials been performed at the required frequency?
			6.8 Does the QA lab know exactly which tests to run and the required test parameters?
			6.9 If pullout or interface shear testing is required, does the QA lab have enough of the applicable soil and the compaction criteria (in addition to the reinforcing materials)?
			6.10 Do the QA test results for the reinforcing materials meet the specified property values?

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			<u>7.0 BACKFILL</u>
			7.1 Is the Contractor submitting QC test results <u>at the specified frequency</u> for:
			7.1.1 Reinforced soil
			7.1.2 Retained soil
			7.1.3 Facing soil
			7.2 Does the QA lab know exactly which tests to run and the required test parameters?
			7.3 Do the QA test results for the various materials meet the specified property values:
			7.3.1 Reinforced Soil
			7.3.2 Retained Soil
			7.3.3 Facing Soil
			<u>8.0 ANCILLARY ITEMS</u>
			8.1 Do any ancillary materials delivered to the site correspond to the approved shop drawings (prefabricated copings, cap blocks and attachment glue, if required, catch basins, pipe, guardrail, etc.)?
			8.2 Do the identification labeling/markings on the ancillary materials delivered to the site correspond to the QC submittals (date of manufacturing, batch number, etc.)?
			8.3 Have the ancillary materials been inspected for damage due to transport, handling, or storage activities?
			8.4 Are the ancillary materials properly stored to prevent damage?
			8.5 If any ancillary materials were found damaged, have they been set aside, rejected, or repaired in accordance with the specifications?
			8.6 Have all requirements to sample/test any aspect of the work product <u>after</u> assembly, installation, compaction been met?

11.2 PREFABRICATED MATERIALS INSPECTION

Material components should be examined at the casting yard (for systems with precast elements) and on site. Typical casting operations are shown on Figure 11-1. Material acceptance should be based on a combination of material testing, certification, and visual observations.

When delivered to the project site, the inspector should carefully inspect all material (precast facing elements, reinforcing elements, bearing pads, facing joint materials, and reinforced backfill). On site, all system components should be satisfactorily stored and handled to avoid damage. The material supplier's construction manual should contain additional information on this matter.

11.2.1. Precast Concrete Elements

At the casting yard, the inspector should assure the facing elements are being fabricated in accordance with the agency's standard specifications. For example, precast concrete facing panels should be cast on a flat surface. Clevis loop embeds, tie strips, and other connection devices must not contact or be attached to the facing element reinforcing steel. Curing should follow required procedures and requirements (e.g., temperature, cover, moisture, etc.).

Facing elements delivered to the project site should be examined prior to erection. Panels should be rejected on the basis of the following deficiencies or defects:

- Insufficient compressive strength.
- Mold defects (e.g., bent molds).
- Honey-combing.
- Severe cracking, chipping, or spalling.
- Significant variation in color of finish.
- Out-of-tolerance dimensions.
- Misalignment of connection devices.

The following maximum facing element dimension tolerances are usually specified for precast concrete:

- Overall dimensions: 1/2-inch (13 mm)
- Connection device locations: 1-inch (25 mm)
- Clevis loop embeds: 1/8-inch (3 mm) horizontal alignment
- Element squareness: 1/2-inch (13 mm) difference between diagonals
- Surface finish: 1/8-inch in 5 ft (2 mm in 1 m) (smooth surface)
- Surface finish: 5/16-inch in 5 ft (5 mm in 1 m) (textured surface)



Figure 11-1. Casting yard for precast facing elements.

In cases where repair to damaged facing elements is possible, it should be accomplished to the satisfaction of the engineer.

For drycast modular blocks, it is essential that compressive strengths and water absorption be carefully checked on a lot basis. The following dimensional tolerances are usually specified:

- Overall dimensions: $\pm 1/8$ -inch (3.2 mm)
- Height of each block: $\pm 1/16$ -inch (1.6 mm)

11.2.2 Reinforcing Elements

Reinforcing elements (strips, mesh, sheets) should arrive at the project site securely bundled or packaged to avoid damage (see Figure 11-2). These materials are available in a variety of types, configurations, and sizes (gauge, length, product styles), and even a simple structure may have different reinforcement elements at different locations. The inspector should verify that the material is properly identified and check the specified designation (AASHTO, ASTM, or agency specifications). Grid reinforcement should be checked for wire diameter, length, width, and spacing of longitudinal and transverse members. For strip reinforcements, the length and thickness should be checked.

Material verification is especially important for geotextiles and geogrids where many product styles look similar but have different properties. In addition to the above measurements, geogrids or geotextile samples should be weighed in the field to compare the mass per unit area with the manufacturer's identification value. Samples should also be sent to the laboratory for verification testing. Color coding of roll ends can be helpful, especially in complex configurations to prevent improper installations. Where more than one style will be used, the roll ends could be painted and when reinforcements are cut to length, the lengths could be painted on the material as shown in Figure 11-2.

Galvanization (application thickness 2 oz/ft² {610 g/m²}), epoxy coatings (thickness 16 mils {0.41 mm}) or other coatings, should be verified by certification or agency conducted tests and checked for defects. Geosynthetic reinforcements should be properly packaged and protective wraps should be maintained during shipping and handling to protect the material from UV (e.g., sunlight) exposure.

Storage areas should meet both specifications and manufacturer's storage requirements. Materials should be stored off the ground to protect reinforcement from mud, dirt, and debris. Geosynthetic reinforcements should not be exposed to temperatures greater than 140° F (60°C) and manufacturer's recommendations should be followed in regards to UV protection from direct sunlight.



Figure 11-2. Inspect reinforcing elements: top photo shows a variety of reinforcements including metallic strips, welded wire mesh, and geosynthetics and bottom photo shows reinforcement length painted on geogrid reinforcement.

At the time of installation, the reinforcement should be rejected if it has defects, flaws, deterioration, or damage incurred during manufacture, transportation, or storage. Metal reinforcements should not contain bent, cut or repaired (e.g., welded or bent and straightened) sections without approval of the MSE wall or RSS design engineer of record. Geosynthetics should not contain tears, cuts or punctures and should be replaced or repaired at the direction of the design engineer.

11.2.3 Facing Joint Materials

Bearing pads (HDPE, EPDM, PVC and neoprene), joint filler and joint cover (e.g., geotextiles) should be properly packaged to minimize damage in unloading and handling. For example, polymer filler material and geotextiles, as previously indicated, must be protected from sunlight during storage.

Although these items are often considered as miscellaneous materials, it is important for the inspector to recognize that use of the wrong material or its incorrect placement can result in significant structure distress. Properties of these materials must be checked, either based on laboratory tests submitted by the supplier or preapproval (e.g., from a qualified products list), for conformance with specification requirements. Samples should be sent to the laboratory for verification testing.

11.2.4 Reinforced Backfill

The backfill in MSE/RSS structures is the key element in satisfactory performance. Both use of the appropriate material and its correct placement are important considerations. Reinforced backfill is normally specified to meet certain gradation, plasticity, soundness, and electrochemical requirements. Depending on the type of contract, tests to ensure compliance may be performed by either the contractor or the owner. The tests conducted prior to construction and periodically during construction for quality assurance form the basis for approval. During construction these tests include gradation and plasticity index testing at the rate required in the agency's or project-specific specifications (e.g., typically one test per 2000 yd³ (1500 m³) of material placed on large projects) and whenever the appearance and behavior of the backfill changes noticeably.

11.3 CONSTRUCTION CONTROL

Each of the steps in the sequential construction of MSE and RSS systems is controlled by certain method requirements and tolerances. Construction manuals for proprietary MSE systems should be obtained from the contractor to provide guidance during construction monitoring and inspection. A detailed description of general construction requirements for MSE walls follows with requirements that apply to RSS systems noted.

11.3.1 Leveling Pad

A concrete leveling pad should have minimum dimensions in conformance with the plans and specifications (typically 6 inches {150 mm} thick by the panel width plus 8 in. {200 mm} wide). The concrete compressive strength should also meet minimum specification requirements. Curing of cast-in-place pads should follow the requirements in the specifications (e.g., typically a minimum of 12 to 24 hours before facing units are placed). Careful inspection of the leveling pad to assure correct line, grade, and offset is important. A vertical tolerance of 1/8-inch (3 mm) to the design elevation is recommended. If the leveling pad is not at the correct elevation, the wall will likely be difficult to construct and the leveling pad elevation should be corrected. An improperly placed leveling pad can result in subsequent facing unit misalignment, cracking, and spalling. Full height precast facing elements may require a larger leveling pad to maintain alignment and provide temporary foundation support. Gravel pads of suitable dimensions may be used with modular block walls used for landscaping type applications. Typical installations are shown on Figure 11-3.

11.3.2 Erection of Facing Elements

Precast facing panels are purposely set at a slight backward batter (toward the reinforced fill) in order to assure correct final vertical alignment after backfill placement as shown on Figure 11-4. Minor outward movement of the facing elements from wall fill placement and compaction cannot be avoided and is expected as the interaction between the reinforcement and reinforced backfill occurs. Typical backward batter for segmental precast panels is 1/2-in. in 4 ft (20 mm per meter) of panel height with steel reinforcements. Modular block units are typically stacked with an offset 1/2 to 1 in. to account for horizontal movements.

Full height precast panels as shown on Figure 11-5 are more susceptible cracking during backfilling and misalignment difficulties than segmental panels. When using full-height panels, the construction procedure should be carefully controlled to maintain tolerances. Special construction procedures such as additional bracing and larger face panel batter may be necessary.



(a)



(b)



(c)

Figure 11-3. Concrete leveling pad showing: a) leveling the concrete, b) completed pad, and c) placing the facing elements on the leveling pad.

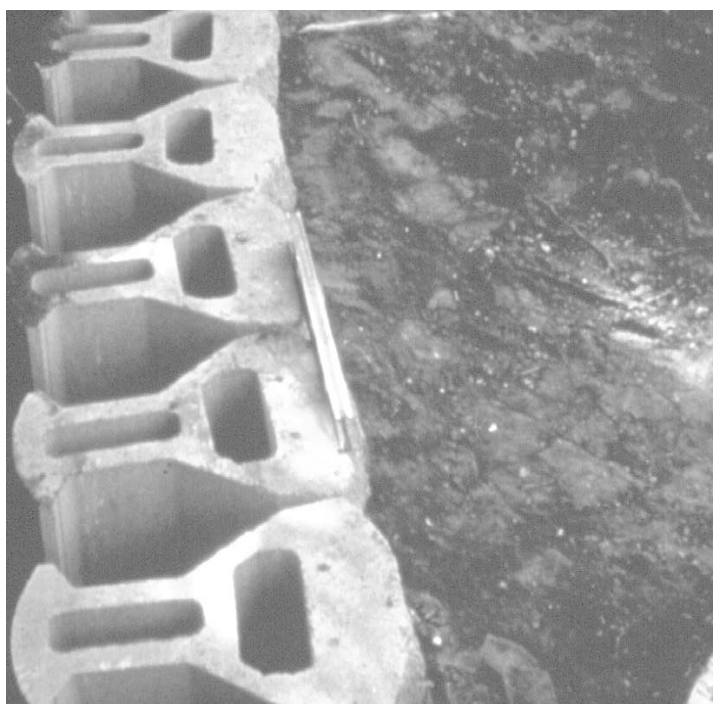


Figure 11-4. Checking facing element batter and alignment.

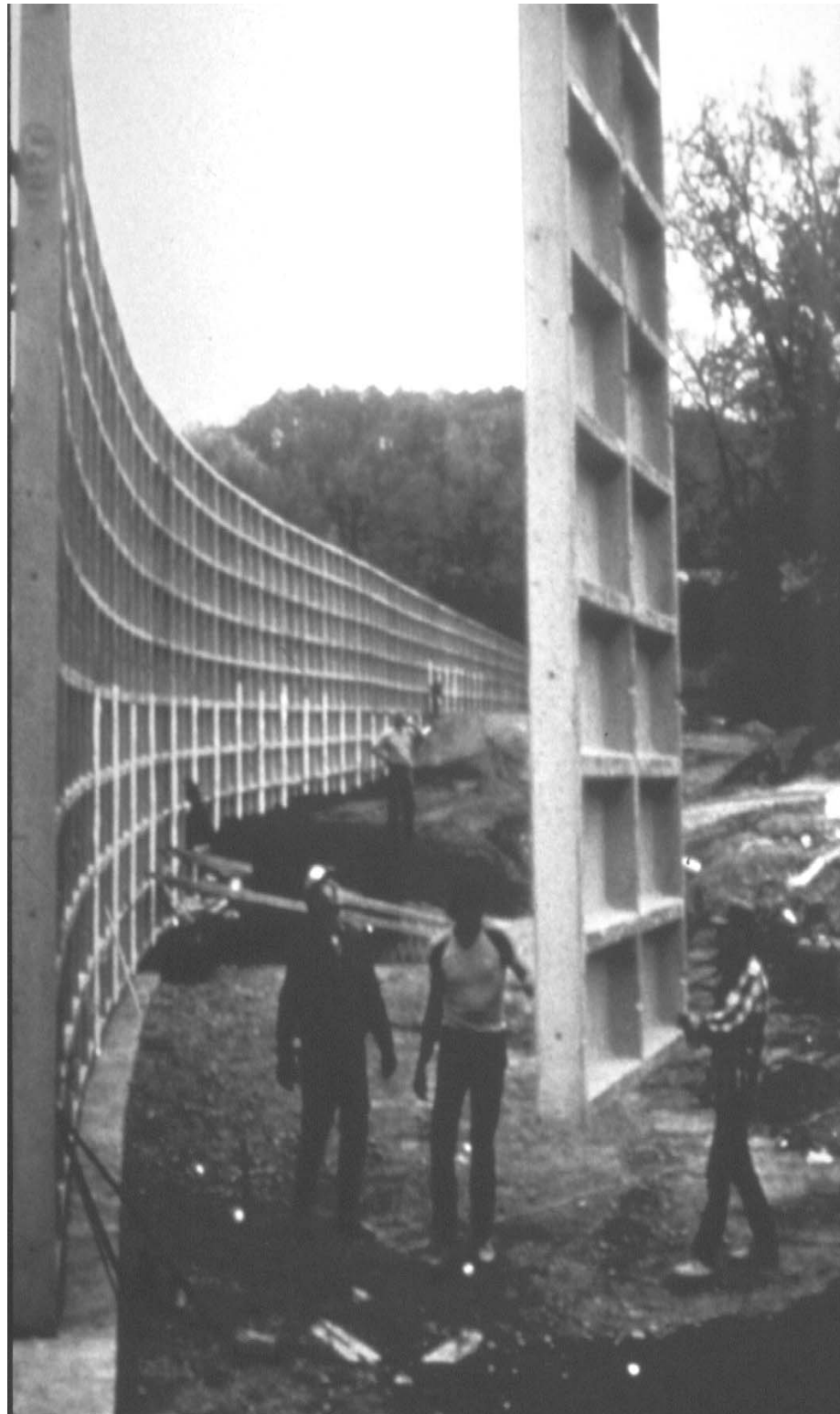


Figure 11-5. Full height facing panels require special alignment care.

First Row of Facing Elements. Setting the first row of facing elements is a key detail as shown in Figure 11-6. Construction should always begin adjacent to any existing structure and proceed toward the open end of the wall. The facing units should be set directly on the concrete leveling pad. Horizontal joint material or shims generally should not be permitted between the first course of panels and the leveling pad unless specifications specifically allow for and provide detail requirements (e.g., material type, properties, maximum thickness (e.g., 1/16-inch, 1/8-inch) and other dimensional requirements) for such materials. Temporary wood wedges may be used between the first course of concrete panels and the leveling pad to set panel batter, but they must be removed during subsequent construction. Some additional important details are:

- The first row of segmental panels must be braced until the bottom several layer(s) of reinforcements has been backfilled. Adjacent panels should be clamped together to prevent individual panel displacement.
- After setting and battering the first row of panels or placing the first row of modular blocks, horizontal alignment should be visually checked (i.e., with survey instruments or with a string-line).
- When using full-height panels, initial bracing and clamping are even more critical because misalignments are difficult to correct as construction continues.
- Most MSE systems use a variety of panel sizes to best fit the wall envelope. Special panels or modular block types may also be used to accommodate aesthetic treatments design requirements (geometric shape, size, color, finish, connection points). The facing element types must be checked to make sure that they are installed exactly as shown on the plans.
- A geotextile filter should be placed over the back of the area of any openings between the facing units and the leveling pad. The geotextile should extend a minimum of 6-in. (150 mm) beyond the edges of the openings. For large openings > than 1 in. (25 mm) in width (such as where stepped leveling pads are required or wall drain outlets are placed over the leveling pad), the openings should either be filled in with concrete or the section should be concurrently backfilled on both sides of the facing unit with soil.



Figure 11-6. Setting first row of precast facing elements.

11.3.3 Reinforced Fill Placement, Compaction

Moisture and density control is imperative for construction of MSE and RSS systems. Even when using high-quality granular materials, problems can occur if compaction control is not exercised. Reinforced wall fill material should be placed and compacted at or within 2 percent dry of the optimum moisture content. If the reinforced fill is free draining with less than 5 percent passing a No. 200 (0.075 mm) U.S. Sieve, water content of the fill may be within ± 3 percentage points of the optimum. Placement moisture content can have a significant effect on reinforcement-soil interaction. Moisture content wet of optimum makes it increasingly difficult to maintain an acceptable facing alignment, especially if the fines content is high. Moisture contents that are too dry may not achieve required density and could result in significant settlement during periods of precipitation (i.e., due to bulking).

A density of 95 percent of T-99 maximum value or 90 percent of T-180 is typically recommended for retaining walls and slopes, and 100 percent of T-99 or 95% of T-180 is usually recommended for abutments and walls or slopes supporting structural foundations. A procedural specification is preferable where a significant percentage of coarse material, generally 30 percent or greater retained on the $\frac{3}{4}$ -inch (19 mm) sieve, prevents the use of the AASHTO T-99 or T-180 test methods. In this situation, typically four to five passes with conventional vibratory roller compaction equipment is adequate to attain the maximum practical density. The actual requirements should be determined based on a test section as discussed in Chapter 3, Section 3.2.1.

Reinforced backfill should be dumped onto or parallel to the rear and middle of the reinforcements and bladed toward and away from the front face as shown on Figure 11-7. At no time should any construction equipment be in direct contact with the reinforcements because the reinforcements can be damaged. Soil layers should be compacted up to 2 in. (50 mm) above but no less than even with the elevation of each level of reinforcement connections prior to placing that layer of reinforcing elements.

Compaction Equipment - With the exception of the 3-foot (1-m) zone directly behind the facing elements or slope face, large, smooth-drum, vibratory rollers should be used to obtain the desired compaction as shown on Figure 11-8a. Sheepsfoot and grid type rollers should not be permitted because of possible damage to the reinforcements. When compacting uniform medium to fine sands (in excess of 60 percent passing a No. 40 sieve) use a smooth-drum static roller or lightweight (walk behind) vibratory roller, especially for the last pass. The use of large vibratory compaction equipment with this type of backfill material will make wall alignment control difficult and actually may loosen the upper surface of the soil.



Figure 11-7. Placement of reinforced fill.



Figure 11-8. Compaction equipment showing: a) large equipment permitted away from face; and b) lightweight equipment within 3 ft (1 m) of the face.

Within 3 ft (1 m) of the wall or slope face, use small single or double drum, walk-behind vibratory rollers or vibratory plate compactors as shown in Figure 11-8b. Placement of the reinforced backfill near the front should not lag behind the remainder of the structure by more than one lift. Poor fill placement and compaction in this area has in some cases resulted in facing movement and/or downdrag on reinforcements, which increases connection stresses. Within this 3 ft (1 m) zone, quality control should be maintained by a method specification such as four passes of a light, walk-behind vibratory plate or drum compactor. Test pads should be constructed to determine the actual number of passes and lift thickness required to achieve compaction requirements with the compaction equipment to be used. Higher quality fill is sometimes used in this zone so that the desired properties can be achieved with less compactive effort. Excessive compactive effort or use of too heavy equipment near the wall face could result in excessive face unit movement (segmental panels and modular blocks) or structural damage (full-height, precast panels), and overstressing of reinforcement layers. For welded wire wall facing systems, caution must be exercised such that struts do not become dislodged during placement of backfill and compaction, which could jeopardize the wall face integrity.

Inconsistent compaction and undercompaction caused by insufficient compactive effort or allowing the contractor to "compact" backfill with trucks and dozers will lead to gross misalignments and settlement problems and should not be permitted. Flooding of the backfill to facilitate compaction should also not be permitted. Compaction control testing of the reinforced backfill should be performed on a regular basis during the entire construction project. A minimum frequency of one test within the reinforced soil zone per lift for every 150 ft (45 m) of wall is recommended.

11.3.4 Placement of Reinforcing Elements

Reinforcing elements for MSE and RSS systems should be installed in strict compliance with spacing and length requirements shown on the plans. Reinforcements should generally be placed perpendicular to the back of the facing panel. In specific situations (e.g., abutments and curved walls) it may be permissible to skew the reinforcements from their design location in either the horizontal or vertical direction. Skewing should not exceed the limits defined in the specifications and overlapping layers of reinforcements should be separated by 3-in. (75-mm) minimum thickness of fill.

Curved walls create special considerations with MSE panel and reinforcement details. Different placement procedures are generally required for convex and concave curves. For reinforced fill systems with precast panels or modular blocks, joints will either be further closed or opened by nominal facing movements that normally occur during construction.

Special considerations also arise when constructing MSE/RSS structures around deep foundation elements or drainage structures. For deep foundations either drive piles prior to face construction or use hollow sleeves at proposed pile locations during reinforced fill erection. The latter method is generally preferred. Predrilling for pile installation through the reinforced soil structure between reinforcements is risky and should be avoided. Reinforcement skew to avoid obstructions must be within specification tolerances and in no case should reinforcements be cut or excessively bent.

Connections. Each MSE system has a unique facing connection detail. Several types of connections are shown on Figure 11-9. Connections are manufacturer specific and must be made in accordance with the approved drawings. For example on Reinforced Earth structures bolts are inverted between tie strips making a connection that acts in shear (i.e., double shear on the connector). Nuts are securely tightened with hand tools.

Flexible reinforcements, such as geotextiles and geogrids, usually require pretensioning to remove any slack in the reinforcement and in the connection to the facing unit. The tension is then maintained by staking or by placing fill during tensioning. Tensioning and staking will reduce subsequent horizontal movements of the panel as the wall fill is placed.

11.3.5 Placement of Subsequent Facing Courses (Segmental Facings)

Throughout construction of segmental panel walls, facing panels should only be set at grade. Placement of a panel on top of one not completely backfilled should not be permitted.

Alignment Tolerances. The key to a satisfactory end product is maintaining reasonable horizontal and vertical alignments during construction. Generally, the degree of difficulty in maintaining vertical and horizontal alignment increases as the vertical distance between reinforcement layers increases. The following alignment tolerances are recommended:

- Adjacent facing panel joint gaps (all reinforcements): $\frac{3}{4}$ -inch $\pm$ $\frac{1}{4}$ -inch (19 mm $\pm$ 6 mm)
- Precast face panel (all reinforcements): 1/2-inch per 10 ft (6 mm per m) (horizontal and vertical directions)
- Wrapped face walls and slopes (e.g., welded wire or geosynthetic facing): 2-inch per 10 ft (15 mm per m) (horizontal and vertical directions)
- Wrapped face walls and slopes (e.g., welded wire or geosynthetic facing) overall vertical: 1-inch per 10 ft (8 mm per m)
- Wrapped face walls and slopes (e.g., welded wire or geosynthetic facing) bulging: 1 to 2 inches (25 to 50 mm) maximum
- Reinforcement placement elevations: 1-inch (25 mm) of connection elevation

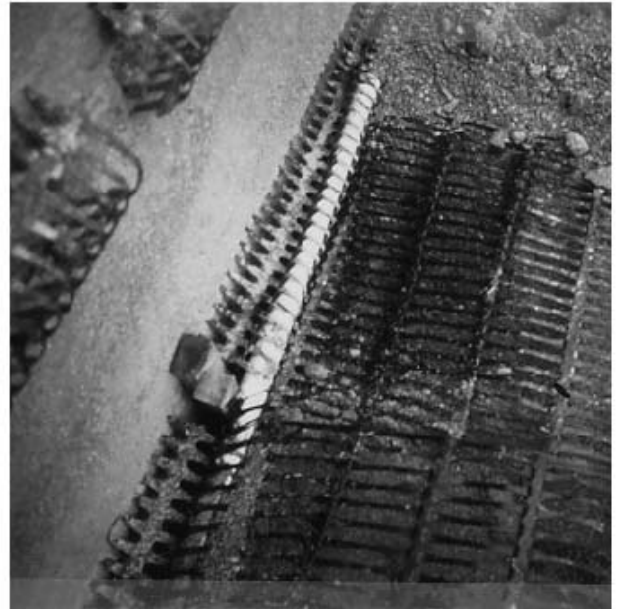
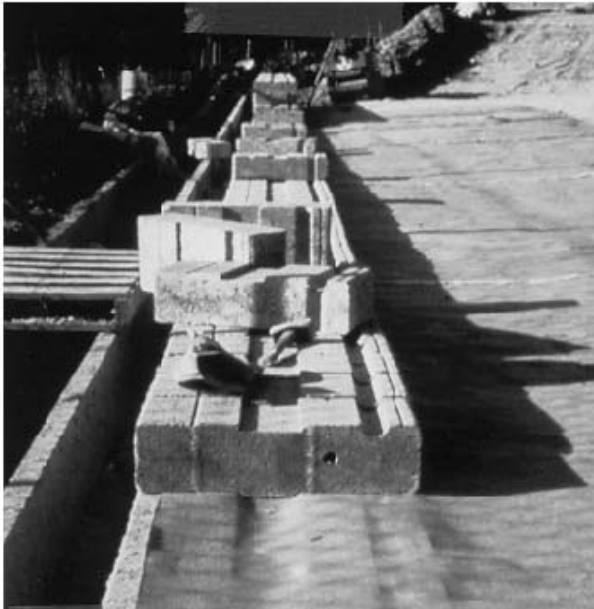


Figure 11-9. Facing connection examples.

Failure to attain these tolerances when following suggested construction practices indicates that changes in the contractor's procedures are necessary. These might include changes in reinforced backfill placement and compaction techniques, construction equipment, and facing panel batter.

Facing elements that are out of alignment should not be pushed or pulled into place because this may damage the panels and reinforcements and, hence, weaken the system. Appropriate measures to correct an alignment problem are the removal of reinforced fill and reinforcing elements, followed by the resetting of the panels. Decisions to reject structure sections that are out of alignment should be made expediently because panel resetting and reinforced fill handling are time consuming and expensive. "Post erection" deformations may be an indication of foundation, drainage (i.e., if after a heavy rain), or retained soil problems and should be evaluated immediately by qualified geotechnical specialists.

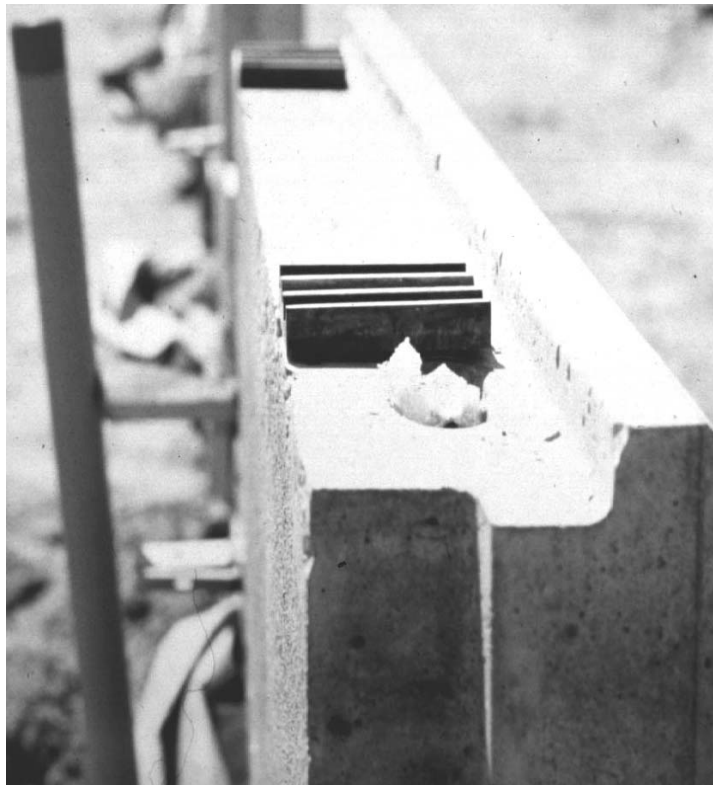
All material suppliers use bearing pads (HDPE, EPDM, PVC or neoprene are typically used) on horizontal joints between segmental facing panels to keep the panel joints open. The thickness of the bearing pads is based on the amount of anticipated short term and long term settlement. Pads that are too thin could result in cracking and spalling of panels due to point stresses and excessively large panel joint openings may result in an unattractive end product. Filter materials (usually geotextiles) are used to prevent erosion of fill through the facing joints while allowing water to pass. These materials should be installed in strict accordance with the plans and specifications, especially with regard to type of material, thickness of bearing pads, opening characteristics of geosynthetics, and quantity. Geotextile joint covers and bearing pads are shown on Figure 11-10.

Wooden wedges shown on Figure 11-6 placed during erection to aid in alignment should remain in place until the third layer of segmental panels are set, at which time the bottom layer of wedges should be removed. Each succeeding layer of wedges should be removed as the succeeding panel layer is placed. When the wall is completed, all temporary wedges should be removed.

At the completion of each day's work, the contractor should grade the wall fill away from the face and lightly compact the surface to reduce the infiltration of surface water from precipitation. At the beginning of the next day's work, the contractor should scarify the backfill surface, especially backfills containing fines, to prevent shear planes from developing between lifts.



(a)



(b)

Figure 11-10. Joint materials: a) geotextile joint cover, and b) EPDM bearing pads.

A summary of several out-of-tolerance conditions and their possible causes is presented in Table 11-4.

Table 11-4. Out-of-Tolerance Conditions and Possible Causes.

MSEW structures are to be erected in strict compliance with the structural and aesthetic requirements of the plans, specifications, and contract documents. The desired results can generally be achieved through the use of quality materials, correct construction/erection procedures, and proper inspection. However, there may be occasions when dimensional tolerances and/or aesthetic limits are exceeded. Corrective measures should quickly be taken to bring the work within acceptable limits. Presented below are several out-of-tolerance conditions and their possible causes.

<u>CONDITION</u>		<u>POSSIBLE CAUSE</u>
1. Distress in wall:	1. a.	Foundation (subgrade) material too soft or wet for proper bearing.
a. Differential settlement or low spot in wall. (Cause 1. a & b apply)	b.	Fill material of poor quality or not properly compacted.
b. Overall wall leaning beyond vertical alignment tolerance. (Cause 1 a&b)	c.	Inadequate spacing in horizontal and vertical joints
c. Spalling, chipping, or cracking of facing units (Cause 1 a – e apply) (e.g., from panel to panel contact or differential movement of modular block facing units).	d.	Use of improper bearing pads
	e.	Stones or concrete pieces between facing units (e.g. units not clean or used to level face units)
2. First panel course difficult (impossible) to set and/or maintain level.	2. a.	Leveling pad not level.
3. Wall out of vertical alignment tolerance (plumbness), or leaning out.	3. a.	Panel not battered sufficiently.
	b.	Oversized compaction equipment working within 3 ft (1 m) of wall facing panels.
	c.	Backfill material placed wet of optimum moisture content. Backfill contains excessive fine materials (beyond the specifications for percent of materials passing a No. 200 sieve).
	d.	Backfill material pushed against back of facing panel before being placed

		and compacted above reinforcing elements.
	e.	Excessive compaction of uniform, medium-fine sand (more than 60 percent passing a No. 40 sieve).
	f.	Backfill material dumped close to free end of reinforcing elements, then spread toward wall face, causing displacement of reinforcements and pushing panel out.
	g.	Shoulder wedges not seated securely.
	h.	Shoulder clamps not tight.
	i.	Slack in reinforcement to facing connections.
	j.	Inconsistent tensioning of geosynthetic reinforcement to facing.
	k.	Localized over-compaction adjacent to MBW unit.
4. Wall out of vertical alignment tolerance (plumbness) or leaning in.	4. a.	Excessive batter set in panels or offset in modular block units for select granular backfill material being used.
	b.	Inadequate compaction of backfill.
	c.	Possible bearing capacity failure.
5. Wall out of horizontal alignment tolerance, or bulging.	5. a.	See <u>Causes</u> 3c, 3d, 3e, 3j, 3k. Backfill saturated by heavy rain or improper grading of backfill after each day's operations.
6. Panels do not fit properly in their intended locations.	6. a.	Panels are not level. Differential settlement (see <u>Cause</u> 1).
	b.	Panel cast beyond tolerances.
7. Large variations in movement of adjacent panels.	7. a.	Backfill material not uniform.
	b.	Backfill compaction not uniform.
	c.	Inconsistent setting of facing panels

Table 11-5. Checklist for Construction. (after FHWA NHI-08-094/095)

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			<u>1.0 DOCUMENTS AND PLANS</u>
			1.1 Has the Contractor furnished a copy of the installation plans or instructions from the MSEW or RSS supplier as required by the Specifications?
			1.2 Have the installation plans or instructions been approved by the Designer and/or Construction Division Manager?
			1.3 Have stockpile and staging areas been discussed and approved?
			1.4 Have access routes and temporary haul roads been discussed and approved?
			<u>2.0 LAYOUT</u>
			2.1 Has the contractor staked out sufficient horizontal and vertical control points, including points required for stepped foundations?
			2.2 Has the contractor accounted for wall batter when staking the base of the wall?
			2.3 Have drainage features and all utilities been located and marked?
			2.4 Have Erosion & Sedimentation Controls been installed?
			<u>3.0 FOUNDATION PREPARATION</u>
			3.1 Has the MSEW or RSS foundation area been excavated to the proper elevation?
			3.2 Has the foundation subgrade been inspected (e.g., proof rolled) as required by the specifications?
			3.3 Has all soft or loose material been compacted or unsuitable materials (e.g., wet soil, organics) been removed and replaced?
			3.4 Has the leveling-pad (if applicable) area been properly excavated and set to the proper vertical and horizontal alignment?
			3.5 Has the leveling pad (if applicable) cured for the specified time (typically at least 12 hours) before the Contractor sets any facing panels?
			<u>4.0 DRAINAGE</u>
			4.1 Is the drainage being installed in the correct location?
			4.2 Are drainage aggregates being kept free of fine materials?
			4.3 Are all holes, rips and punctures in geotextiles being repaired in accordance with the specifications?
			4.4 Are composite drain materials being placed with the proper side to the seepage face?
			4.5 Do all collection and outlet pipes have a positive slope?

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			<u>5.0 FACING</u>
			5.1 Is the first row of facing panels (when applicable) properly placed? Do they have proper spacing, bracing, batter, and do they have the wood spacers installed?
			5.2 Is the Contractor using the correct facing unit (correct size, shape, color, and with the proper number of connections) for the applicable location and elevation?
			5.3 Is a geotextile filter being properly placed over joints in the facing panels?
			5.4 Are the lower tiers of facing baskets (when applicable) properly placed? Are they setback correctly to result in the designed slope angle? Are the struts spaced correctly?
			5.5 Have secondary reinforcing layers (e.g., biaxial geogrid) and vegetated matting (where applicable) been properly placed? Are they setback correctly to result in the designed slope angle?
			5.6 Is the vertical elevation and horizontal alignment being checked periodically and adjusted as needed?
			5.7 Is the contractor removing the wooden wedges as per the specifications? (Typically removed as soon as erection and backfilling the panel above the wedged panel is completed.)
			5.8 Is the spacing between individual facing units (or for RSS and wrapped face walls, overlap of reinforcement) in accordance with the specifications?
			<u>6.0 REINFORCING</u>
			6.1 Is the reinforcement being properly connected (connections tight and all of the slack in the reinforcing layers removed)
			6.2 Is the reinforcement in the proper alignment?
			6.3 Is the reinforcement the right type?
			6.4 Is the reinforcement the correct length?
			6.5 Is the reinforcement being placed at the correct spacing and location?
			6.6 Is the fill being brought up to 2" above the soil reinforcement elevation before the reinforcement is connected?
			6.7 Is construction equipment being kept from operating directly on the reinforcement (i.e., until adequate soil cover is placed over the reinforcement)?
			<u>7.0 BACKFILL</u>
			7.1 At the end of each day's operation is the Contractor grading the upper surface of reinforced and retained soil to ensure runoff of storm water away from the MSEW or RSS face or provide a positive means of controlling runoff away from the construction area?

<u>YES</u>	<u>NO</u>	<u>NA</u>	
			7.2 Where applicable, has the Contractor backfilled in front of the MSEW or RSS?
			7.3 Is the Contractor placing the reinforced soil in lifts that are thin enough to ensure good compaction, but thick enough not to damage the reinforcement?
			7.4 If the Contractor is using water to adjust the moisture of the reinforced, retained, or facing soil, does it meet the requirements set forth in the specifications?
			7.5 Is the reinforced soil being placed to prevent damage to the reinforcement?
			7.6 Are the lifts being spread to prevent excessive tension or excess slack in the reinforcement?
			7.7 Is the fill being compacted using the correct equipment and in the correct pattern?
			7.8 Is the soil moisture content within the specified range?
			7.9 Is the soil compaction (dry density) within the specified range?
			7.10 Is large compaction equipment being kept at least 3' from the face?
			<u>8.0 ANCILLARY ITEMS AND FINISHED PRODUCT</u>
			8.1 Could installation of ancillary components (e.g., catch basins, storm-water piping, guardrail) affect the reinforcing or facing components already installed?
			8.2 Have ancillary items been installed in accordance with the drawings and specifications?
			8.3 Are ancillary items being installed at the proper locations?
			8.4 Are diversion ditches, collection ditches, or slope drains installed in accordance with the drawings and specifications?
			8.5 Is permanent or temporary erosion control blanket installed at the required locations and using the details shown on the drawings?
			8.6 Are there any visible signs of MSEW or RSS tilting, bulging, or deflecting?
			8.7 Has the vertical and horizontal alignment been confirmed by survey?
			8.8 Is there a need to confirm the vertical or horizontal alignment at a future time to evaluate whether movement is occurring?
			8.9 Are there any signs of distress to the facing components (e.g., fracturing or spalling of concrete panels, bowing of wire baskets, etc)?

11.4 PERFORMANCE MONITORING PROGRAMS

Since MSE wall and RSS technologies are well established, the need for monitoring programs should be limited to cases in which new features or materials have been incorporated in the design, substantial post construction settlements are anticipated and/or construction rates require control, where degradation/corrosion rates of reinforcements are to be monitored (e.g., to allow use of marginal fills), or for asset management.

Degradation/Corrosion monitoring schemes are fully outlined in the companion *Corrosion/Degradation* document.

11.4.1 Purpose of Monitoring Program

The first step in planning a monitoring program is to define the purpose of the measurements. Every instrument on a project should be selected and placed to assist in answering a specific question.

If there is no question, there should be no instrumentation. Both the questions that need to be answered and the clear purpose of the instrumentation in answering those questions should be established. The most significant parameters of interest should be selected, with care taken to identify secondary parameters that should be measured if they may influence primary parameters.

Important parameters that may be considered include:

- Horizontal movements of the face (for MSEW structures).
- Vertical movements of the surface of the overall structure.
- Local movements or deterioration of the facing elements.
- Drainage behavior of the backfill.
- Performance of any structure supported by the reinforced soil, such as approach slabs for bridge abutments or footings.
- Horizontal movements within the overall structure.
- Vertical movements within the overall structure.
- Lateral earth pressure at the back of facing elements.
- Vertical stress distribution at the base of the structure.
- Stresses in the reinforcement, with special attention to the magnitude and location of the maximum stress.
- Stress distribution in the reinforcement due to surcharge loads.
- Relationship between settlement and stress-strain distribution.

- Stress relaxation in the reinforcement with time.
- Aging condition of reinforcement such as metal losses due to corrosion or degradation of polymeric reinforcements.
- Pore pressure response below structure.
- Temperature which often is a cause of real changes in other parameters, and also may affect instrument readings.
- Rainfall which often is a cause of real changes in other parameters.
- Barometric pressure, which may affect readings of earth pressure and pore pressure measuring instruments.

The characteristics of the subsurface, backfill material, reinforcement, and facing elements in relation to their effects on the behavior of the structure must be assessed prior to developing the instrumentation program. It should be remembered that foundation settlement will affect stress distribution within the structure. Also, the stiffness of the reinforcement will affect the anticipated lateral stress conditions within the retained soil mass.

11.4.2 Limited Monitoring Program

Limited observations and monitoring that should be performed on practically all structures will typically include:

- Horizontal and vertical movements of the face (for MSEW structures).
- Vertical movements of the surface of the overall structure.
- Local movements or deterioration of the facing elements.
- Performance of any structure supported by the reinforced soil, such as approach slabs for bridge abutments or footings.

Horizontal and vertical movements can be monitored by surveying methods, using suitable measuring points on the retaining wall facing elements or on the pavement or surface of the retained soil. Permanent benchmarks are required for vertical control. For horizontal control, one horizontal control station should be provided at each end of the structure.

The **maximum** lateral movement of the wall face during construction is anticipated to be on the order of $H/250$ for inextensible reinforcement and $H/75$ for extensible reinforcement. Tilting due to differential lateral movement from the bottom to the top of the wall would be anticipated to be less than $\frac{1}{4}$ -inch per 5 ft (4 mm per m) of wall height for either system. Post-construction horizontal movements are anticipated to be very small. Post construction vertical movements should be estimated from foundation settlement analyses, and measurements of actual foundation settlement during and after construction should be made.

11.4.3 Comprehensive Monitoring Program

Comprehensive studies involve monitoring of surface behavior as well as internal behavior of the reinforced soil. A comprehensive program may involve the measurement of nearly all of the parameters enumerated above and the prediction of the magnitude of each parameter at working stress to establish the range of accuracy for each instrument.

Whenever measurements are made for construction control or safety purposes, or when used to support less conservative designs, a predetermination of warning levels should be made. An action plan must be established, including notification of key personnel and design alternatives so that remedial action can be discussed or implemented at any time.

A comprehensive program may involve all or some of the following key purposes:

- Deflection monitoring to establish gross structure performance and as an indicator of the location and magnitude of potential local distress to be more fully investigated.
- Structural performance monitoring to primarily establish tensile stress levels in the reinforcement and or connections. A second type of structural performance monitoring would measure or establish degradation rates of the reinforcements.
- Pullout resistance proof testing to establish the level of pullout resistance within a reinforced mass as a function of depth and elongation.

The possible instruments for monitoring are outlined in Table 11-6.

11.4.4 Program Implementation

Selection of instrument locations involves three steps. First, sections containing unique design features are identified. For example, sections with surcharge or sections with the highest stress. Appropriate instrumentation is located at these sections. Second, a selection is made of cross sections where predicted behavior is considered representative of behavior as a whole. These cross sections are then regarded as primary instrumented sections, and instruments are located to provide comprehensive performance data. There should be at least two "primary instrumented sections." Third, because the selection of representative zones may not be representative of all points in the structure, simple instrumentation should be installed at a number of "secondary instrumented sections" to serve as indices of comparative behavior. For example, surveying the face of the wall in secondary cross sections would examine whether comprehensive survey and inclinometer measurements at primary sections are representative of the behavior of the wall.

Table 11-6. Possible Instruments for Monitoring Reinforced Soil Structures.

<u>PARAMETERS</u>	<u>POSSIBLE INSTRUMENTS</u>
Horizontal movements of face	Visual observation Surveying methods Horizontal control stations Tiltmeters
Vertical movements of overall structure	Visual observation Surveying methods Benchmarks Tiltmeters
Local movements or deterioration of facing elements	Visual observation Crack gauges
Drainage behavior of backfill	Visual observation at outflow points Open standpipe piezometers
Horizontal movements within overall structure	Surveying methods Horizontal control stations Probe extensometers Fixed embankment extensometers Inclinometers Tiltmeters
Vertical movements within overall structure	Surveying methods Benchmarks Probe extensometers Horizontal inclinometers Liquid level gauges
Performance of structure supported by reinforced soil	Numerous possible instruments (depends on details of structure)
Lateral earth pressure at the back of facing elements	Earth pressure cells Strain gauges at connections Load cells at connections
Stress distribution at base of structure	Earth pressure cells
Stress in reinforcement	Resistance strain gauges Induction coil gauges Hydraulic strain gauges Vibrating wire strain gauges Multiple telltales
Stress distribution in reinforcement due to surcharge loads	Same instruments as for stress in reinforcement

<u>PARAMETERS</u>	<u>POSSIBLE INSTRUMENTS</u>
Relationship between settlement and stress-strain distribution	Same instruments as for: • vertical movements of surface of overall structure • vertical movements within mass of overall structure • stress in reinforcement Earth pressure cells
Stress relaxation in reinforcement	Same instruments as for stress in reinforcement
Total stress within backfill and at back of reinforced wall section	Earth pressure cells
Pore pressure response below structures	Open standpipe piezometers Pneumatic piezometers Vibrating wire piezometers
Temperature	Ambient temperature record Thermocouples Thermistors Resistance temperature devices Frost gauges
Rainfall	Rainfall gauge
Barometric pressure	Barometric pressure gauge

Access to instrumentation locations and considerations for survivability during construction are also important. Locations should be selected, when possible, to provide cross checks between instrument types. For example, when multipoint extensometers (multiple telltales) are installed on reinforcement to provide indications of global (macro) strains, and strain gauges are installed to monitor local (micro) strains, strain gauges should be located midway between adjacent extensometer attachment points.

Most instruments measure conditions at a point. In most cases, however, parameters are of interest over an entire section of the structure. Therefore, a large number of measurement points may be required to evaluate such parameters as distribution of stresses in the reinforcement and stress levels below the retaining structure. For example, accurate location of the locus of the maximum stress in the reinforced soil mass will require a significant number of gauge points, usually spaced on the order of 1-foot (300 mm) apart in the critical zone. Reduction in the number of gauge points will make interpretation difficult, if not impossible, and may compromise the objectives of the program.

In preparing the installation plan, consideration should be given to the compatibility of the installation schedule and the construction schedule. If possible, the construction contractor should be consulted concerning details that might affect his operation or schedule.

Step-by-step installation procedures should be prepared well in advance of scheduled installation dates for installing all instruments. Detailed guidelines for choosing instrument types, locations and installation procedures are given in FHWA RD 89-043 (Christopher et al., 1989) and FHWA HI-98-034 (Dunncliff, 1998).

11.4.5 Data Interpretation

Monitoring programs have failed because the data generated was never used. If there is a clear sense of purpose for a monitoring program, the method of data interpretation will be guided by that sense of purpose. Without a purpose, there can be no interpretation.

When collecting data during the construction phase, communication channels between design and field personnel should remain open so that discussions can be held between design engineers who planned the monitoring program and field engineers who provide the data.

Early data interpretation steps should have already been taken, including evaluation of data, to determine reading correctness and also to detect changes requiring immediate action. The essence of subsequent data interpretation steps is to correlate the instrument readings with other factors (cause and effect relationships) and to study the deviation of the readings from the predicted behavior.

After each set of data has been interpreted, conclusions should be reported in the form of an interim monitoring report and submitted to personnel responsible for implementation of action. The report should include updated summary plots, a brief commentary that draws attention to all significant changes that have occurred in the measured parameters since the previous interim monitoring report, probable causes of these changes, and recommended action.

A final report is often prepared to document key aspects of the monitoring program and to support any remedial actions. The report also forms a valuable bank of experience and should be distributed to the owner and design consultant so that any lessons may be incorporated into subsequent designs.

REFERENCES

- AASHTO (2007). *LRFD Bridge Design Specifications*. 4th Edition, with 2008 and 2009 Interims, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO (2006). *Roadside Design Guide*, 3rd Edition, American Association of State Transportation and Highway Officials, Washington, D.C.
- AASHTO (2006). Corrugated Polyethylene Drainage Pipe - M 252, *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 26th Edition, American Association of State Transportation and Highway Officials, Washington, D.C.
- AASHTO (2006). Standard Specifications for Geotextiles - M 288, *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, 26th Edition, American Association of State Transportation and Highway Officials, Washington, D.C.
- AASHTO (2004). *LRFD Bridge Construction Specifications*, 2nd Edition, 2004, with 2006, 2007, 2008, and 2009 Interim Revisions
- AASHTO (2002). *Standard Specifications for Highway Bridges*, 17th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO (1994). *1994 Interims to 1992 Standard Specifications for Highway Bridges*, 15th Edition, American Association of State Highway and Transportation Officials, Washington, D.C.
- AASHTO (1986). *AASHTO Guide for Design of Pavement Structures*, American Association of State Highway and Transportation Officials, Washington, D.C.
- ADAMA Engineering, Inc. (2008). *ReSSA3: Reinforced Slope Stability Analysis (3RD edition)*, www.msew.com, Newark, DE,
- ADAMA Engineering, Inc. (2001). *ReSSA: Reinforced Slope Stability Analysis*, developed under contract with Federal Highway Administration, www.msew.com, Newark, DE,
- ADAMA Engineering, Inc. (2000). *Mechanically Stabilized Earth Walls: Program MSEW*, developed under contract with Federal Highway Administration, www.msew.com, Newark, DE.
- Adib M.E. (1988). "Internal Lateral Earth Pressure in Earth Walls," Doctor of Philosophy Dissertation, Department of Civil Engineering, University of California, Berkeley.

- Andrawes, K. Z., McGown, A. and Murray, R. T. (1986). "The Load-Strain-Time Behavior of Geotextiles and Geogrids," *Proceedings of the Third International Conference on Geotextiles*, Vienna, Austria, pp. 707-712.
- AGA, 2004. *Hot-Dip Galvanizing for Corrosion Protection of Steel Products*, American Galvanizing Association, Centennial, OH, 12 pp.
- Allen, T.M. and Bathurst, R.J. (2006), "Design and performance of an 11-m high block-faced geogrid wall," *Proceedings of the 8th International Conference on Geosynthetics*, Yokohama, Japan, pp. 953-956.
- Allen, T.M., Bathurst, R.J., Lee, W. F., Holtz, R.D. and Walters, D.L., (2004) "A New Method for Prediction of Loads in Steel Reinforced Walls", *ASCE Journal of Geotechnical and Geo-environmental Engineering*, Vol. 130, No. 11, pp. 1109-1120.
- Allen and Bathurst 2003
- Allen et al. 2004
- Allen et al. 2003
- Allen, T., Christopher, B.R., Elias, V.E. and DiMaggio, J. (2001) *Development of the Simplified Method for Internal Stability Design of Mechanically Stabilized Earth (MSE) Walls*, Washington State Department of Transportation Research Report WA-RD 513.1, 108 p.
- Allen et al. 1993
- Allen, T.M., Christopher, B.R. and Holtz, R.D., (1992), "Performance of a 12.6 m High Geotextile Wall in Seattle, Washington," *Geosynthetics Reinforced Soil Retaining Walls*, J.T.H. Wu Editor, A.A. Balkema, Rotterdam, pp. 81-100.
- Anderson D.G., Martin, G.R., Lam, I. And Wang, J.N. (2008). *Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments*, Project NCHRP 12-70, National Cooperative Highway Research Program, NCHRP Report 611, Transportation Research Board, National Research Council, Washington, D.C.
- Anderson, L.R., Nelson, K.J. and Sampaco, C.L. (1995). *Mechanically Stabilized Earth Walls*, Transportation Research Circular Number 444, Transportation Research Board, Washington, D.C., 17 p.
- Anderson, S.A., Alzamora, D. and DeMarco, (2008). Asset Management Systems for Retaining Walls, *Proceedings of the 2008 Colorado ASCE Geotechnical Seminar Program in Lakewood, CO*, pp 162 – 177.

ASTM (2008). *Annual Book of Standards*, Vol. 4.13, Geosynthetics, ASTM International, Philadelphia, PA, USA.

Arizona DOT 2009

Bathurst, R.J., Nernheim, A., and Allen, T.M. (2009). “Predicted loads in steel reinforced soil walls using the AASHTO Simplified Method,” *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 2, pp. 177-184.

Bathurst, R. J., Miyata, Y., Nernheim, A., and Allen, T. M., (2008a), “Refinement of K-stiffness Method for geosynthetic reinforced soil walls,” *Geosynthetics International*, Vol. 15, No. 4, pp. 269-295.

Bathurst, R.J., Allen, T.M., and Nowak, A.S., (2008b), “Calibration Concepts for Load and Resistance Factor Design (LRFD) of Reinforced Soil Walls,” *Canadian Geotechnical Journal*, Vol. 45, pp. 1377-1392.

Bathurst, R.J., Nernheim, A. and Allen, T.M. (2008c). “Comparison of measured and predicted loads using the Coherent Gravity Method for steel soil walls.” *Ground Improvement*, Vol. 161, No. 3, 113-120.

Bathurst, R.J., Walters, D., Vlachopoulos, N., Burgess P., and Allan, T.H., (2000). “Fullscale Testing of Geosynthetic Reinforced Walls,” *Proceedings of GeoDenver*, ASCE Special Publication 2000.

Berg, R.R., Christopher, B.R. and Samtani, N.C. (2009). Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, FHWA NHI-10-024 Volume I and NHI-10-025 Volume II, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 306 p (Vol I) and 380p (Vol II).

Berg, R.R., (2000). *Minnesota Department of Transportation Standard MSEW and RSS Designs, Project Report – Volume I, Summary Report*, Minnesota Department of Transportation, Oakdale, MN, 98 p.

Berg, R.R. (1993), *Guidelines for Design, Specification, and Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations*, FHWA-SA-93-025, Federal Highway Administration, Washington, D.C.

- Berg, R.R., Anderson, R.P., Race, R.J., and Chouery-Curtis, V.E. (1990). "Reinforced Soil Highway Slopes," Transportation Research Record No. 1288, Geotechnical Engineering, Transportation Research Board, Washington, D.C., pp. 99-108.
- Berg, R.R., Chouery-Curtis, V.E., and Watson, C.H., (1989). "Critical Failure Planes in Analysis of Reinforced Slopes," *Proceedings of Geosynthetics '89, Volume 1*, San Diego, February.
- Bligh et al. (2009). *Design of Roadside Barrier Systems Placed on MSE Retaining Walls-Report in Review*, Project NCHRP 22-20, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C.
- Bonaparte, R., Holtz, R.D. and Giroud, J.P., (1987). "Soil Reinforcement Design Using Geotextiles and Geogrids", *Geotextile Testing and The Design Engineer, ASTM, Special Technical Publication 952*.
- Bonczkiewicz, C., Christopher, B. R., and Atmatzidis, D. K. (1988). "Evaluation of Soil Reinforcement Interaction by Large-Scale Pullout Tests," *Transportation Research Record 1188*, pp. 1-18.
- Bush, D.I. (1990). "Variation of Long-Term Design Strength of Geosynthetics in Temperatures up to 40°C", *Proceedings of the Fourth International Conference on Geotextiles, Geomembranes, and Related Products*, The Hague, Netherlands, pp. 673-676.
- Cedergren, H.R. (1989). Seepage, Drainage, and Flow Nets, Third Edition, John Wiley and Sons, New York, 465 p.
- Chan C., Hover, K.C., Folliard, K.J., Hance R.M. and Trejo, D. (2007). *Durability of Segmental Retaining Wall Blocks: Final Report*, FHWA HRT-07-021, FHWA, McLean, VA, 271 p.
- Chow and Van Laeken (1991). "Viscoelastic Relaxation in oriented Semicrystalline Polymers," *Polymer*, Vol. 32, pp. 1798-1802.
- Christopher, B. R. (1993). *Deformation Response and Wall Stiffness in Relation to Reinforced Soil Wall Design*, Ph.D. Dissertation, Purdue University, 352 pp.
- Christopher, B. R., Gill, S. A., Giroud, J. P., Juran, I., Mitchell, J. K., Schlosser, F., and Dunncliff, J., (1990). "Design and Construction Guidelines for Reinforced Soil Structure - Volume 1." FHWA-RD-89-043, Federal Highway Administration, Washington, D. C.

- Christopher, B. R., Gill, S. A., Giroud, J. P., Juran, I., Mitchell, J. K., Schlosser, F., and Dunncliff, J. (1989). *Design and Construction Guidelines - Volume II, Summary of Research and Systems Information*, FHWA RD 89-043, FHWA, Washington, D.C.
- Collin, J.G. (1996). "Controlling Surficial Stability Problems on Reinforced Steepened Slopes" *Geotechnical Fabrics Report* IFAI.
- Collin, J. G., Berg, R.R., and Meyers, M. (2002). *Segmental Retaining Wall Drainage Manual*, National Concrete Masonry Association, Herdon, Virginia, 2002, 96 p.
- Collin J.G. (1986). Earth Wall Design, Doctor of Philosophy Dissertation, University of Berkeley, 419 p.
- Daniel, D.E. and Koerner, R.M. (1993). Quality Assurance and Quality Control for Waste Containment Facilities, U.S. EPA Technical Guidance Document, EPA/600/R-93/182, Cincinnati, OH, 305 p.
- den Hoedt, G., Voskamp, W., van den Heuvel, C. J. M. (1994). "Creep and Time-to-Rupture of Polyester Geogrids at Elevated Temperatures," *Proceedings of the Fifth International Conference on Geotextiles, Geomembranes, and Related Products*, Singapore, pp. 1125-1130.
- DiMaggiio, J. (1994). *Design and Construction Monitoring of Mechanically Stabilized Earth Structures*, FHWA.
- Dunncliff, J. (1998). Geotechnical Instrumentation, U.S. Department of Transportation, Federal Highway Administration, Washington DC, FHWA-HI-98-034, 238 p.
- Elias, V., Fishman, K.L., Christopher, B.R. and Berg, R.R. (2009). Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, U.S. Department of Transportation, Federal Highway Administration, Washington DC, FHWA-NHI-09-087, 146 p.
- Elias, V., Welsh, J., Warren, J., Lukas, R., Collin, J.G. and Berg, R.R. (2006). *Ground Improvement Methods*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., FHWA NHI-06-019 (Vol. I) and FHWA NHI-06-020 (Vol. II), 2006.
- Elias, V, Christopher, B.R. and Berg, R.R. (2001). *Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines*, U.S. Department of Transportation, Federal Highway Administration, Washington DC, FHWA-NHI-00-043, 418 p.
- Elias, V. (2000). Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, *FHWA-NHI-00-044*, Washington, D.C., 94p.

- Elias, V., Salman, A., Juran, I., Pearce, E., Lu, S., (1999). "Testing Protocols for Oxidation and Hydrolysis of Geosynthetics," *FHWA RD-97-144*, Washington, D.C.
- Elias, V. and Christopher, B.R., (1997). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines, Federal Highway Administration, FHWA SA-96-071, Washington, D.C., 371 p.
- Elias V. (1989). "Corrosion/Durability of Soil Reinforced Structures" *FHWA RD 89-186*. Washington D.C., 105 p.
- Elias, V., Zehong, Y., Swan, R.H., and Bachus, R.C., (1998). *Testing Protocols for Confined Creep and Extension Testing of Geosynthetics*, FHWA-RD-97-143, FHWA, Washington, D.C., 211 p.
- Embacher, R., Schultz, A. and Snyder, M. (2001a). Condition and Durability of Segmental Concrete Block Retaining Walls Along Roadways in Minnesota, Minnesota Department of Transportation, 348 p.
- Embacher, R., Schultz, A. and Snyder, M. (2001b).
- Eurocode 7 Geotechnical Design, European Committee for Standardization
- Ferry, J. D. (1980). *Viscoelastic Properties of Polymers*, 3rd Edition, John Wiley and Sons, NY.
- FHWA Federal Lands Highway Division's Roadway Aesthetic Treatments Photo Album (RATPA) available at <http://gallery.company39.com/FLH/gallery/>.
- Findley, W. N., Lai, J. S., Onaran, K. (1976). *Creep and Relaxation of Nonlinear Viscoelastic Materials*, Dover Publications, NY, 369 pp.
- Gray, D.H. and Sotir, R. (1995). *Biotechnical and Soil Bioengineering Slope Stabilization, A Practical Guide for Erosion Control*, John Wiley & Sons, New York, NY.
- Hayden, R.F., Schmertmann, G.R., Qedan, B.Q., and McGuire, M.S. (1991) "High Clay Embankment Over Cannon Creek Constructed with Geogrid Reinforcement," Proceedings of Geosynthetics '91, Atlanta, GA.
- Helwany, B., Wu, J. T. H., 1992, "A Generalized Creep Model for Geosynthetics," *International Symposium on Earth Reinforcement Practice*, Japan, pp.
- Holtz, R. D., Christopher, B. R., and Berg, R. R. (2008). Geosynthetic Design and Construction Guidelines, Federal Highway Administration, Report FHWA-NHI-07-092, McLean, VA.

Huntington, W.C. (1957). *Earth Pressures and Retaining Walls*, John Wiley and Sons, New York, pp. 175-217.

ISO/FDIS 9080 (2001)

Jewell, R.A. (1990). "Revised Design Charts for Steep Reinforced Slopes, Reinforced Embankments: Theory and Practice in the British Isles", *Thomas Telford*, London, U.K.

Jewell, R.A. and Greenwood, J.H. (1988). "Long-Term Strength and Safety in Steep Soil Slopes Reinforced by Polymer Materials", *Geotextiles and Geomembranes*, Vol. 7, Nos. 1 and 2, pp. 81-118.

Jurgenson, L. (1934). "The Shearing Resistance of Soils", *Journal of the Boston Society of Civil Engineers*, Also in *Contribution to Soil Mechanics, 1925-1940*, Boston Society of Civil Engineers, pp 134-217.

Kavazanjian, E. (2009). Personal communication.

Kenney, T.C. and Lau, D. (1986). Reply (to discussions), Vol. 23, No. 3, pp. 420-423, *Internal Stability of Granular Filters*, Canadian Geotechnical Journal, Vol. 22, No. 2, 1985, pp. 215-225.

Kenney, T.C. and Lau, D. (1985). *Internal Stability of Granular Filters*, Canadian Geotechnical Journal, Vol. 22, No. 2, 1985, pp. 215-225.

Koerner, G.R., Hsuan, Y.G. and Hart, M. (2000). *Field Monitoring and Laboratory Study of Geosynthetics in Reinforcement Applications, Tasks B1 and B2*, Geosynthetic Research Institute, Philadelphia, PA, 116 p.

Koerner, R.M., (1998). Designing With Geosynthetics, 4th Edition, Prentice-Hall Inc., Englewood Cliffs, NJ, 761p.

Lazarte, C. A., Elias, V., Espinoza, R. D., Sabatini, P.J. (2003). "Soil Nail Walls", *Geotechnical Engineering Circular*, No. 7, Federal Highway Administration Report, Report No. FHWA-IF-03-017, Washington, D.C.

Leshchinsky and Han (2004).

Leshchinsky, E. and Boedeker, R.H. (1989). "Geosynthetic Reinforced Soil Structures", *Journal of Geotechnical Engineering*, ASCE, Volume 115, Number 10, pp. 1459-1478.

Lostumbo, J.M. (2009). "Yeager Airport Runway Extension: Tallest Known 1H:1V Slope in U.S.," *Proceedings of Geosynthetics 2009*, Salt Lake City, Utah, pp. 593-598.

- Lothspeich, S., and Thornton, S. (2000). "Comparison of Long-term Design Reduction Factors Based on Creep Rupture and Creep Strength Criteria for Several Geosynthetic Reinforcing Materials," *EuroGeo 2*, pp. 341-346.
- Mayne, P. W., Christopher, B. R., and DeJong, J. (2002). "Manual on Subsurface Investigations", National Highway Institute Publication, Federal Highway Administration, Report FHWA-NHI-01-031, Washington, D.C.
- McGown, A., Paine, N., and DuBois, D.D. (1984). "Use of Geogrid Properties in Limit Equilibrium Analysis", *Proceedings of the Symposium on Polymer Grid Reinforcement in Civil Engineering*, Paper No. 1.4, pp. 1-5.
- Meyerhof, G.G., (1951). "The Ultimate Bearing Capacity of Foundations." *Geotechnique* Vol. 2 pp. 301-332.
- Minnesota Department of Transportation (Mn/DOT), (2008). Approved Products List for MBW Unit – Geosynthetic Reinforcement Combinations (available at <http://www.dot.state.mn.us/materials/geoteng.html>).
- Minnesota Department of Transportation (Mn/DOT), (2008). Standard Drawings.
- Minnesota Department of Transportation (Mn/DOT), (2008). Use of Dry-Cast Segmental Masonry Retaining Wall Units, Technical Memorandum No. 08-06-MRR-01, April 7, 2008, 5 p. (available at www.dot.state.mn.us/design/tech-memos/active).
- Minnesota Department of Transportation (Mn/DOT), (2001). *Technical Memorandum No.: 01-05-MR-01*, Program Support Group, Minnesota Department of Transportation, 8 February 2001, 8 p.
- Morrison, K. F., Harrison, F. E., Collin, J. G., Dodds, A., and Arndt, B. (2006). "Shored Mechanically Stabilized Earth (SMSE) Wall Systems", Technical Report, Federal Highway Administration, Washington, D.C., Report No. FHWA-CFL/TD-06-001, 212 p.
- Munfakh, G., Arman, A., Collin, J.G., Hung, J.C-J. and Brouillette, R.P. (2001). Shallow Foundations, Federal Highway Administration, FHWA-NHI-01-023, Washington, D.C., 222 p.
- Murray, R. T. and McGown, A. (1988). "Assessment of the Time Dependent Behavior of Geotextiles for Reinforced Soil Applications," *Durability of Geotextiles RILEM*, Chapman and Hall.
- Musser and Denney (2005).

- NCMA (2009). Design Manual for Segmental Retaining Walls, National Concrete Masonry Association, 3rd Edition, Herndon, VA.
- NCMA (1997). Design Manual for Segmental Retaining Walls, National Concrete Masonry Association, Collin, J.G., editor 2nd Edition, Herndon, VA, TR-127A.
- Newmark, N.M., (1965). “Effects of Earthquakes on Dams and Embankments,” *Geotechnique*, Vol. 14, No. 2, pp. 139-160.
- O’Neill, M.W. and Reese, L.C. (1999). *Drilled Shafts: Construction Procedures and Design Methods*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., FHWA-NHI-99-025.
- Popelar, et al., (1990). “Viscoelastic Material Characterization and Modeling for Polyethylene,” *Polymer Engineering and Science*, Vol. 30, no. 10, pp. 577-586.
- Popelar, C.H., Kenner, V.H., and Wooster, J.P. (1991). “An Accelerated Method for Establishing the Long Term Performance of Polyethylene Gas Pipe Materials”, *Polymer Engineering and Science*, Vol. 31, pp. 1693-1700.
- Prandtl, L. (1921). Über die eindingungsfestigkeit (hate) plastischer baustoffe und die festigkeit von schneiden, *Zeitschrift für Angewandte Mathematik und Mechanik*, Vol 1, No. 1, pp. 15-20.
- Reissner H. (1924). Zum erddruckproblem, *Proceedings of the 1st Intem. Congress of Applied Mechanics*, Delft, Germany, pp. 295-311.
- Ruegger, R. (1986). "Geotextile Reinforced Soil Structures on which Vegetation can be Established", *Proceedings of the 3rd International Conference on Geotextiles*, Vienna, Austria, Volume II, pp. 453-458.
- Sabatini, P. J., Pass, D. G. and Bachus, R. C. (1999). “Ground Anchors and Anchored Systems.” *Geotechnical Engineering Circular*, No. 4, U.S. Department of Transportation, Federal Highway Administration, FHWA-SA-99-015.
- Samtani, N. C. and Nowatzki, E. A. (2006). “*Soils and Foundations*”, Reference Manual - Volume I and II, Report No. FHWA NHI-06-088 and FHWA NHI-06-089, Federal Highway Administration, U. S. Department of Transportation.
- Sankey, J.E., Bailey, M.J., and Chen, B. (2007). “SeaTac Third Runway: Design and Performance of MSE Tall Wall,” *New Horizons in Earth Reinforcement*, 5th International Symposium on Earth Reinforcement, IS Kyushu 2007, Fukuoka, Japan.

- Schmertmann, G.R., Chouery-Curtis, V.E., Johnson, R.D. and Bonaparte, R. (1987). Design Charts for Geogrid-Reinforced Soil Slopes, *Proceedings of Geosynthetics '87*, New Orleans, LA, Volume 1, pp. 108-120.
- Schwanz, N., Meyers, M.S., and Berg, R.R. (1997). Specifying and Bidding Segmental Concrete Faced MSE Walls on Corps of Engineers Projects, *Proceedings of Geosynthetics '97 Conference*, Long Beach, California, pp. 789-801.
- Silvestri, V. (1983). "The Bearing Capacity of Dykes and Fills Founded on Soft Soils of Limited Thickness", *Canadian Geotechnical Journal*, Volume 20, Number 3, pp. 428-436.
- Simac, M.R., Bathhurst R.J. Berg R.R., Lothspeich S.E., (1993). Design Manual for Segmental Retaining Walls, National Concrete Masonry Association. Herndon, VA, 1993, 336 p.
- Sotir, R.B. and Christopher, B.R. (2000). "Soil Bioengineering and Geosynthetics for Slope Stabilization," *Proceedings, Geosynthetics Asia 2000*, Selangor Daruf, Ehsan Malaysia.
- Sotir, R.B. and Stulgis, R.P. (1999). "Slope Stabilization Using Soil Bioengineering Case Studies", *Proceedings ASCE International Water Resource Engineering Conference Program*, Seattle, WA.
- Sotir, R.B., Difini, J.T. and McKown, A.F. (1998). "Soil Bioengineering Biotechnical Stabilization of a Slope Failure," *Proceedings, The Sixth International Conference on Geosynthetics*, Atlanta, GA.
- Stuedlein, A.W., Mikkelsen, P.E. and Bailey, M.J. (2007). Instrumentation and Performance of the Third Runway North MSE Wall at Seattle-Tacoma International Airport, Field Measurements in Geomechanics 2007, GSP No. 175, ASCE, J. DiMaggio and P. Osborn (Eds.), pp 1-14.
- Takaku, A. (1981). "Effect of Drawing on Creep Fracture of Polypropylene Fibers", *Journal of Applied Polymer Science*, Vol. 26, pp. 3565-3573.
- Takemura (1959). Theoretical Consideration of the Effect of Crystallinity on the Viscoelastic Properties of Polyethylene," *Journal of Applied Polymer Science*, Vol. 38, pp. 471-478.
- Tanyu, B.F., Sabatini, P.J. and Berg, R.R. (2008). *Earth Retaining Structures*, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., FHWA-NHI-07-071, 2008.

- Terzaghi, K., Peck, R.B., and Mesri, G. (1996). Soil Mechanics in Engineering Practice, Third Edition, John Wiley & Sons, New York, pp 330-332.
- Thornton, J. S., Allen, S. R., and Thomas, R. W., (1997) “Approaches for the Prediction of Long Term Viscoelastic Properties of Geosynthetics from Short-Term Tests,” *Geosynthetics '97*, IFAI, Long Beach, CA, Vol. 1, pp. 277-291.
- Thornton, J. S., Allen, S. R., Thomas, R. W., and Sandri, D., 1998b, “The Stepped Isothermal Method for Time-Temperature Superposition and Its Application to Creep Data of Polyester Yarns,” *Sixth International Conference on Geotextiles, Geomembranes, and Related Products*, Atlanta, GA, pp.699-706.
- Thornton, J. S., and Baker, T. L., 2002, “Comparison of SIM and conventional Methods for Determining Creep-Rupture Behavior of a Polypropylene Geotextile,” 7th International Geosynthetics Conference, Nice, France, Vol. 4, pp. 1545-1550.
- Thornton, J. S., Paulson, J., and Sandri, D., (1998a), “Conventional and Stepped Isothermal Methods for Characterizing Long-Term Creep Strength of Polyester Geogrids,” *Sixth International Conference on Geotextiles, Geomembranes, and Related Products*, Atlanta, GA, pp.691-698.
- Vesic, A.S. (1975). Bearing Capacity of Shallow Foundations, *Foundation Engineering Handbook*, Winterkorn, H. and H.Y. Fang, H.Y. (Eds.), Van Nostrand Reinhold Co., New York, NY, Chapter 3, pp. 121-147.
- Wadsworth, H. (1998). *Handbook of Statistical Methods for Engineers and Scientists*, McGraw-Hill, 688 pp.
- Washington Department of Transportation (2009). Standard Practice T 925, Standard Practice for Determination of Long-Term Strength for Geosynthetic Reinforcement, WSDOT Materials Manual M46-01.03.
- Washington State Department of Transportation (WSDOT), (2006). Geotechnical Design Manual, M 46-03, Chapter 15 – Abutments, Retaining Walls, and Reinforced Slopes.
- Wayne and Wilcosky (1995). "An innovative use of a nonwoven geotextile in the repair of Pennsylvania State Route 54", *Geotechnical Fabrics Report*, Volume 13, Number 2.
- Wilding, M.A. and Ward, I.M. (1978). “Tensile and Creep Recovery in Ultra-High Modulus Polyethylenes,” *Polymer*, Vol. 19, pp.
- Wilding, M.A. and Ward, I.M. (1981). “Creep and Recovery of Ultra High Modulus Polyethylene”, *Polymer*, Vol. 22, pp. 870-876.

Wright, S. (2005)

Wright, S.G. and Duncan, J.M. (1990). "Limit Equilibrium Stability Analyses for Reinforced Slopes," presented at the *Transportation Research Board 70th Annual Meeting*, January 13-17, Washington D.C.

Wrigley et al. 2000

Wrigley et al. 1999

Wu, J.T.H, Lee, K., Helwany, S., and Ketchart, K. (2006). *Design and Construction Guidelines for Geosynthetic-Reinforced Soil Bridge Abutments with a Flexible Facing System*, NCHRP Report 556, Project 12-59, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Washington, D.C.

Wu, J.T.H. (1994). *Design and Construction of Low Cost Retaining Walls – The Next Generation in Technology*, Colorado Transportation Institute, Report CTI-UCD-1-94, 152p.

Yarger, T.L. and Barnes R.S. (1993), "Montana Department of Transportation's Introduction to Geogrid Use for Steepened Embankment Design", *Proceedings of Geosynthetics '93*, Vancouver, B.C., Canada, pp 311-326.

Zhurko 1965

Zornberg J., Barrows, R., Christopher, B.R. and Wayne, M. (1995). "Construction and Instrumentation of a Highway Slope Reinforced with High-Strength Geotextiles", *Proceedings of Geosynthetics '95*, Nashville, TN.

APPENDIX A
LRFD LOAD NOTATION,
LOAD COMBINATIONS, AND LOAD FACTORS

A.1 LOAD NOTATION

From AASHTO 3.3.2, the following notation is used for permanent and transient loads and forces.

Permanent Loads

CR	=	Force effects due to creep
DD	=	Downdrag force
DC	=	Dead load of structural components and nonstructural attachments
DW	=	Dead load of wearing surfaces and utilities
EH	=	Horizontal earth loads
EL	=	Miscellaneous locked-in force effects resulting from the construction process, including jacking apart cantilevers in segmental construction
ES	=	Earth surcharge load
EV	=	Vertical pressure from dead load of earth fill
PS	=	Secondary forces from post-tensioning
SH	=	Force effects due shrinkage

Transient Loads

BR	=	Vehicular braking force
CE	=	Vehicular centrifugal force
CT	=	Vehicular collision force
CV	=	Vessel collision force
EQ	=	Earthquake load
FR	=	Friction load
IC	=	Ice load
IM	=	Vehicular dynamic load allowance
LL	=	Vehicular live load
LS	=	Live load surcharge
PL	=	Pedestrian live load
SE	=	Force effect due to settlement
TG	=	Force effect due to temperature gradient
TU	=	Force effect due to uniform temperature
WA	=	Water load and stream pressure
WL	=	Wind on live load
WS	=	Wind load on structure

A.2 LOAD COMBINATIONS

Load combinations and load factors from AASHTO 3.4, Table 3.4.1-1 are listed below.

Load Combinations and Load Factors (Table 3.4.1-1, AASHTO, 2007)

Load Combination	DC DD DW EH EV ES EL PS CR SH	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
Limit State													
STRENGTH I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
STRENGTH IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—
STRENGTH V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—
EXTREME EVENT I	γ_p	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—
EXTREME EVENT II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00
SERVICE I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—
SERVICE III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—
SERVICE IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—
FATIGUE – LL, IM & CE ONLY	—	0.75	—	—	—	—	—	—	—	—	—	—	—

A.3 LOAD FACTORS FOR PERMANENT LOADS

Load factors for permanent loads, from AASHTO 3.4, Table 3.4.1-2 are listed below.

Load Factors for Permanent Loads, γ_p (Table 3.4.1-2, AASHTO, 2007)

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
<i>DC</i> : Component and Attachments		1.25	0.90
<i>DC</i> : Strength IV only		1.50	0.90
<i>DD</i> : Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
<i>DW</i> : Wearing Surfaces and Utilities		1.50	0.65
<i>EH</i> : Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• <i>AEP</i> for anchored walls		1.35	N/A
<i>EL</i> : Locked-in Construction Stresses		1.00	1.00
<i>EV</i> : Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures other than Metal Box Culverts		1.95	0.90
• Flexible Metal Box Culverts		1.50	0.90
<i>ES</i> : Earth Surcharge		1.50	0.75

APPENDIX B

DETERMINATION OF PULLOUT RESISTANCE FACTORS and MBW UNIT – GEOSYNTHETIC CONNECTION STRENGTH

Pullout resistance of soil reinforcement is defined by the ultimate pullout resistance required to cause outward sliding of the reinforcement through the soil. Reinforcement specific data has been developed and is presented in Chapter 3. The empirical data uses different interaction parameters, and it is therefore difficult to compare the pullout performance of different reinforcements.

The method for determining reinforcement pullout presented herein, consists of the normalized approach recommended in the FHWA manual FHWA-RD-89-043 (Christopher et al., 1990). The pullout resistance, F^* is a function of both frictional and passive resistance, depending on the specific reinforcement type. The scale effect correction factor, α , is a function of the nonlinearity in the pullout load - mobilized reinforcement length relationship observed in pullout tests. Inextensible reinforcements usually have little, if any nonlinearity in this relationship, resulting in α equal to 1.0, whereas extensible reinforcements can exhibit substantial nonlinearity due to a decreasing shear displacement over the length of the reinforcement, resulting in an α of less than 1.0.

Both F^* and α must be determined through product specific tests, or empirically/theoretically using the procedures provided herein and in Section 3.3, in particular Table 5. It should be noted that the empirical procedures provided in this appendix for the determination of F^* reduce, for the most part, to the equations currently provided in 2007 AASHTO for pullout design.

The pullout resistance of partial/full friction facing/reinforcement connections is defined as the load required to cause sliding of the reinforcement relative to the facing blocks or reinforcement rupture at the facing connection, whichever occurs first.

B.1 EMPIRICAL PROCEDURES TO DETERMINE F^* AND α

Pullout resistance can be estimated empirically/theoretically using the method provided in Chapter 3. F^* using this method, is calculated as follows:

$$\begin{aligned} F^* &= \text{Frictional Resistance} + \text{Passive Resistance} \\ &= \tan \rho + F_q \alpha \beta \end{aligned}$$

where $\tan \rho$ is an apparent friction coefficient for the specific reinforcement, ρ is the soil-reinforcement interface friction angle, F_q is the embedment (or surcharge) bearing capacity factor, and α_β is a structural geometric factor for passive resistance. The determination of each of these parameters is provided in Table 5, Chapter 3, with α estimated analytically using direct shear test data and the "t-z" method used in the design of friction piles. However, since some test data is required and the analytical method is complex, it is better to obtain α directly from pullout test data or use conservative default values for α . If pullout test data is not available, a default value of 1.0 can be used for α for inextensible reinforcements and a default value of 0.6 to 0.8 can be used for extensible reinforcements.

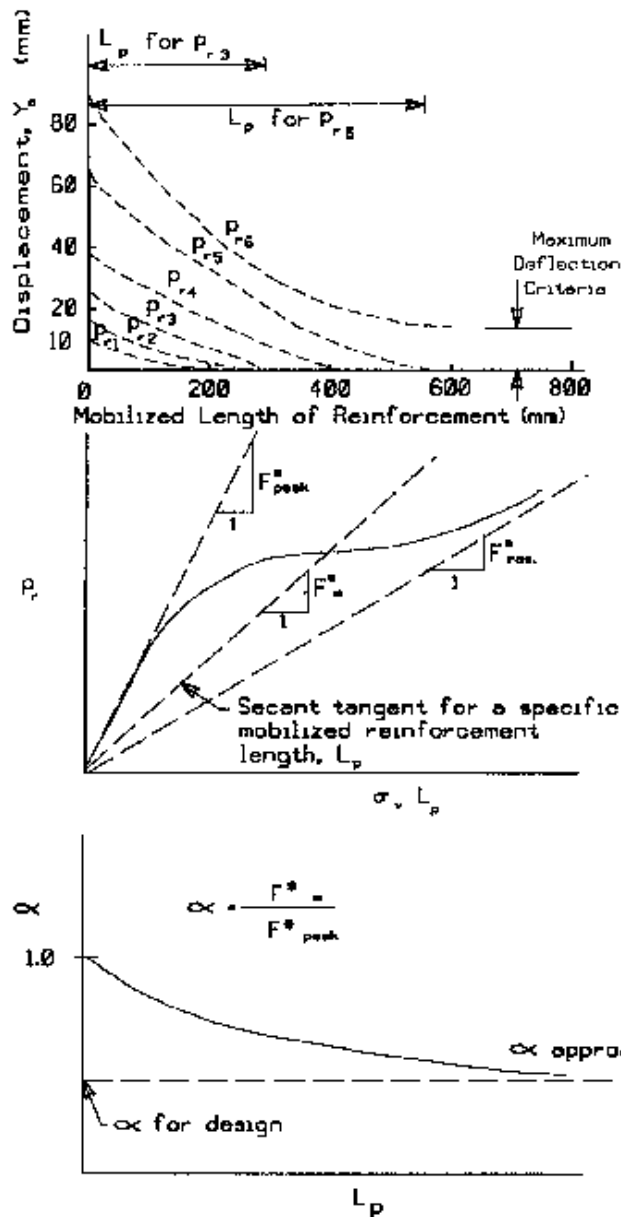
B.2 EXPERIMENTAL PROCEDURES TO DETERMINE F^* AND α

Two types of tests are used to obtain pullout resistance parameters: the direct shear test, and the pullout test. The direct shear test is useful for obtaining the peak or residual interface friction angle between the soil and the reinforcement material. ASTM D-5321 should be used for this purpose. In this case, F^* would be equal to $\tan \rho_{\text{peak}}$. F^* can be obtained directly from this test for sheet and strip type reinforcements. However, the value for α must be assumed or analytically derived, as α cannot be determined directly from direct shear tests. A pullout test can also be used to obtain pullout parameters for these types of soil reinforcement. A pullout test must be used to obtain pullout parameters for bar mat and grid type reinforcements, and to obtain values for α for all types of reinforcements. In general, the pullout test is preferred over the direct shear test for obtaining pullout parameters for all soil reinforcement types. It is recommended that ASTM D6706 test procedure using the controlled strain rate method be used. For long-term interaction coefficients, the constant stress (creep) method can be used. For extensible reinforcements, it is recommended that specimen deformation be measured at several locations along the length of the specimen (e.g., three to four points) in addition to the deformation at the front of the specimen. For all reinforcement materials, it is recommended that the specimen tested for pullout have a minimum embedded length of 24 inches (600 mm). Additional guidance is provided herein regarding interpretation of pullout test results.

For geogrids, the grid joint, or junction strength, must be adequate to allow the passive resistance on the transverse ribs to develop without failure of the grid joint throughout the design life of the structure. To account for this, F^* for geogrids should be determined using one of the following approaches:

- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706) and creep testing (per ASTM D5262) of the geogrid with clamping and loading through-the-junction.
- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706), but with the geogrid transverse ribs severed.
- Using quick effective stress pullout tests (i.e., constant rate of displacement method for short-term loading condition per ASTM D6706) if the summation of the shear strengths of the joints occurring in a 1 ft. (300 mm) length of grid sample is equal to or greater than the ultimate strength of the grid element to which they are attached. If this joint strength criteria is used, grid joint shear strength should be measured in accordance with GRI:GG2 (GRI, 1988).
- Conduct long-term effective stress pullout tests of the entire geogrid structure in accordance with the constant stress (creep) method of ASTM D6706.

For pullout tests, a normalized pullout versus mobilized reinforcement length curve should be established as shown in Figure B-1. Different mobilized lengths can be obtained by instrumenting the reinforcement specimen. Strain or deformation measuring devices such as wire extensometers attached to the reinforcement surface at various points back from the grips should be used for this purpose. A section of the reinforcement is considered to be mobilized when the deformation measuring device indicates movement at its end. Note that the displacement versus mobilized length plot (uppermost plot in figure) represents a single confining pressure. Tests must be run at several confining pressures to develop the P_r versus $\sigma_v L_p$ plot (middle plot in figure). The value of P_r selected at each confining pressure to be plotted versus $\sigma_v L_p$ is the lessor of either the maximum value of P_r (i.e., maximum sustainable load), the load which causes rupture of the specimen, or the value of P_r obtained at a predefined maximum deflection measured at either the front or the back of the specimen. Note that P_r is measured in terms of load per unit reinforcement width.



P_R = applied pullout load per unit width of reinforcement

L_p = mobilized length of reinforcement

Y_o = displacement of reinforcement

σ_v = vertical Stress on the reinforcement

F^*_{peak} = peak pullout resistance factor

F^*_a = ave. pullout resistance factor

$F^*_{res.}$ = residual pullout resistance factor

F^* = $\tan \rho$ if frictional resistance controls pullout capacity (sheet and strip reinforcement, and grids if $S_t \leq S_{opt.}$)

F^* = $F_R \alpha_g$ if passive resistance controls pullout capacity (grids if $S_t \geq S_{opt.}$)

F_R = embedment bearing capacity factor

α_g = structural geometric factor for passive resistance

Figure B-1 Experimental procedure to determine F^* and α for soil reinforcement using pullout test.

It is recommended that for inextensible reinforcements, a maximum deflection of $\frac{3}{4}$ -inch (20 mm) measured at the front of the specimen be used to select P_r if the maximum value for P_r or rupture of the specimen does not occur first. For extensible reinforcements, it is recommended that a maximum deflection of 15 mm ($\frac{5}{8}$ -inch) measured at the back of the specimen be used to select P_r if the maximum value for P_r or rupture of the specimen does not occur first. Note that it is acceptable, as an alternative, to define P_r for inextensible reinforcements based on a maximum deflection of $\frac{5}{8}$ -inch (15 mm) measured at the back of the specimen as is recommended for extensible reinforcements.

F_{peak}^* and F_m^* are determined from the pullout data as shown in Figure B-1. The method provided in this figure is known as the corrected area method (Bonczkiewicz et. al., 1988). The determination of α is also illustrated in Figure B-1. Typical values of F^* and α for various types of reinforcements are provided by Christopher (1993).

Note that the conceptualized curves provided in Figure B-1 represent a relatively extensible material. For inextensible materials, the deflection at the front of the specimen will be nearly equal to the deflection at the back of the specimen, making the curves in the uppermost plot in the figure nearly horizontal. Therefore, whether the deflection criteria to determine P_r for inextensible reinforcements is applied at the front of the specimen or at the back of the specimen makes little difference. For extensible materials, the deflection at the front of the specimen can be considerably greater than the deflection at the back of the specimen. The goal of the deflection criteria is to establish when pullout occurs, not to establish some arbitrary serviceability criteria. For extensible materials, the pullout test does not model well the reinforcement deflections which occur in full scale structures. Therefore, just because relatively large deflections occur at the front of an extensible reinforcement material in a pullout test when applying the deflection criteria to the back of the specimen does not mean that unacceptable deflections will occur in the full scale structure.

B.3 CONNECTION RESISTANCE AND STRENGTH OF PARTIAL AND FULL FRICTION SEGMENTAL BLOCK/REINFORCEMENT FACING CONNECTIONS

For reinforcement connected to the facing through embedment between facing elements using a partial or full friction connection (e.g., segmental concrete block faced walls), the connection strength can be determined directly through long-term testing of the connection to failure. The test set up should be in general accordance with ASTM D6638 with the modifications as described in the interim Long-Term Connection Strength Testing Protocol described below. Extrapolation of test data should be conducted in general accordance with

Appendix D. Tests should be conducted at a confining stress that is greater than or equal to the highest confining stress considered for the wall system, and as necessary at additional confining stresses below that level to determine behavior for the full range of confining stresses anticipated.

Regardless of the mode of failure extrapolation of the time to failure envelope must be determined. Once the failure envelope has been determined, a direct comparison between the short-term ultimate strength of the connection and the creep rupture envelope for the geosynthetic reinforcement in isolation can be accomplished to determine RF_{CR} . The connection strength obtained from the failure envelope must also be reduced by the durability reduction factor RF_D . This reduction factor should be based on the durability of the reinforcement or the connector, whichever is failing in the test.

If it is determined that the connectors failed during the connection test and not the geosynthetic, the durability of the connector, not the geosynthetic, should be used to determine the reduction factors for the long-term connection strength in this case. If the connectors between blocks are intended to be used for maintaining block alignment during wall construction and are not intended for long-term connection shear capacity, the alignment connectors should be removed before assessing the connection capacity for the selected block-geosynthetic combination. If the pins or other connection devices are to be relied upon for long-term capacity, the durability of the connector material must be established.

The connection strength reduction factor resulting from long term testing, CR_{cr} , is evaluated as follows:

$$CR_{cr} = \frac{T_{cre}}{T_{lot}} \quad (B-1)$$

where T_{cre} is the extrapolated (75 - 100 year) connection test strength and T_{lot} is the ultimate wide width tensile strength (ASTM D4595) for the reinforcement material lot used for connection strength testing.

The connection strength reduction factor resulting from quick tests, CR_{ult} , is evaluated as follows:

$$CR_{ult} = \frac{T_{ultconn}}{T_{lot}} \quad (B-2)$$

where $T_{ultconn}$ is the peak connection load at each normal load.

Testing Protocol

Objective: Determine the sustained load capacity of the connection between a modular block wall (MBW) facing element and a geosynthetic reinforcing material.

Method: Construct a test apparatus of full-scale MBW units and geosynthetic reinforcing material in a laboratory. Perform a series of tests at different normal loads (confining pressures) to model different wall heights, varying the applied load from 95 percent of the peak connection capacity determined from the quick connection test (SRWU-1) to 50 percent of the peak connection capacity. Measure and record the deflections and time to pullout or rupture of the connection.

Procedure:

1. Determine index properties of the geosynthetic reinforcing roll being tested:

- a. Wide width tensile strength (ASTM D4595)

Note: it is preferable to perform the D4595 test on the roll sample being tested and to perform the test in the same apparatus being used for the long-term connection testing. This will help remove uncertainty in the test results from using different lots of the geosynthetic reinforcement material and from comparing test results from different test equipment.

- b. Creep rupture envelope for geosynthetic: develop a rupture envelope for the specific geosynthetic being tested based on creep rupture tests, Appendix D, using the same longitudinal strip of reinforcement.

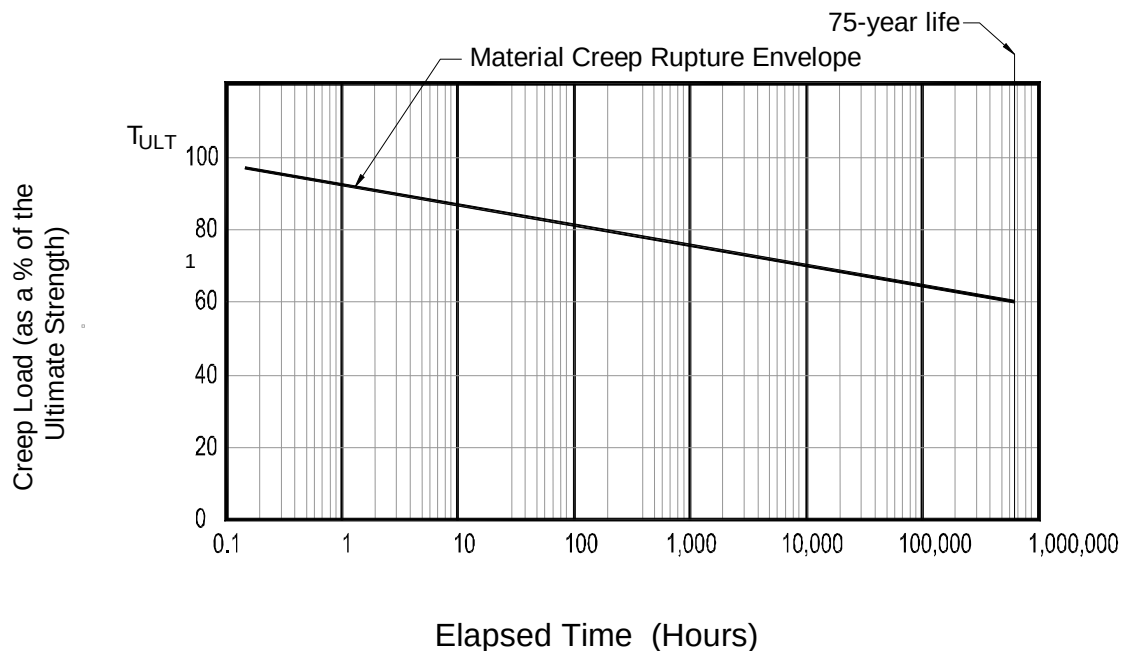


Figure B-2

Creep Rupture Envelope for Geosynthetic Reinforcement

2. Determine short-term (quick test) connection properties of the MBW unit/geosynthetic reinforcement combination, per ASTM D6638, as modified below.
 - a. Construct a test setup in general accordance with the ASTM D6638 test method with the following revisions:
 - i. Testing shall be carried out on a single width block specimen. Setup shall consist of two MBW units at the base with one MBW unit centered over the two base units.
 - ii. Geosynthetic reinforcement width shall be as close as possible to the length of the MBW unit (for geogrids this is dependent on the transverse aperture). In no case shall the geosynthetic be wider than the length of the MBW unit.
 - iii. Geosynthetic specimen shall have sufficient length to cover the interface surface as specified by the user. The specimen must be trimmed to provide sufficient anchorage at the geosynthetic loading clamp and a free length between the back of the MBW units and loading clamp ranging from a minimum of 8 in. to a maximum of 24 in. (203 to 610 mm). The same free length used for the short-term test shall be used for the long-term test. The same longitudinal strip of reinforcement shall be used for all short-term and long-term connection tests.
 - iv. The temperature in the test space, especially close to the gage length of the specimen shall be maintained within $\pm 2^{\circ}\text{C}$ ($\pm 4^{\circ}\text{F}$) of the targeted value.
 - v. Where granular infill is required in the connection, half units may be used to provide confinement for the granular fill on each side of the single top unit. Granular fill may or may not be used in the short-term and long-term test as desired. Whichever condition (with or without infill) is selected for the short-term tests shall be the same for the long-term tests. Where granular infill is not required as part of the connection, the single unit may be used.
 - vi. Normal load shall be applied to the top of the MBW unit to provide the desired confining pressure by a mechanism capable of maintaining the desired load for a period of not less than one year. *(It has been observed that under rapid loading some blocks may rotate and short-term instantaneous high normal loads can result if the vertical loading system does not have the mechanical compliance necessary to dilate. Tests shall be run for a period of 1,000 hours, however the apparatus should be capable of sustaining loads for longer periods if determined later during the test.)*
 - vii. Tension loads shall be applied to the reinforcing member in a direction parallel to the connection interface, and in the plane of the connection interface. *(The mechanism for applying the tensile loads shall be capable of sustaining an applied load for periods of not less than one-year.)*

- b. Perform a series of quick tests in accordance with ASTM D6638, as modified above, on the MBW unit/ geosynthetic reinforcement combination at different normal loads to establish the $T_{ultconn}/\text{Normal Load}$ connection curve.
3. Determine the normal and tensile load levels for sustained load testing on the MBW unit/geosynthetic reinforcement combination.
 - a. The highest normal load for the sustained load test may not exceed point A (Figure B-3) when the $T_{ultconn}/\text{Normal load}$ curve is bilinear or multilinear or point B (Figure B-3) when the slope of the curve is linear. $T_{ultconn}$ is defined as the ultimate connection strength determined from ASTM D6638. Additional normal loads may be evaluated to determine the long-term connection strength as a function of normal load.
 - b. From the connection strength verses displacement curve (Figure B-4) for the quick test, using the normal load determined in step A, determine the applied tensions loads for a range of percentages of the Peak Connection Capacity (e.g., 95, 90, 85, 80, 75, 66 and 50 percent of Peak Connection capacity). The tensile loads should be selected to define the connection rupture curve for 1000 hours.
4. Perform sustained load testing on the MBW unit/geosynthetic reinforcement combination at the normal and tensile load levels determined from step 3 using the same test apparatus used to determine the short-term connection properties. A different test apparatus may be used to perform the long-term tests as long as a correlation is made between the two test machines. Unless otherwise agreed upon, a minimum of four normal load levels shall be used to develop the connection rupture curve.
 - a. Assemble the MBW unit/geosynthetic reinforcement test as done in step 3, and apply the normal load desired to the top MBW unit.
 - b. Apply the full load (e.g., 95, 90, 85, and 80 percent of $T_{ultconn}$) tensile load rapidly and smoothly to the specimen, preferably at a strain rate of $10 \pm 3\%/min$. Record the total time for loading.
 - c. Measure the extension/deflection of the connection, at the back of the MBW unit in accordance with the following approximate time schedule: 1, 2, 6, 10, 30 min, and 1, 2, 5, 10, 30, 100, 200, 500 and 1,000 hrs (Note: shorter reading times may be required).

Record the time to failure of the connection.

 - d. Repeat steps A through C for the other normal load levels recording the loads and time to failure.
5. Presentation of data.
 - a. Plot the results of the creep rupture test on a log time plot extrapolated to a minimum of 75 years, per Appendix D. The extrapolated load is the (75 - 100 year) connection load, T_{crc}
 - b. On the same graph, plot the time to failure for the results of the sustained load tests on the reinforcement itself from Step 1.

- c. From the data plot, extrapolate to 75 years (670,000 hrs), per Appendix D.
- d. All deviations from the connection test setup from the actual connection used for construction shall be noted in the test report.

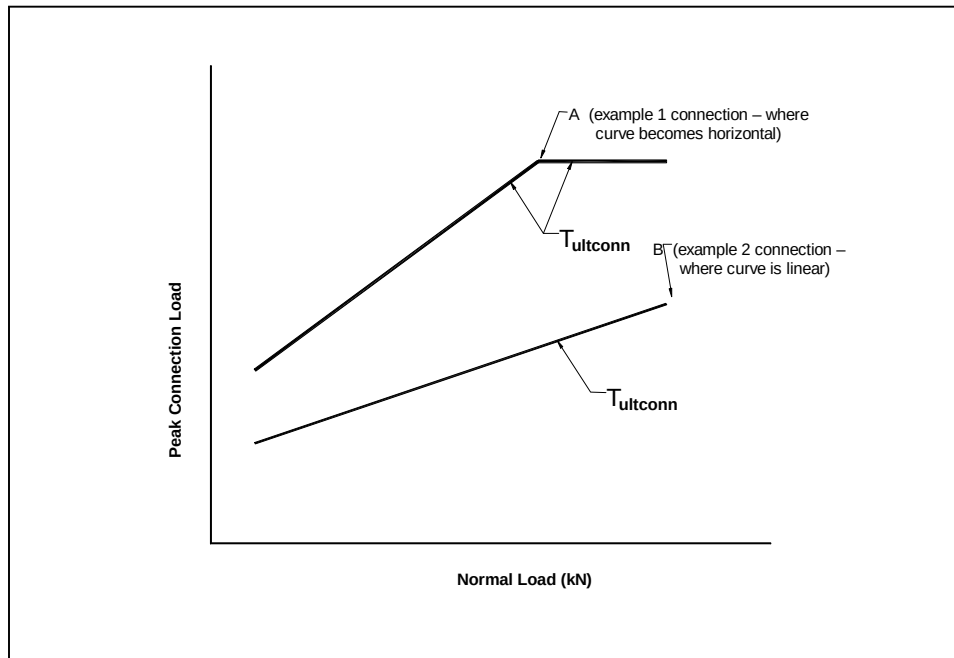


Figure B-3 Connection Strength verses Normal Load



Figure B-4 Connection Strength verses Displacement

B.4 CONNECTION RESISTANCE DEFINED WITH SHORT-TERM TESTING

B.4.1 Protocol

As discussed in Section 4.4.7.i, the long-term connection strength reduction factor to account for reduced ultimate strength resulting from the connection, CR_{cr} , may be obtained from long-term or short-term tests. as described below.

Short-term (i.e., quick) ultimate strength tests, per ASTM D6638, are used to define an ultimate connection strength, $T_{ultconn}$, at a specified confining pressure. Tests should be performed in accordance with ASTM D6638, *Determining Connection Strength Between Geosynthetic Reinforcement and Segmental Concrete Units (Modular Concrete Blocks)*. With short-term testing, CR_{cr} , is defined as follows:

$$CR_{cr} = \frac{T_{ultconn}}{RF_{cr} T_{lot}} \quad (4-43)$$

RF_{cr} is the geosynthetic creep reduction factor (see Chapter 3), and T_{lot} is the ultimate wide width tensile strength of the reinforcement material roll/lot used for the connection strength testing.

The raw data from short-term connection strength laboratory testing should not be used for design. The wall designer (and/or system supplier) should evaluate the data and define the nominal long-term connection strength, T_{alc} . Steps for this data reduction are summarized and discussed below.

- Step 1: Separate laboratory test data by failure mode – pullout and rupture
 The laboratory data is separated by observed failure mode – pullout or rupture. Note that observed *pullout* may be more of a combination of pullout and rupture versus a clearly defined pullout.

- Step 2: Replot data and develop equations for ultimate connection strength, $T_{ultconn}$
 Data should be plotted and $T_{ultconn}$ defined as a function of normal load or normal pressure. $T_{ultconn}$ is defined in one or two straight-line segments on the plot. Data points for the two different failure modes should be plotted as separate lines.

- Step 3: Evaluate data with consideration of other tests

Evaluate data with consideration of data from testing of different grade(s) of reinforcement with same MBW unit. Replot data and develop equations, as appropriate. Trends in data between different tests should be rational.

- Step 4: Determine short-term ultimate connection strength reduction factor, CR_u
 CR_u is the short-term ultimate connection strength reduction factor defined as follows:

$$CR_u = \frac{T_{ultconn}}{T_{lot}} \quad (B-3)$$

- Step 5: Determine reinforcement creep reduction factor, RF_{CR}
 The creep reduction factor for the geosynthetic reinforcement was previously defined (see Chapter 3 and Appendix D).

- Step 6: Determine long-term connection strength, CR_{cr}
 CR_{cr} , the long-term ultimate connection strength reduction factor (based upon short-term testing) is defined as follows:

$$CR_{cr} = \frac{CR_u}{RF_{CR}} \quad (B-4)$$

Note that the creep reduction factor is applied to short-term ultimate connection strength regardless of observed failure mode (i.e., rupture or pullout).

- Step 7: Determine nominal long-term connection strength, T_{alc}
 Use equation 4-41 to determine T_{alc} .
 The nominal long-term connection strength, T_{alc} developed by frictional and/or structural means, determined as follows:

$$T_{alc} = \frac{T_{ult} \times CR_{cr}}{RF_D} \quad (4-41)$$

where:

- T_{alc} = nominal long-term reinforcement/facing connection strength per unit reinforcement width at a specified confining pressure
 T_{ult} = ultimate tensile strength of the geosynthetic soil reinforcement, defined as the minimum average roll value (MARV)

RF_D = reduction factor to account for chemical and biological degradation

CR_{cr} = long-term connection strength reduction factor to account for reduced ultimate strength resulting from the connection

Step 8: Define limits of applicability of the defined T_{alc}

The nominal long-term connection strength equation defined is applicable to the test conditions utilized. The limits of the testing program normal loading should be stated with T_{alc} . Extrapolation of the connection strength equation should not be extended significantly above or below this normal loading range.

B.4.2 Example Calculation

An example problem is presented within to demonstrate the analysis of laboratory connection strength data of modular block wall (MBW) units and geosynthetic reinforcements. Data is analyzed to determine the nominal strength envelope to use in design. This example is for a large MBW unit where one grade of a geogrid was tested, and all failure occurred by rupture of the reinforcement. (Fictional MBW unit and geosynthetic manufacturer names are used within.)

Table B-1. Summary of steps for data reduction of MBW connection strength, with short-term (quick) test data.

Step	Item
1	Separate laboratory test data by failure mode – pullout and rupture
2	Replot data and develop equations for ultimate connection strength, $T_{ultconn}$
3	Evaluate data with consideration of data from testing of different grade(s) of reinforcement with same MBW unit. Replot data and develop equations, as appropriate.
4	Determine short-term ultimate connection strength reduction factor, CR_u
5	Determine reinforcement creep reduction factor, RF_{CR}
6	Determine long-term connection strength, CR_{cr}
7	Determine nominal long-term connection strength, T_{alc}
8	Define limits of applicability (i.e., limits of testing program normal loading)

Laboratory Report

The following information was provided in the two laboratory test reports (a report for each grade of geogrid used).

AA MBW Unit and Grade II, Type XX Geogrid”

Apparatus and General

Test Procedure: per ASTM D6638

Large Modular Block Unit: AA

42 in. wide (toe to heel), 18 in. high, 48 in. long

Weight $\cong$ 2,200 lbs per unit

Geogrid: Grade II, Type XX

$T_{ult-MARV} = 4,300$ lb/ft (reported by manufacturer)

Lot and roll numbers provided

$T_{LOT} = 4,730$ lb/ft

Connection Test Results: Tensile loads at peak capacity

Tensile loads at $\frac{3}{4}$ -inch displacement

7 normal loads

Two tests at one normal load

Data – see table below

Recommended design curve and equation presented

Connection Test Notes: All tests ended in geogrid rupture after large deformation.

Evidence of slippage of the geogrid within the MBW unit-geogrid interface in all tests.

Amount of slippage diminished with increased normal loading.

The actual design capacity envelope could be lower than presented if the quality of construction in the field is less than that adopted in this controlled laboratory investigation.

Test Number	Normal Load (lb/ft)	Approximate Wall Height (ft)	Tensile Capacity (lb/ft) at 3/4-inch Displacement	Peak Tensile Capacity (lb/ft)
1	816	2.4	557	1073
2	1665	4.9	659	1431
3	2498	7.3	620	1259
4	2509	7.3	669	1445
5	2509	7.3	688	1293
6	3353	9.8	641	1390
7	4219	12.4	667	1582
8	5074	14.9	859	1637

Peak connection capacity, $T_{ultconn} = 1068 + N \tan 6^\circ$ in (lb/ft of geogrid) and N in lb/ft of wall length

Evaluate Data

Step 1. Separate data by failure mode – pullout and rupture.

All data for this MBW unit and soil reinforcement is rupture failure.

Step 2. Replot raw data and check equation for $T_{ultconn}$.

Equation provided in report checks. $T_{ultconn} = 1068 + N \tan 6^\circ$ in (lb/ft) and N in lb/ft

Step 3. Evaluate data with consideration of data from different grade of soil reinforcement.

There is no additional test data with similar products to compare this data to.

Step 4. Determine CR_u

The short-term connection strength reduction factor is:

$$CR_u = \frac{T_{ultconn}}{T_{lot}}$$

T_{lot} is the ultimate tensile strength of the material used in the connection testing. The was laboratory test report listed a $T_{lot} = 4,730$ lb/ft.

Step 5. Determine the creep reduction factor, RF_{CR}

The agency had previously evaluated the long-term (i.e., nominal) strength of the Grade II, Type XX geogrid. The agency evaluation used a creep reduction factor equal to 1.9, which was based upon an evaluation of the data supplied by the manufacturer.

Step 6. Determine CR_{cr}

Short-term connection strength testing was used. Therefore, the long-term connection strength reduction factor to account for reduced ultimate strength resulting from the connection, using Equation 4-43, is equal to:

$$CR_{cr} = \frac{T_{ultconn}}{RF_{cr} T_{lot}} = \frac{1068 + N \tan 6^\circ}{1.9 \times 1.10 (T_{ult-MARV})} = \frac{1068 + N \tan 6^\circ}{2.09 (T_{ult-MARV})}$$

Step 7. Determine T_{alc}

The nominal long-term reinforcement/facing connection strength per unit reinforcement width at a specified normal load, N , using Equation 4-41, is equal to:

$$T_{alc} = \frac{T_{ult} \times CR_{cr}}{RF_D} = \frac{T_{ult-MARV} \times \frac{1068 + N \tan 6^\circ}{2.09 (T_{ult-MARV})}}{1.15} = 444 + 0.044 N$$

where T_{alc} is in terms of lb/ft width of reinforcement and N is in terms of lb/ft width of wall facing length.

The laboratory test data and the nominal long-term connection strength lines are presented in Figure B-5.

Step 8. Define limits of applicability

The nominal long-term connection strength equation defined is applicable to the test conditions utilized. The limits of the testing program normal loading should be stated with T_{alc} . Extrapolation of the connection strength equation should not be extended significantly above or below this normal loading range.

As noted in the laboratory test report (see data table under Laboratory Report) the limits of this test program are approximate wall heights of 2.4 to 14.9 ft.

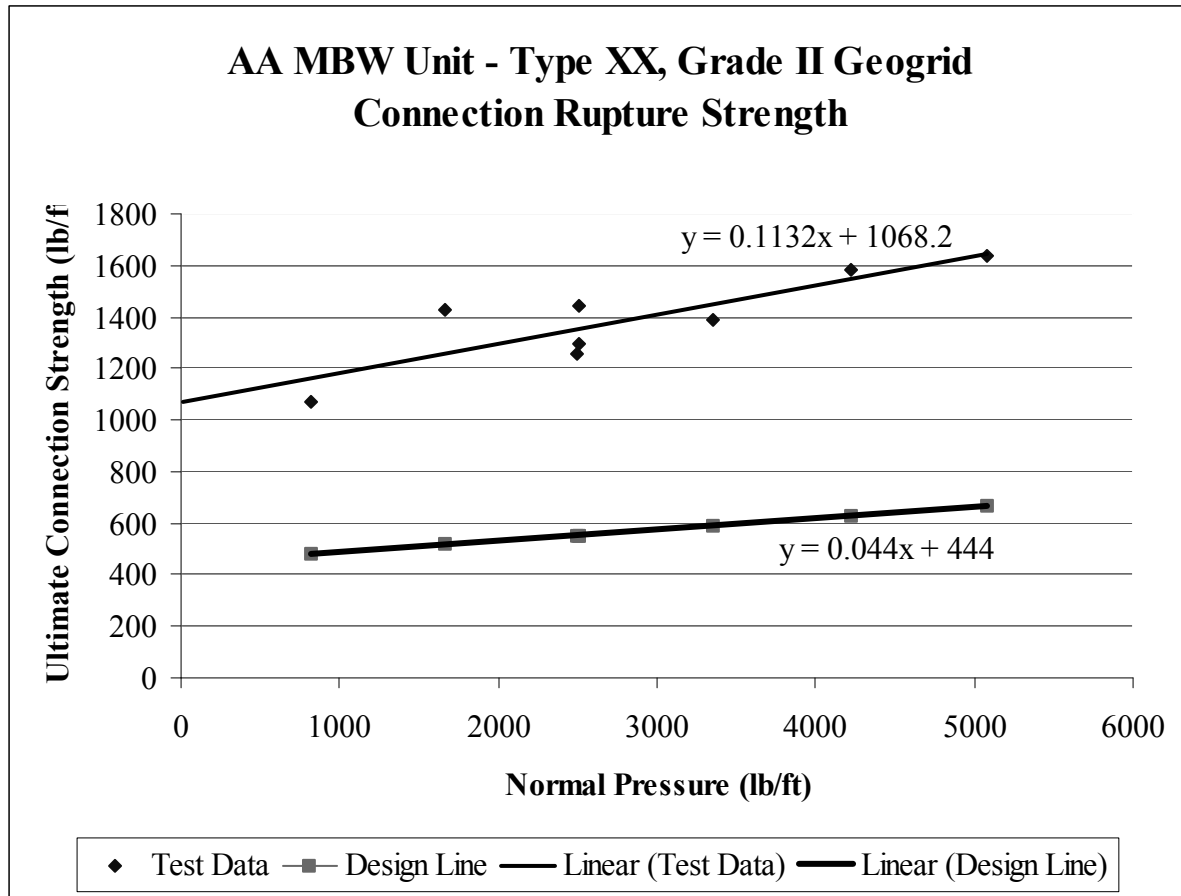


Figure B-5. Laboratory test data and the nominal long-term connection strength lines for AA MBW unit and Type XX, Grade II geogrid.

Practical Considerations

- The nominal long-term connection strength should be compared that of other soil reinforcements with this MBW unit and to other MBW units with this soil reinforcement, as a check for reasonableness.
- The laboratory test reports presented “design connection strength” lines. These were based on ultimate strength reduced by a factor of safety equal to 1.5 and a 3/4 –inch displacement criteria. These “design connection strength” lines are for a design standard that is different from AASHTO, and therefore should not be used for transportation works. Data should be evaluated in accordance with AASHTO/FHWA criteria, as detailed within this example.

APPENDIX C

STEEL SOIL REINFORCEMENTS

Linear Strip Reinforcements				
Type	Dimensions	F_y/F_u	Vertical Spacing	Horizontal Spacing
Steel Strips (ribbed)	5/32 in. thick by 2 in. wide (4 mm thick by 50 mm wide)	65/75 ksi (450/520 MPa)	30 in. (750 mm)	Varies, but typically 12 to 30in, (300 to 750 mm)

Welded Wire						
Wire Designation	Wire Area	Wire Diameter	Wire Area	Wire Diameter		
	in ²	in.	mm ²	mm		
W3.5	0.035	0.211	22.6	5.4	F _y /F _u	65/80 ksi (450/550 MPa)
W4	0.040	0.226	25.8	5.7	Longitudinal Wire Spacing	Typically 6 in. (150 mm)
W4.5*	0.045	0.239	29.0	6.0		
W5	0.050	0.252	32.3	6.4	Transverse Wire Spacing	Typically varies 9 to 24 in. (230 to 600 mm)
W7	0.070	0.298	45.2	7.6		
W9.5	0.095	0.348	61.3	8.8	Mat Spacing: For welded wire faced walls, vertically 12, 18 or 24 in. (300, 450, or 600 mm) and continuous horizontally. For precast concrete faced walls, vertically 24 to 30 in. (600 to 750 mm), horizontally 3.6 to 4 ft. (1.1 to 1.2 m) wide mats spaced at 6.2 ft (1.9 m) center-to-center or continuous	
W11	0.110	0.374	71.0	9.5		
W12	0.120	0.391	77.4	9.9		
W14	0.140	0.422	90.3	10.7		
W16	0.160	0.451	103	11.5		
W20	0.200	0.505	129	12.8		
*Typical min. size for permanent walls						

Bar Mats						
Wire Designation	Wire Area in ²	Wire Diameter in.	Wire Area mm ²	Wire Diameter mm		
W8	0.080	0.319	51.6	8.1	F_y/F_u	65/75 ksi (450/520 MPa)
W11	0.110	0.374	71.0	9.5	Longitudinal Wire Spacing	Typically 6 in. (150 mm) with 4 to 7 longitudinal bars per mat
W15	0.150	0.437	96.8	11.1	Transverse Wire Spacing	Typically varies 6 to 24 in. (150 to 600 mm)
W20	0.200	0.505	129	12.8	Mat Spacing: Typically 30 in. (750 mm) vertically and 5 ft (1.5 m) center-to-center horizontally	

Specific wall manufacturers may be able to provide a much wider range of reinforcement configurations depending on the design needs.

APPENDIX D
DETERMINATION OF CREEP STRENGTH REDUCTION FACTOR, RF_{CR} AND
DETERMINATION LONG-TERM ALLOWABLE STRENGTH, T_{al}
(after WSDOT Standard Practice T 925, Standard Practice for Determination
of Long-Term Strength for Geosynthetic Reinforcement)

D.1 BACKGROUND

The effect of long-term load/stress on geosynthetic reinforcement strength and deformation characteristics should be determined from the results of product specific, controlled, long-term laboratory creep tests conducted for a range of load levels and durations in accordance with ASTM D5262 adequate for extrapolation purposed to the desired design life, carried out to rupture when possible. Creep testing in accordance with ASTM D5262, but carried out to rupture where feasible, is described herein as the “conventional method.” A limited number of conventional creep tests may be supplemented and extended to longer creep rupture times using ASTM D6992 (Stepped Isothermal Method, or SIM) as described in this appendix. Specimens should be tested in the direction in which the load will be applied in use. Test results should be extrapolated to the required structure design life. Based on the extrapolated test results, the following is to be determined:

- For ultimate limit state design, the highest load level, designated T_l , which precludes both ductile and brittle creep rupture within the required lifetime.
- For the limit state design, creep test results should be extrapolated to the required design life and design site temperature in general accordance with the procedures outlined in this Appendix.
- In both cases, unless otherwise specified or mutually agreed upon by the geosynthetic supplier, the testing laboratory, and the owner, a baseline testing temperature of 68° F (20° C) shall be used for this testing. Higher test temperatures shall be considered as elevated temperatures to be used for the purpose of time extrapolation. ASTM D5262 requires that the testing temperature be maintained at $\pm 3.6^\circ$ F (2° C). For some polymers, this degree of variance could significantly affect the accuracy of the shift factors and extrapolations determined in accordance with this appendix. For polymers that are relatively sensitive to temperature variations, this issue should be considered when extrapolating creep data using time-temperature superposition techniques, or minimized by using a tighter temperature tolerance.

- The creep reduction factor, RF_{CR} , is determined by comparing the long-term creep strength, T_1 , to the ultimate tensile strength (ASTM D4595 for geotextiles, ASTM D6637 for geogrids) of the sample tested for creep. The sample tested for ultimate strength should be taken from the same lot, and preferably the same roll, of material that is used for the creep testing. For ultimate limit state design, the strength reduction factor to prevent long-term creep rupture is determined as follows:

$$RF_{CR} = \frac{T_{ultlot}}{T_1} \quad (D-1)$$

where, T_{ultlot} is the average lot specific ultimate tensile strength (ASTM D4595) for the lot of material used for the creep testing.

At present, creep tests are conducted in-isolation (ASTM D5262) rather than confined in-soil (e.g., FHWA RD-97-143, Elias et al., 1998), even though in-isolation creep tests tend to overpredict creep strains and underpredict the true creep strength when used in a structure. Note that the procedures provided in this appendix are for in-isolation creep rupture testing.

Considering that typical design lives for permanent MSE structures are 75 years or more, extrapolation of creep data is required. No standardized method of geosynthetic creep data modeling and extrapolation exists at present, though a number of extrapolation and creep modeling methods have been reported in the literature (Findley et al., 1976; Wilding and Ward, 1978; Wilding and Ward, 1981, Takaku, 1981; McGown et al., 1984; Andrawes et al., 1986; Murray and McGown, 1988; Bush, 1990; Popelar et al., 1991; Helwany and Wu, 1992). Many of the methods discussed in the literature are quite involved and mathematically complex. Therefore, rather than attempting to develop mathematical models which also have physical significance to characterize and extrapolate creep, as is often the case in the literature (for example, using Rate Process Theory to develop rheological models of the material), *a simplified visual/graphical approach will be taken*. This does not mean that the more complex mathematical modeling techniques cannot be used to extrapolate creep of geosynthetics; they are simply not outlined in this appendix.

The determination of T_1 can be accomplished through the use of stress rupture data. Rupture data is necessary to determine the creep reduction factor for ultimate limit state conditions.. Stress rupture test results, if properly accelerated and extrapolated can be used to investigate the effects of stress cracking and the potential for a ductile to brittle transition to occur.

Since the primary focus of creep evaluation in current practice is at rupture, *only extrapolation of stress rupture data will be explained in this appendix*. Creep strain data can

be used to estimate T_1 , provided that the creep strain data is not extrapolated beyond the estimated long-term rupture strain. The use of rupture data, and not strain data, is recommended. Therefore, no guidance is provided regarding extrapolation of creep strain data to determine T_1 .

Single ribs for geogrids, or yarns or narrow width specimens for woven geotextiles may be used for creep testing for ultimate limit state design provided that it can be shown through a limited creep testing program (conducted as described in Section D.5) that the rupture behavior and envelope for the single ribs, yarns, or narrow width specimens are the same as that for the full product, with product width as defined in ASTM D5262. This comparison must demonstrate that there is no statistical difference between the full width product creep rupture regression line and the single rib, yarn, or narrow width specimen regression line at a time of 1,000 hours using a student-t distribution at a confidence level of 0.10 (see Equation D.3-1).

Considering that typical design lives for permanent MSE structures are 75 years or more, extrapolation of creep data will be required. Current practice allows creep data to be extrapolated up to one log cycle of time beyond the available data without some form of accelerated creep testing, or possibly other corroborating evidence (Jewell and Greenwood, 1988; GRI, 1990). Based on this, unless one is prepared to obtain 7 to 10 years of creep data, temperature accelerated creep data, or possibly other corroborating evidence, must be obtained.

It is well known that temperature accelerates many chemical and physical processes in a predictable manner. In the case of creep, this means that the creep strains under a given applied load at a relatively high temperature and relatively short times will be approximately the same as the creep strains observed under the same applied load at a relatively low temperature and relatively long times. Temperature affects time to rupture at a given load in a similar manner. This means that the time to a given creep strain or to rupture measured at an elevated temperature can be made equivalent to the time expected to reach a given creep strain or to rupture at in-situ temperature through the use of a time shift factor.

The ability to accelerate creep with temperature for polyolefins such as polypropylene (PP) or high density polyethylene (HDPE) has been relatively well defined (Takaku, 1981; Bush, 1990; Popelar et al., 1991). Also for polyolefins, there is some risk that a "knee" in the stress rupture envelope due to a ductile to brittle transition could occur at some time beyond the available data (Popelar et al., 1991). Therefore, temperature accelerated creep data is strongly recommended for polyolefins. However, in practice, a ductile to brittle transition for polyolefin geosynthetic reinforcement products has so far not been observed, likely due to

the highly oriented nature of polymer resulting from the processing necessary to make fibers and ribs. In general, the degree of orientation of the polymer is an important factor regarding the potential for ductile to brittle transitions.

For polyester (PET) geosynthetics, available evidence indicates that temperature can also be used to accelerate PET creep, based on data provided by den Hoedt et al., (1994) and others. However, the creep rupture envelopes for PET geosynthetics tend to be flatter than polyolefin creep rupture envelopes, and accurate determination of time-shift factors can be difficult for PET geosynthetics because of this. This may require greater accuracy in the PET stress rupture data than would be required for polyolefin geosynthetics to perform accurate extrapolations using elevated temperature data. This should be considered if using elevated temperature data to extrapolate PET stress rupture data. Note that a "knee" in the stress rupture envelope of PET does not appear to be likely based on the available data and the molecular structure of polyester.

If elevated temperature is used to obtain accelerated creep data, it is recommended that minimum increments of 10° C be used to select temperatures for elevated temperature creep testing. The highest temperature tested, however, should be below any transitions for the polymer in question. If one uses test temperatures below 70 to 75° C for polypropylene (PP), high density polyethylene (HDPE), and PET geosynthetics, significant polymer transitions will be avoided. If higher temperatures must be used, the effect of any transitions on the creep behavior should be carefully evaluated. One should also keep in mind that at these high temperatures, significant chemical interactions with the surrounding environment are possible, necessitating that somewhat lower temperatures or appropriate environmental controls be used. These chemical interactions are likely to cause the creep test results to be conservative. Therefore, from the user's point of view, potential for chemical interactions is not detrimental to the validity of the data for predicting creep limits. However, exposure to temperatures near the upper end of these ranges could affect the stress-strain behavior of the material due to loss of molecular orientation, or possibly other effects that are not the result of chemical degradation. Therefore, care needs to be exercised when interpreting results from tests performed at temperatures near the maximum test temperature indicated above. In general, if the stiffness of the material after exposure to the environment is significantly different from that of the virgin material, the stress-strain properties, and possibly the strength, of the material may have been affected by the exposure temperature in addition to the chemical environment. If the stiffness has been affected, the cause of the stiffness change should be thoroughly investigated to determine whether or not the change in stiffness is partially or fully due to the effect of temperature, or alternatively not use the data obtained at and above the temperature where the stiffness was affected.

Unless otherwise specified or required by site-specific temperature data, an effective design temperature of 20° C (T_{amb}) should be assumed.

A number of extrapolation and creep modeling methods have been reported in the literature (Findley et al., 1976; Wilding and Ward, 1978; Wilding and Ward, 1981; Takaku, 1981; McGown et al., 1984; Andrawes et al., 1986; Murray and McGown, 1988; Bush, 1990; Popelar et al., 1991; Helwany and Wu, 1992). Many of the methods discussed in the literature are quite involved and mathematically complex.

Two creep extrapolation techniques are provided herein for creep rupture evaluation: the conventional method, which utilizes a simplified visual/graphical approach, temperature acceleration of creep, regression techniques, and statistical extrapolation, and the Stepped Isothermal Method (SIM). This does not mean that the more complex mathematical modeling techniques cannot be used to extrapolate creep of geosynthetics; they are simply not explained herein. These two techniques are described in more detail as follows:

D.2 STEP-BY-STEP PROCEDURES FOR EXTRAPOLATING STRESS RUPTURE DATA – CONVENTIONAL METHOD

Step 1: Plot the creep rupture data as log time to rupture versus log load level or versus load level, as shown in Figure D.2-1. Do this for each temperature in which creep rupture data is available. The plotting method that provides the best and most consistent fit of the data should be used. In general, 12 to 18 data points (i.e., combined from all temperature levels tested to produce the envelope for a given product, with a minimum of 4 data points at each temperature) are required to establish a rupture envelope (Jewell and Greenwood, 1988; ASTM D5262, 2007). The data points should be evenly distributed through each log cycle of time. Rupture points with a time to rupture of less than 5 hours should in general not be used, unless it can be shown that these shorter duration points are consistent with the rest of the envelope (i.e., they do not contribute to non-linearity of the envelope). As a guide:

- three of the test results should have rupture times (not shifted by temperature acceleration) of 10 to 100 hours,
- four of the test results should have rupture times between 100 and 1,000 hours, and
- four of the test results should have rupture times of 1,000 to 10,000 hours, with at least one additional test result having a rupture time of approximately 10,000 hours (1.14 years) or more.

It is recommended that creep strain be measured as well as time to rupture, since the creep strain data may assist with conventional time-temperature shifting and in identifying any change in behavior that could invalidate extrapolation of the results.

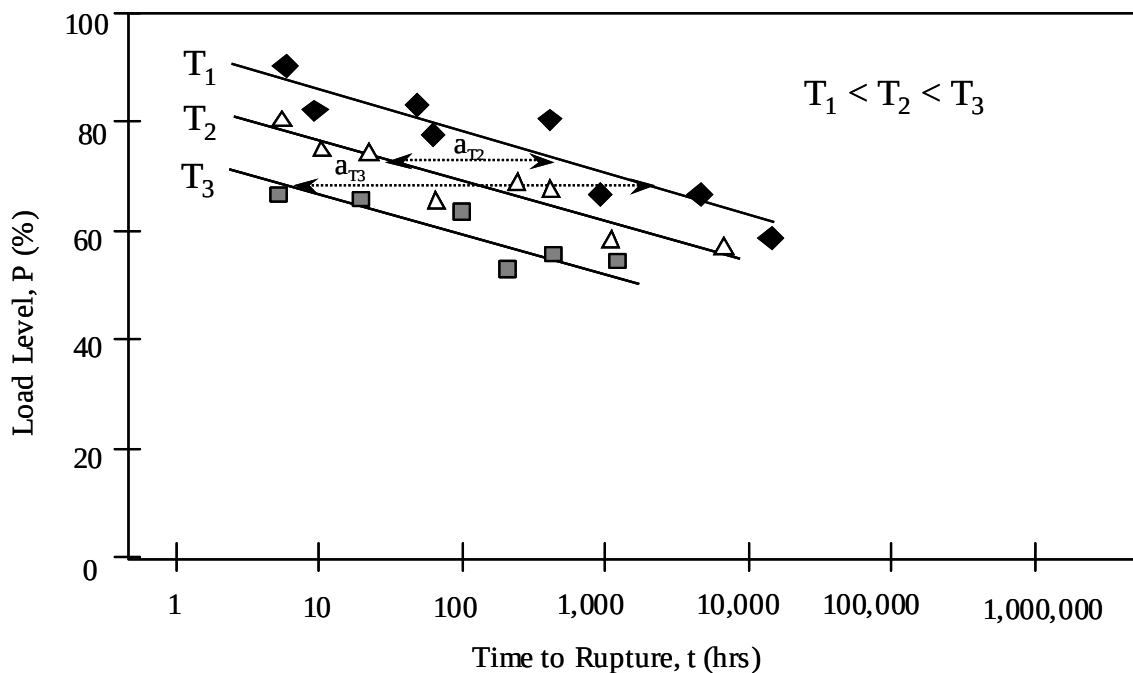


Figure D.2-1 Typical stress rupture data for geosynthetics and the determination of shift factors for time-temperature superposition.

It is acceptable to establish rupture points for times of 10,000 hours or more by assuming that specimens subjected to a given load level which have not yet ruptured to be near a state of rupture. Therefore, the time to rupture for those particular specimens would be assumed equal to the time the load has been in place. Note that this is likely to produce conservative results.

Step 2: Extrapolate the creep rupture data. Elevated temperature creep rupture data can be used to extrapolate the rupture envelope at the design temperature through the use of a time shift factor, a_T . If the rupture envelope is approximately linear as illustrated in Figure D.2-1, the single time shift factor a_T will be adequate to perform the time-temperature superposition. This time-temperature superposition procedure assumes that the creep-rupture curves at all temperatures are linear on a semi-logarithmic or double logarithmic scale and parallel. It has been found empirically that the curves for PET are semi-logarithmic and approximately parallel, or double logarithmic and approximately parallel in the case of HDPE and PP. It should be pointed out that the theory of Zhurkov (1965), which assumes that the fracture

process is activated thermally with the additional effect of applied stress, predicts that the creep-rupture characteristics should be straight when plotted on a double logarithmic diagram, and that their gradients should be stress-dependent.

Use of a single time shift factor to shift all the creep rupture data at a given temperature, termed “block shifting,” assumes that the shift factor a_T is not highly stress level dependent and that the envelopes at all temperatures are parallel, allowing an average value of a_T to be used for all of the rupture points at a given temperature. While research reported in the literature indicates that a_T may be somewhat stress level dependent and that the curves at all temperatures are not completely parallel, this assumption tends to result in a more conservative assessment of the creep reduction factor RF_{CR} (Thornton and Baker, 2002).

The time to rupture for the elevated temperature rupture data is shifted in accordance with the following equation:

$$t_{amb} = (t_{elev}) (a_T) \quad (D.2-1)$$

where, t_{amb} is the predicted time at in-situ temperature to reach rupture under the specified load, t_{elev} is the measured time at elevated temperature to reach a rupture under the specified load, and a_T is the time shift factor. a_T can be approximately estimated using a visual/graphical approach as illustrated in Figures D.2-1 and D.2-2. The preferred approach, however, is to use a computer spreadsheet optimization program to select the best shift factors for each constant temperature block of data to produce the highest R^2 value for the combined creep rupture envelope to produce the result in Figure D.2-2.

Note that incomplete tests may be included, with the test duration replacing the time to rupture, but should be listed as such in the reported results, provided that the test duration, after time shifting, is 10,000 hours or more. The rule for incomplete tests is as follows. The regression should be performed with and without the incomplete tests included. If the incomplete test results in an increase in the creep limit, keep the incomplete tests in the regression, but if not, do not include them in the regression, in both cases for incomplete tests that are 10,000 hours in duration after time shifting or more. Record the duration of the longest test which has ended in rupture, or the duration of the longest incomplete test whose duration exceeds its predicted time to failure: this duration is denoted as t_{max} .

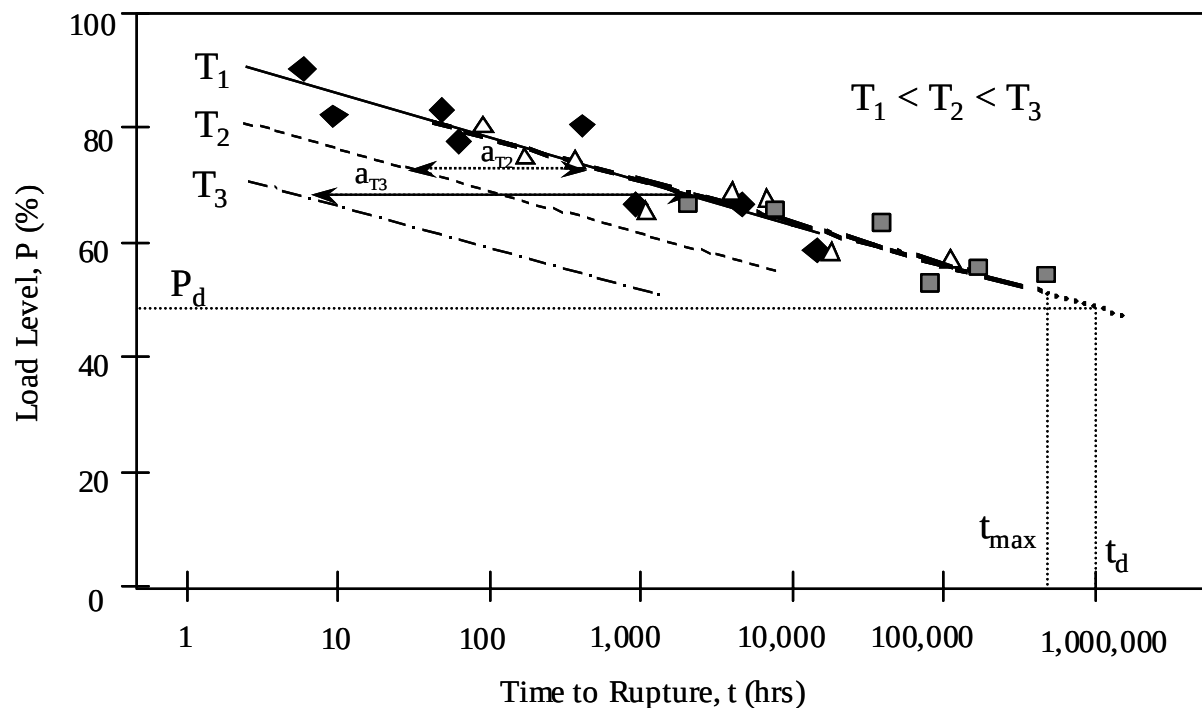


Figure D.2-2. Extrapolation of Stress Rupture Data and the Determination of the Creep Limit Load.

It is preferred that creep rupture data be extrapolated statistically beyond the elevated temperature time shifted data using regression analysis (i.e., curve fitting) up to a maximum of one log cycle of time for all geosynthetic polymers (greater extrapolation using only statistical methods is feasible, but uncertainty in the result increases substantially and must be taken into account). Therefore, adequate elevated temperature data should be obtained to limit the amount of statistical extrapolation required.

Also note that there may be situations where extrapolation to create a creep rupture envelope at a lower temperature than was tested is necessary. Situations where this may occur include the need to elevate the ambient temperature to have greater control regarding the temperature variations during the creep testing (i.e., ambient laboratory temperature may vary too much), or for sites where the effective design temperature is significantly lower than the “standard” reference temperature used for creep testing (e.g., northern or high elevation climates). In such cases, it is feasible to use lower bound shift factors based on previous creep testing experience to allow the creep rupture envelope to be shifted to the lower temperature, as shift factors for the materials typically used for geosynthetic reinforcement are reasonably consistent. Based on previous creep testing experience and data reported in the literature

(Chow and Van Laeken, 1991; Thornton et al., 1998a; Thornton, et al., 1998b; Lothspeich and Thornton, 2000; Takemura 1959; Bush, 1990; Popelar et al., 1990; Wrigley et al., 2000; Takaku 1981; Thornton and Baker, 2002), shift factors for HDPE and PP geosynthetics are typically in the range of 0.05 to 0.18 decades (i.e., log cycles of time) per 1° C increase in temperature (i.e., a 10° C increase would result in a time shift factor of 12 to 15) and 0.05 to 0.12 decades per 1° C increase in temperature for PET geosynthetics. It is recommended that if shifting the creep rupture envelope to temperatures below the available data is necessary, that a shift factor of 0.05 decades per 1° C increase in temperature for PP, HDPE, and PET be used. This default shift factor should not be used to shift the creep rupture data more than 10° C.

Step 3: Once the creep data has been extrapolated, determine the design, lot specific, creep limit load by taking the load level at the desired design life directly from the extrapolated stress rupture envelope as shown in Figure D.2-2. If statistical extrapolation beyond the time shifted stress rupture envelopes (PP or HDPE), or beyond the actual data if temperature accelerated creep data is not available, is necessary to reach the specified design life, the calculated creep load T_1 should be reduced by an extrapolation uncertainty factor as follows:

$$T_1 = \frac{P_{cl}}{(1.2)^{x-1}} \quad (D.2-2)$$

where P_{cl} is the creep limit load taken directly from the extrapolated stress rupture envelope, and "x" is the number of log cycles of time the rupture envelope must be extrapolated beyond the actual or time shifted data, and is equal to $\log t_d - \log t_{max}$ as illustrated in Figure D.2-2. The factor $(1.2)^{x-1}$ is the extrapolation uncertainty factor. If extrapolating beyond the actual or time shifted data less than one log cycle, set "x-1" equal to "0". This extrapolation uncertainty factor only applies to statistical extrapolation beyond the actual or time shifted data using regression analysis and assumes that a "knee" in the rupture envelope beyond the actual or time shifted data does not occur.

Note that a condition on the extrapolation is that there is no evidence or reason to believe that the rupture behavior will change over the desired design life. It should be checked that at long durations, and at elevated temperatures if used:

- There is no apparent change in the gradient of the creep-rupture curve
- There is no evidence of disproportionately lower strains to failure
- There is no significant change in the appearance of the fracture surface.

Any evidence of such changes, particularly in accelerated tests, should lead to the exclusion of any reading where either the gradient, strain at failure or appearance of the failure is different to those in the test with the longest failure duration. Particular attention is drawn to

the behavior of unoriented thermoplastics under sustained load, where a transition in behavior is observed in long-term creep-rupture testing (i.e., the so called “ductile to brittle transition – Popelar et al., 1991). The effect of this transition is that the gradient of the creep-rupture curve becomes steeper at the so-called “knee” such that long-term failures occur at much shorter lifetimes than would otherwise be predicted. The strain at failure is greatly reduced and the appearance of the fracture surface changes from ductile to semi-brittle. If this is observed, any extrapolation should assume that the “knee” will occur. For the method of extrapolation reference should be made to ISO/FDIS 9080 (2001), ASTM D5262 (2007), and Popelar et al., (1991).

This extrapolation uncertainty factor also assumes that the data quality is good, data scatter is reasonable, and that approximately 12 to 18 data points which are well distributed (see Step 1 for a definition of well distributed) defines the stress rupture envelope for the product. If these assumptions are not true for the data in question, this uncertainty factor should be increased. The uncertainty factor may also need to be adjusted if a method other than the one presented in detail herein is used for extrapolation. This will depend on how well that method compares to the method provided in this appendix. This extrapolation uncertainty factor should be increased to as much as $(1.4)^x$ if there is the potential for a "knee" in the stress rupture envelope to occur beyond the actual or time shift data, or if the data quality, scatter, or amount is inadequate. Furthermore, if the data quantity or over the time scale is inadequate, it may be necessary to begin applying the extrapolation uncertainty factor before the end of the time shifted data.

Note that based on experience, the R^2 value for the composite (i.e., time shifted) creep rupture envelope should be approximately 0.8 to 0.9 or higher to be confident that Equation B.2-3 will adequately address the extrapolation uncertainty. If the R^2 value is less than approximately 0.6 to 0.7, extrapolation uncertainty is likely to be unacceptably high, and additional testing and investigation should be performed. In general, such low R^2 values are typically the result of data that is too bunched up, unusually high specimen-to-specimen variability, or possibly poor testing technique.

D.3 PROCEDURES FOR EXTRAPOLATING CREEP RUPTURE DATA – STEPPED ISOTHERMAL METHOD (SIM)

An alternative creep strain/rupture analysis and extrapolation approach that has recently become available for geosynthetics is the Stepped Isothermal Method (SIM) proposed, illustrated, and investigated by Thornton et al. (1997), Thornton et al. (1998a), Thornton et al. (1998b), and Thornton and Baker (2000). SIM has been applied successfully to PET

geogrids and PP geotextiles. SIM utilizes an approach similar to the Williams-Landell-Ferry, or WLF, approach to creep extrapolation (Ferry, 1980), where master creep curves for a given material are produced from a series of short-term tests (i.e., creep test durations on the order of a few hours) on the same specimen over a wide range of temperatures (i.e., while the load on the specimen is held constant, the temperature is increased in steps). The sections of creep curve at the individual temperatures are shifted in time and combined to form a continuous prediction of the creep strain at the starting temperature.

Though the general principles of this method have been in use for many years in the polymer industry (Ferry, 1980), it has been only recently that this approach has been used for geosynthetics. Though this approach was initially developed to extrapolate creep strain data, it has been adapted to produce stress rupture data by taking the specimen to rupture once the highest test temperature is reached. In effect, through time shifting of the creep strain data obtained prior to rupture, the rupture point obtained has an equivalent shifted time that is several orders of magnitude greater than the actual test time, which could be on the order of only a few days.

The method is conducted in accordance with ASTM D6992. Key issues are the very short test time used for this method, potential use of temperatures that are well above transitions in the geosynthetic material, and its complexity. Key technical advantages of the method, however, include more accurate determination of time shift factors, since the same specimen is used at the same load level at all of the temperatures (the “conventional” method must deal with the effect of specimen to specimen variability when determining the shift factors), and that time shift factors between temperatures are determined at the same load level, eliminating the effect of load level in the determination of the shift factors (in the “conventional” method, the shift factors used are in fact an average value for a wide range of loads).

SIM can be considered for use in generating and extrapolating geosynthetic creep and creep rupture data provided this method is shown to produce results which are consistent with the “conventional” extrapolation techniques recommended in this appendix. To this end, creep-rupture testing shall be conducted using conventional tests (ASTM D5262) and SIM tests (ASTM D6992). At least six SIM rupture tests and six conventional rupture tests and shall be conducted on one of the products in the product line being evaluated. Of the six SIM rupture tests, four shall have rupture times (shifted as appropriate) between 100 and 2000 hours and two shall have rupture times greater than 2000 hours. All of the conventional creep rupture points shall be obtained at the reference temperature (i.e., not temperature shifted). Creep rupture plots shall be constructed, regression lines computed and the log times to rupture determined at a load level that corresponds to 1,000 hours and 50,000 hours

on the conventional creep rupture envelope, for the two data sets. The log time to rupture for the SIM regression at this load level shall be within the upper and lower 90% confidence limits of the mean conventional regressed rupture time at the same load level using Student's *t* test.

The following minimum creep rupture data points are recommended where conventional and SIM data points are used in combination:

- 4 conventional rupture and 4 SIM rupture data points between 100 and 2,000 hours (after shifting)
- 2 conventional and 2 SIM rupture data points between 2,000 and ~10,000 hours (after shifting), with
 - 1 conventional rupture data point at ~10,000 hours or greater with 1 SIM rupture data point at ~10,000 hours or greater (after shifting); OR
 - 2 conventional rupture data points at ~10,000 hours or greater without SIM data point at ~10,000 hours or greater (after shifting)

The confidence limit for the regression performed for the conventional creep rupture data is given by (Wadsworth, 1998):

$$\log t_L = \log t_{reg} \pm \left[t_{\alpha, n-2} \sqrt{\frac{1}{n} + \frac{(P - \bar{P})^2}{\sum (P_i - \bar{P})^2}} \right] \times \sigma \quad (D.3-1)$$

and

$$\sigma = \sqrt{\frac{\sum [\log t_i - \log \bar{t}]^2 - \frac{\left\{ \sum [(P_i - \bar{P})(\log t_i - \log \bar{t})] \right\}^2}{\sum (P_i - \bar{P})^2}}{n - 2}} \quad (D.3-2)$$

where:

- $\log t_L$ = lower and upper bound confidence limit. The + or – term in Equation B.2-1 results in the lower and upper bound confidence limits, respectively.
- t_{reg} = time corresponding to the load level from the conventional creep rupture envelope at which the comparison between the two envelopes will be made (e.g., at 1,000 and 50,000 hrs after time shifting)
- $t_{\alpha, n-2}$ = value of the *t* distribution determined from applicable Student *t* table (or from the Microsoft EXCEL function TINV($\alpha, n-2$)) at $\alpha = 0.10$ and $n-2$ degrees of freedom (this corresponds to the 90% two-sided prediction limit).
- n = the number of rupture or allowable run-out points in the original test sample (i.e., the conventional creep rupture data)

- P = load level obtained at t_{reg} from the regression line developed from the conventional creep rupture testing
- $\bar{P}$ = the mean rupture load level for the original test sample (i.e., all rupture or run-out points used in the regression to establish the conventional creep rupture envelope)
- P_i = the rupture load level of the i 'th point for the rupture points used in the regression for establishing the conventional creep rupture envelope
- $\log \bar{t}$ = the mean of the log of rupture time for the original test sample (i.e., all rupture or run-out points used in the regression to establish the conventional creep rupture envelope)
- t_i = the rupture time of the i 'th point for the rupture points used in the regression for establishing the conventional creep rupture envelope

Once $\log t_L$, both upper and lower bound, has been determined at the specified load level, compare these values to the log rupture time (i.e., $\log t_{SIM}$) obtained for the SIM creep rupture envelope test at the specified load level (e.g., 1,000 and 50,000 hours). The value of $\log t_{SIM}$ at the two specified load levels must be between the upper and lower bound confidence limits ($\log t_L$). If this requirement is not met, perform two additional SIM tests at each load level P for the specified t_{reg} where this comparison was made and develop a new SIM creep rupture envelope using all of the SIM data. If for the revised SIM regression envelope resulting from these additional tests this criterion is still not met, perform adequate additional conventional creep rupture testing to establish the complete rupture envelope for the product in accordance with this appendix).

If the criterion provided above is met, the SIM testing shall be considered to be consistent with the conventional data, and SIM may be used in combination with the conventional data to meet the requirements of Section D.2 regarding the number of rupture points and their distribution in time and maximum duration. Therefore, the combined data can be used to create the creep rupture envelope as shown in Figure D.2-2. In that figure, the SIM data shall be considered to already be time shifted. Equation D.2-3 is then used to determine T_1 .

D.4 DETERMINATION OF RF_{CR}

Step 4: The creep reduction factor, RF_{CR} , is determined by comparing the long-term creep strength, T_1 , to the ultimate tensile strength (ASTM D4595 or ASTM D6637) of the sample tested for creep (T_{lot}). The sample tested for ultimate tensile strength should be taken from the same lot, and preferably the same roll, of material that is used for the creep testing. For

ultimate limit state design, the strength reduction factor to prevent long-term creep rupture is determined as follows:

$$RF_{CR} = \frac{T_{ultlot}}{T_1} \quad (D.4-1)$$

where, T_{ultlot} is the average lot specific ultimate tensile strength (ASTM D4595 or ASTM D6637) for the lot of material used for the creep testing. Note that this creep reduction factor takes extrapolation uncertainty into account, but does not take into account variability in the strength of the material. Material strength variability is taken into account when RF_{CR} , along with RF_{ID} and RF_D , are applied to T_{ult} to determine the long-term allowable tensile strength, as T_{ult} is a minimum average roll value. The minimum average roll value is essentially the value that is two standard deviations below the average value.

D.5 USE OF CREEP DATA FROM "SIMILAR" PRODUCTS and EVALUATION OF PRODUCT LINES

Long-term creep data obtained from tests performed on older product lines, or other products within the same product line, may be applied to new product lines, or a similar product within the same product line, if one or both of the following conditions are met:

- The chemical and physical characteristics of tested products and proposed products are shown to be similar. Research data, though not necessarily developed by the product manufacturer, should be provided which shows that the minor differences between the tested and the untested products will result in equal or greater creep resistance for the untested products.
- A limited testing program is conducted on the new or similar product in question and compared with the results of the previously conducted full testing program.

For polyolefins, similarity could be judged based on molecular weight and structure of the main polymer (i.e., is the polymer branched or crosslinked, is it a homopolymer or a blend, percent crystallinity, etc.), percentage of material reprocessed, tenacity of the fibers and processing history, and polymer additives used (i.e., type and quantity of antioxidants or other additives used). For polyesters and polyamides, similarity could be judged based on molecular weight or intrinsic viscosity of the main polymer, carboxyl end group content, percent crystallinity, or other molecular structure variables, tenacity of the fibers and processing history, percentage of material reprocessed or recycled, and polymer additives

used (e.g., pigments, etc.). The untested products should also have a similar macrostructure (i.e., woven, nonwoven, extruded grid, needlepunched, yarn structure, etc.) and fiber dimensions (e.g., thickness) relative to the tested products. It should be noted that percent crystallinity is not a controlled property and there is presently no indication of what an acceptable value for percent crystallinity should be.

For creep evaluation of a similar product not part of the original product line, this limited testing program should include creep tests taken to at least 1,000 to 2,000 hours in length before time shifting if using the “conventional” creep testing approach, with adequate elevated temperature data to permit extrapolation to 50,000 hours or more. If it has been verified that SIM can be used, in accordance with Section D.3, durations after time shifting due to elevated temperature up to a minimum of 50,000 hours are required. A minimum of 4 data points per temperature level tested should be obtained to determine time shift factors and to establish the envelope for the similar product. These limited creep test results must show that the performance of the similar product is equal to or better than the performance of the product previously tested. This comparison must demonstrate that there is no statistical difference between the old product regression line and the regression line obtained for the similar product at a time of 2,000 hours (not temperature accelerated) and 50,000 hours (after time shifting) using a student-*t* distribution at a confidence level of 0.10 (see Equation D.3-1). If no statistical difference is observed, the results from the full testing program on the older or similar product could be used for the new/similar product. If this is not the case, then a full testing and evaluation program for the similar product should be conducted.

Similarly, for extension of the creep data obtained on one product in the product line (i.e., the primary product tested, which is typically a product in the middle of the range of products in the product line) to the entire product line as defined herein, a limited creep testing program must be conducted on at least two additional products in the product line. The combination of the three or more products must span the full range of the product line in terms of weight and/or strength. The limited test program described in the preceding paragraph should be applied to each additional product in the product line. The loads obtained for the data in each envelope should then be normalized by the lot specific ultimate tensile strength, T_{lot} . All three envelopes should plot on top of one another, once normalized in this manner, and the two additional product envelopes should be located within the confidence limits for the product with the more fully developed creep rupture envelope (i.e., the “primary” product) as described above for “similar” products. If this is the case, then the creep reduction factor for the product line shall be the lesser of the reduction factor obtained for the product with the fully developed rupture envelope and the envelope of all three products combined, and normalization using the ultimate tensile strength shall be considered acceptably accurate.

If this is not the case, then the creep rupture envelopes for the other two products, plus enough other products within the product line, to establish the trend in RF_{CR} as a function of product weight or ultimate tensile strength, so that the RF_{CR} for the other products within the product line can be accurately interpolated. Furthermore, T_{al} must be determined in accordance with the following:

Note that normalization using the ultimate lot specific tensile strength may not be completely accurate for some geosynthetic products regarding characterization of creep rupture behavior, and other normalization techniques may be needed (Wrigley et al., 1999). In such cases, individual creep reduction factors for each product in the product line may need to be established through fully developed creep rupture envelopes for representative products obtained at the low, middle, and high strength end of the product series. Once the creep limited strength, P_{cl} and the creep reduction factors are established for each product, in this case, product variability must still be taken into account. In such cases, T_{al} must be the lesser of the determination from Equation 1 and the following determination:

$$T_{al} = \frac{P_{95}}{RF_{ID} \times RF_D}$$

where,

P_{95} = the tensile strength determined from the 95% lower bound prediction limit for the creep rupture envelope at the specified design life (see Equations 4 and 5 in “Quality Assurance (QA) Criteria for Comparison to Initial Product Acceptance Test Results”)

D.6 CREEP EXTRAPOLATION EXAMPLES USING STRESS RUPTURE DATA

A creep extrapolation example using stress rupture data is provided. The example uses hypothetical stress rupture data, which is possible for PET geosynthetics, to illustrate the simplest extrapolation case.

D.6.1 Stress Rupture Extrapolation Example

The following example utilizes hypothetical stress rupture data for a PET geosynthetic. The data provided in this example is for illustration purposes only.

Given: A PET geosynthetic proposed for use as soil reinforcement in a geosynthetic MSE wall. A design life of 1,000,000 hours is desired. The manufacturer of the geogrid has provided stress rupture data at one temperature for use in establishing the creep limit for the material. The stress rupture data came from the same lot of material as was used for the wide

width load-strain tests. The wide width ultimate strength data for the lot is as provided in Figure D.6-1. The stress rupture data is provided in Figure D.6-2.

Find: The long-term creep strength, T_1 , at a design life of 1,000,000 hours and a design temperature of 20° C, and the design reduction factor for creep, RF_{CR} using the stress rupture data.

Solution: The step-by-step procedures provided for stress rupture data extrapolation will be followed. Step 1 has already been accomplished (Figure D.6-2).

Step 2: Extrapolate the stress rupture data. Use regression analysis to establish the best fit line through the stress rupture data. Extend the best fit line to 1,000,000 hours as shown in Figure D.6-2.

Step 3: Determine the design, lot specific, creep limit load from the stress rupture envelope provided in Figure D.6-2. The load taken directly from the rupture envelope at 1,000,000 hours is 63.4 kN/m. This value has been extrapolated 1.68 log cycles beyond the available data. Using Equation D.4,

$$T_1 = (63.4 \text{ kN/m}) / (1.2)^{1.68-1} = 56.0 \text{ kN/m}$$

Step 4: The strength reduction factor to prevent long-term creep rupture RF_{CR} is determined as follows (see Equation D.1):

$$RF_{CR} = \frac{T_{ultlot}}{T_1}$$

where, T_{ultlot} is the average lot specific ultimate tensile strength for the lot material used for creep testing. From Figure D.6-1, T_{ultlot} is 110 kN/m. Therefore,

$$RF_{CR} = (110 \text{ kN/m}) / (56.0 \text{ kN/m}) = 2.0$$

In summary, using rupture based creep extrapolation, $T_1 = 56.0 \text{ kN/m}$, and $RF_{CR} = 2.0$

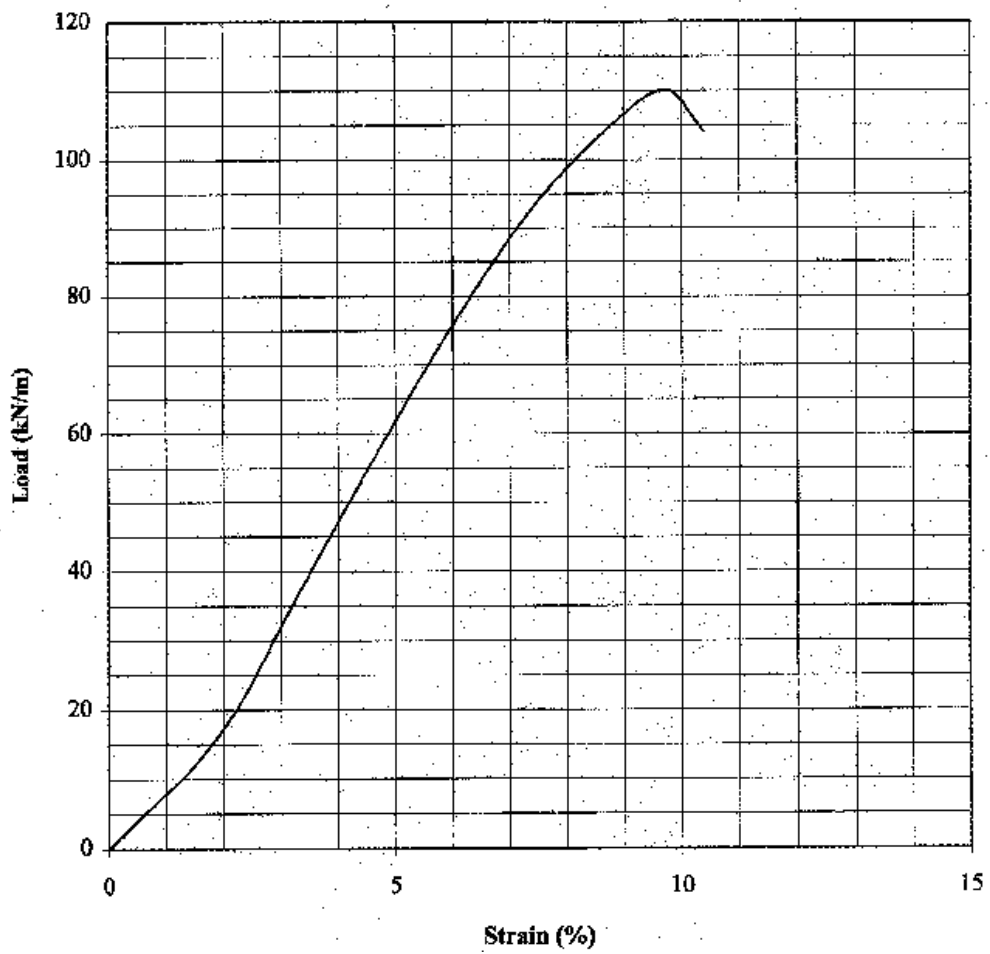


Figure D.6-1 Wide width load-strain data for PET geosynthetic at 20 C.

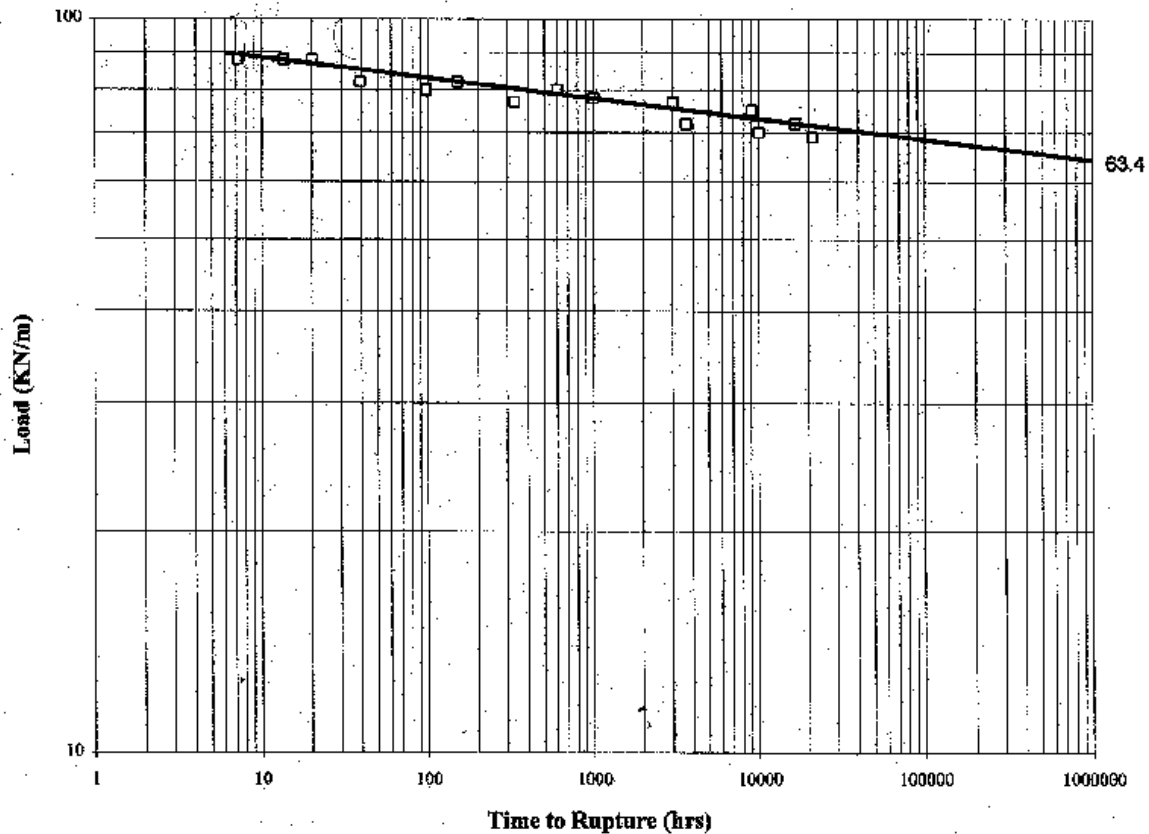


Figure D.6-2 Stress rupture data for PET geosynthetic at 20 C.

D.7 RECOMMENDED PROCEDURES TO DETERMINE T_{al}

(after WSDOT Standard Practice T 925, Standard Practice for Determination of Long-Term Strength for Geosynthetic Reinforcement)

The AASHTO LRFD Bridge Design Specifications provide minimum requirements for the assessment of T_{al} for use in the design of geosynthetic reinforced soil structures. A framework for the use of installation damage, creep, and durability test data that can be obtained from available ASTM, ISO, and GRI test standards to determine RF_{ID} , RF_{CR} , and RF_D is presented below. This protocol should be used to establish values of RF_{ID} , RF_{CR} , and RF_D that are not project or site specific, that can applied to the typical situations a given agency or owner will face. These reduction factors could then be applied to most design situations. Using this approach, a generalized step-by-step procedure to determine these reduction factors is as follows:

1. Characterize the typical environment to which the geosynthetic reinforcement will be exposed during installation and throughout its life. Key environmental parameters to be considered include the soil gradation and its angularity above and below the geosynthetic layers (RF_{ID}), likely backfill placement procedures (RF_{ID}), in-soil “average” site temperature to be used for design (RF_{CR} and RF_D), backfill pH range likely to be present (RF_D), potential exposure to sunlight, in particular UV light, and special soil conditions that may affect aging, such as summarized in Table 3-9 (RF_D). “Average” site temperature to be used for design is defined as the temperature which is halfway between the average yearly air temperature and normal daily air temperature for the highest month at the site. This site temperature definition is considered to be a conservative estimation of the average effective temperature in the soil. Note that at the connection between the soil reinforcement and the facing elements, the temperature could be significantly higher than this, especially if the facing has a southern exposure.
2. To determine RF_{CR} , conduct laboratory creep tests as described previously, using the “average” site temperature as the baseline test temperature. For those located in the northern tier of states within the USA, in most cases, it is sufficiently accurate, and a little conservative, to use a default baseline temperature of 20° C. For those located in the southern reaches of the U.S., where “average” in-soil temperatures could approach 30° C or higher, a higher baseline temperature should be used. Using the creep test results and time-temperature superposition to shift elevated temperature creep data to the baseline temperature timescale (see following section), create a creep rupture envelope for the baseline temperature, making sure that the rupture envelope extends out to the desired design life (typically 75 years). If necessary, extrapolate the envelope to the design life beyond the time shifted data using regression analysis techniques.
3. To determine RF_D , conduct the index durability tests described previously and as summarized in Table 3-11, provided that the environment to which the geosynthetic will be exposed during its life (i.e., step 1 above) is within the boundaries of conditions to which the index test results are applicable. These environment boundaries are as follows:
 - Granular soils (sands, gravels) used in the reinforced volume.
 - pH as determined by AASHTO T289 ranging from $4.5 \leq \text{pH} \leq 9$ for permanent applications and $3 \leq \text{pH} \leq 10$ for temporary applications
 - Site temperature < 85° F (30° C) for permanent applications and < 95° F (35° C) for temporary applications
 - Maximum backfill particle size of ¾-inch (19 mm), unless full scale installation damage tests conducted in accordance with ASTM D5818 are

available and indicate that RF_{ID} for the site backfill soil and geosynthetic combination is less than 1.7, and

- Soil organic content, as determined by AASHTO T267 for material finer than the 0.075 mm (No. 20) sieve ≤ 1 percent.

Site conditions not within these boundaries should be considered to be aggressive with regard to the determination of RF . If the test results meet the established criteria to consider the geosynthetic adequately durable, a default value for RF_D as specified herein may be used. If the index test results do not meet the specified criteria, or if the anticipated environment is likely to be outside the boundaries applicable to the index tests, long-term durability tests such as described by Elias et al. (2009) should be considered to determine RF_D directly.

4. To determine RF_{ID} , field expose samples of the geosynthetic to three or more different fill materials that encompass the range of soil conditions likely to be encountered. For state agencies, the selection of backfill gradations could be tied to the standard backfill materials used for reinforced walls and slopes by the agency. Once the samples exposed to installation stresses are tested to determine tensile strength loss for each backfill condition, the tensile strength loss and RF_{ID} could be plotted as a function of a key gradation parameter, such as the d_{50} size, to enable selection of RF_{ID} for the specific backfill gradation being considered.

The four step approach provided above is also applicable to specifically target the determination of these reduction factors to a specific site environment. The most common adaptation for targeting a site specific condition is to conduct installation damage tests using the actual backfill material to be used in the reinforced soil structure. The value of RF_{ID} derived from that site specific testing is then used with the values of RF_{CR} and RF_D determined as described in the above four step process. Site specific determination of RF_{CR} , primarily in consideration of a site specific baseline temperature, can also be accomplished, provided that adequate creep data is available to establish a rupture envelope for the site specific baseline temperature (assuming that site specific temperature is significantly different from the baseline temperature used for the available creep test data). If inadequate creep rupture data is available to accomplish that, it is generally cost and time prohibitive to conduct a new suite of creep tests targeted to the site specific temperature as a baseline. Also, determination of RF_D for site specific conditions is time and cost prohibitive and is rarely done, as such testing typically takes one to two years or more to complete.

Once the reduction factors are determined, then T_{al} can be determined in accordance with Equation 3-12 and used to design the geosynthetic structure (see Chapter 4).

APPENDIX E EXAMPLE CALCULATIONS

This appendix presents ten example problems that illustrate the application of the various equations and principles for design of MSE walls and slopes discussed in Chapters 2 to 8. The ten example problems were chosen to encompass a variety of geometries, soil reinforcements, and loading conditions. The first seven examples are for MSE walls, and the final three examples are for reinforced soil slopes (RSS). A summary of the example problems is included in Table E0.

Table E0
Summary of Example Problems

No.	Problem Description
<u>MSE Walls</u>	
E1	Modular Block Wall (MBW) Faced MSE wall with broken back sloping fill and live load surcharge, reinforced with geogrids
E2	Bearing check for sloping toe conditions, with and without high groundwater
E3	Segmental precast panel MSE wall with sloping backfill surcharge, reinforced with steel strips
E4	Segmental precast panel MSE wall with level backfill and live load surcharge, reinforced with steel bar mats
E5	Bridge abutment with spread footing on top of a segmental precast panel faced MSE wall with steel strips
E6	Example E4 with traffic barrier impact loading
E7	Example E4 with seismic loading
<u>RSS</u>	
E8	Road widening
E9	High slope for new road construction
E10	Facing stability calculation

MSE Wall Examples:

While using LRFD methodology care has to be taken in using the correct load factors and load combinations. There are several different ways in which the computations using LRFD can be performed. As noted in Chapter 4, for simple problem geometries the critical loading conditions can be readily identified while for complex geometries this may not be possible because the critical load effects due to various combinations of maximum and minimum loads may not be clear without performing all the intermediate computations for various load

factors. Therefore, the MSE wall example problems in this appendix have been solved in the following two formats:

Format A: In this format designs are developed based on critical load combinations that are readily identifiable based on the problem geometry. This format is used in Example E1.

Format B: In this format, the computations for load effects are first performed using maximum load factors and minimum load factors. Then, using the values computed for maximum and minimum load combinations, the critical load effects are obtained by suitably combining the maximum and minimum loads. This format is used in Examples E2, E3 and E4.

Format B involves more computations than Format A. However, in the LRFD context, Format B is essential while evaluating MSE walls with complex geometries such as those discussed in Chapter 6. This is because the critical combination of various loads may not be readily apparent until the complex system of surcharges on and within the MSE walls are analyzed with applicable maximum and minimum load factors.

Rather than introduce the more comprehensive Format B in Example E5 which addresses a case of complex geometry, a conscious attempt was made to first introduce Format B with respect to relatively simpler geometries. Thus, Example E3 and E4 have been solved with Format B. Example E3 is similar to Example E1 in the sense that they both include sloping backfill. Thus, the reader can develop a good feel for the design using both formats. Then, in Example E5 it will become evident that Format B represents a more logical way of handling complex geometries.

Format B also permits easier incorporation of extreme events such as vehicular impact and seismic events as demonstrated in Examples E6 and E7, respectively. Format B will also provide a more adaptable solution scheme in the event that load factors are modified and/or additional recommendations are developed for load combinations in future versions of AASHTO.

EXAMPLE E1

MBW UNIT FACED, MSE WALL WITH BROKEN BACKSLOPE AND LIVE LOAD SURCHARGE, REINFORCED WITH GEOGRIDS

E1-1 INTRODUCTION

This example problem demonstrates the analysis of an MSE wall with a broken backslope and live load traffic surcharge. The MSE wall is faced with modular block wall (MBW) units and has geogrid soil reinforcements. The MSE wall configuration to be analyzed is shown in Figure E.1-1.

This MSE wall is (assumed to be) a “simple” structure and, therefore, is analyzed with the load factors that typically control external stability analyses (see Figure 4-1). The design steps used in these calculations follow the basic design steps presented in Table 4-3, of which, the primary steps are presented in Table E1-1. Each of the steps and sub-steps are sequential. Therefore, if the design is revised at any step or sub-step the previous computations need to be re-examined. Each step and sub-step follow.

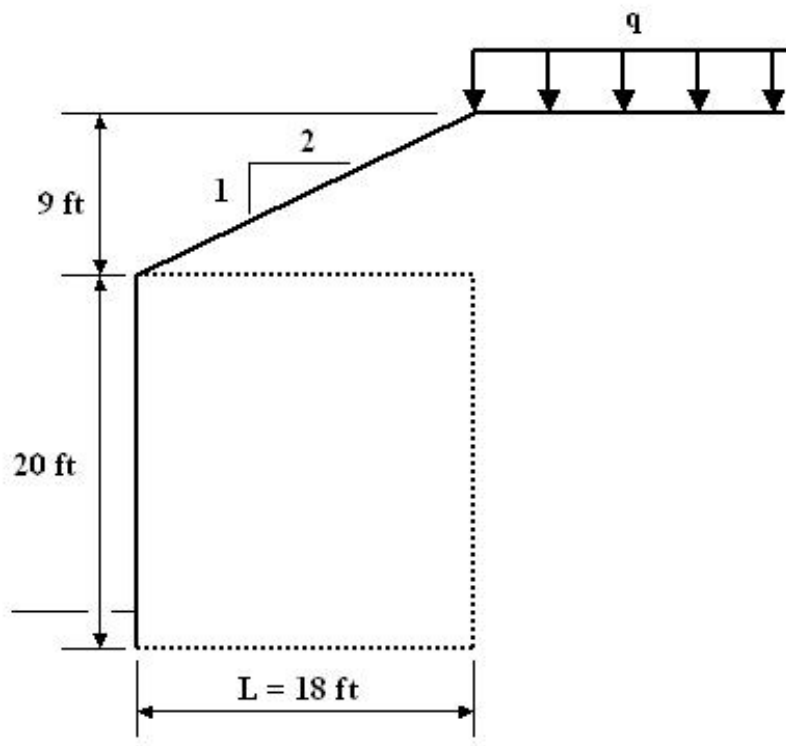


Figure E.1-1 Configuration of example problem E1.

Table E.1-1. Primary Design/Analysis Steps

Step 1.	Establish Project Requirements
Step 2.	Establish Project Parameters
Step 3.	Estimate Wall Embedment Depth, Design Height(s), and Reinforcement Length
Step 4.	Define unfactored loads
Step 5.	Summarize Load Combinations, Load Factors, and Resistance Factors
Step 6.	Evaluate External Stability
Step 7.	Evaluate Internal Stability
Step 8.	Design of Facing Elements
Step 9.	Assess Overall Global Stability
Step 10.	Assess Compound Stability
Step 11.	Design Wall Drainage Systems

Step 1. Establish Project Requirements

- Geometry
 - Exposed wall height above finished grade, $H_e = 18$ ft
 - MBW unit facing, with 3° batter
 - 2H:1V broken backslope, 9 feet high
 - Level toe slope
- Loading Conditions
 - Broken back slope
 - Traffic surcharge
 - No loads from adjacent structures
 - No seismic
 - No traffic barrier impact
- Performance Criteria
 - Design code – AASHTO/FHWA LRFD
 - Maximum tolerable differential settlement = 1/200
 - Design life = 100 years

Step 2. Establish Project Parameters

- Subsurface conditions
 - Foundation soil, $\phi'_f = 30^\circ$, $\gamma_f = 125$ pcf
 - Factored Bearing resistance of foundation soil
 - For service limit consideration, $q_{nf-ser} = 7.50$ ksf for 1-inch of total settlement
 - For strength limit consideration, $q_{nf-str} = 10.50$ ksf
 - No groundwater influence
- Reinforced wall fill, $\phi'_r = 34^\circ$, $\gamma_r = 125$ pcf, pH = 7.3, maximum size $\frac{3}{4}$ -inch
- Retained backfill, $\phi'_b = 30^\circ$, $\gamma_b = 125$ pcf

Step 3. Estimate Wall Embedment Depth and Reinforcement Length

The minimum embedment depth = $H/20$ for walls with horizontal ground in front of wall, see Table 2-1; i.e., 0.9 ft for exposed wall height of 18 ft. Therefore, use minimum embedment depth of 2.0 ft. Thus, design height of the wall, $H = 20$ ft.

Due to the 2H:1V backslope and traffic surcharge on the retained backfill, the initial length of reinforcement is assumed to be $0.9H$ or 18 ft. This length will be verified as part of the design process.

Step 4 – Define Unfactored Loads

The primary sources of external loading on an MSE wall are the earth pressure from the retained backfill behind the reinforced zone and any surcharge loadings above the reinforced zone. The 3° batter is a *near vertical* face, therefore assume a vertical face and that the MSE wall acts as a rigid body with earth pressures developed on a vertical pressure plane at the back end of the reinforcements. Estimate the earth pressures on wall for the broken backslope condition as shown in Figure 4-4 (reproduced below) and with Equations 4-2 and 4-3.

From figure:

$$H = 20.0 \text{ ft}$$

$$2H = 40.0 \text{ ft}$$

$$\text{Height of slope} = 9 \text{ ft}$$

$$\text{Therefore, angle } I = \arctan(9/40) = 12.7^\circ$$

$$\text{Slope crest is at the end of the reinforcement length, therefore, } h = 20 + 9 = 29 \text{ ft}$$

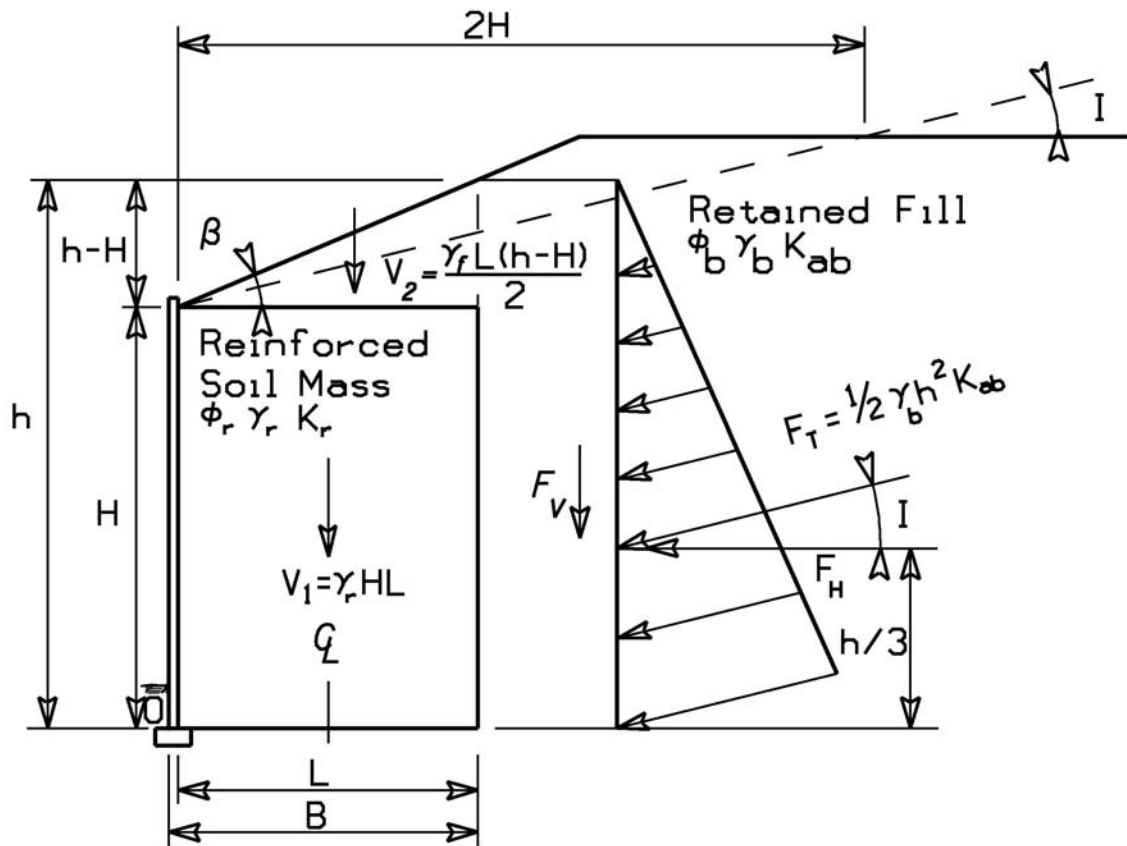


Figure 4-4. External analysis: earth pressure; broken backslope case (after AASHTO, 2007).

Using Eq. 4-3, and $\beta = I$, $\delta = I$, $\theta = 90^\circ$ for vertical and near vertical wall face, and $\phi'_b = 30^\circ$

$$\Gamma = \left[1 + \frac{\sin(\phi'_b + \delta) \sin(\phi'_b - \beta)}{\sin(\theta - \delta) \sin(\theta + \beta)} \right]^2 = \left[1 + \frac{\sin(30 + 12.7) \sin(30 - 12.7)}{\sin(90 - 12.7) \sin(90 + 12.7)} \right]^2 = 2.133$$

The external lateral pressure coefficient, K_{ab} , using Eq. 4-2, is equal to:

$$K_{ab} = \frac{\sin^2(\theta + \phi'_b)}{\Gamma \sin^2 \theta \sin(\theta - \delta)} = \frac{\sin^2(90 + 30)}{2.133 \sin^2(90) \sin(90 - 12.7)} = 0.360$$

Traffic Load

The traffic load is on the level surface of the retained backfill. For external stability, traffic load for walls parallel to traffic have an equivalent height of soil, h_{eq} equal to 2.0 ft.

Unfactored Loads:

$$F_1 = \frac{1}{2} \gamma_b h^2 K_{ab} = \frac{1}{2} (125 \text{ pcf}) (29 \text{ ft})^2 (0.360) = 18.92 \text{ k/ft}$$

$$F_{H1} = F_1 \cos I = 18.92 \text{ k/ft} (\cos 12.7^\circ) = 18.46 \text{ k/ft}$$

$$F_{V1} = F_1 \sin I = 18.92 \text{ k/ft} (\sin 12.7^\circ) = 4.16 \text{ k/ft}$$

$$q = 2.0 \text{ ft} (125 \text{ pcf}) = 250 \text{ psf}$$

$$F_2 = q h K_{ab} = 250 \text{ psf} (29 \text{ ft}) (0.360) = 2.61 \text{ k/ft}$$

$$F_{H2} = F_2 \cos I = 2.61 \text{ k/ft} (\cos 12.7^\circ) = 2.55 \text{ k/ft}$$

$$F_{V2} = F_2 \sin I = 2.61 \text{ k/ft} (\sin 12.7^\circ) = 0.57 \text{ k/ft}$$

$$V_1 = \gamma_r H L = 125 \text{ pcf} (20 \text{ ft}) (18 \text{ ft}) = 45.00 \text{ k/ft}$$

$$V_2 = \frac{1}{2} \gamma_r L (h - H) = \frac{1}{2} (125 \text{ pcf}) (18 \text{ ft}) (29 \text{ ft} - 20 \text{ ft}) = 10.12 \text{ k/ft}$$

Step 5. Summarize Load Combinations, Load Factors, and Resistance Factors

The design requires checking Strength I and Service I limit states. This is a *simple* wall. Note that examination of only the critical loading combination, as described in Section 4.2, is sufficient for simple walls. Load factors typically used for MSE walls are listed in Tables 4-1 and 4-2. Load factors applicable to this problem are listed in Table E1-5.1.

Table E1-5.1. Summary of applicable load factors

Load Combination	Load Factors		
	EV	ES	EH
Strength I (maximum)	1.35	1.50	1.50
Strength I (minimum)	1.00	0.75	0.90
Service I	1.00	1.00	1.00

Resistance factors for external stability and for internal stability are summarized in Table E1-5.2, see Tables 4-6 and 4-8 for more detail and AASHTO reference.

Table E1-5.2. Summary of applicable resistance factors for evaluation of resistances

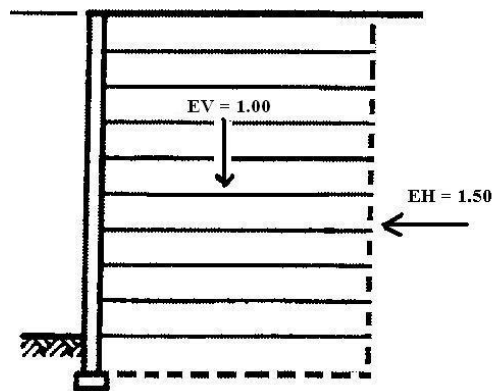
Item	Resistance Factor
Sliding of MSE wall on foundation soil	$\phi_s = 1.00$
Bearing resistance	$\phi_b = 0.65$
Tensile resistance and connectors for geosynthetic reinforcement – static	$\phi_t = 0.90$
Pullout resistance – static	$\phi_p = 0.90$

Step 6. Evaluate External Stability

The external stability is a function of the various forces and moments that are shown in Figure E1-2. In the LRFD context the forces and moments need to be categorized into various load types. For this example problem, the primary load types are soil loads (EV, EH and ES).

6.1 Evaluate Sliding Stability

This is a simple wall and, therefore, which load factor – minimum or maximum – is readily identified, see Figure E1.6-1 below (or Figure 4-1) for load factors for sliding and eccentricity checks.



E.1.6-1 Typical load factors for sliding stability and eccentricity check.

The factored resistance against failure by sliding (R_R) can be estimated with Eq. 4-4:

$$R_R = \phi_\tau R_\tau$$

- 1) Calculate nominal thrust, per unit width, acting on the back of the reinforced zone. From Step 4:

$$F_{H1} = 18.46 \text{ k/ft}$$

$$F_{H2} = 2.55 \text{ k/ft}$$

- 2) Calculate the nominal and the factored horizontal driving forces. For a broken back slope and uniform live load surcharge, use Equations 4-9, 4-10, and 4-11 to calculate the factored driving force. Use the maximum load factors of $\gamma_{EH} = 1.50$ and $\gamma_{LS} = 1.75$ in these equations because it creates the maximum driving force effect for the sliding limit state.

$$P_d = \gamma_{EH} F_{H1} + \gamma_{LS} F_{H2}$$

$$P_d = \gamma_{EH} F_{H1} + \gamma_{LS} F_{H2} = 1.50 (18.46) + 1.75 (2.55) = 27.69 + 4.46 = 32.15 \text{ k/ft}$$

- 3) Assume that the critical sliding failure is along the foundation soil. Thus, the frictional property is $\tan \phi'_f$. Since this is a sheet type of reinforcement, sliding should also be checked at the elevation of the first layer of soil reinforcement (and applicable height).

$$\mu = \tan \phi'_f = \tan 30^\circ = 0.577$$

- 4) Calculate the nominal components of resisting force and the factored resisting force per unit length of wall. The minimum EV load factor ($= 1.00$) is used because it results in minimum resistance for the sliding limit state. The maximum EH and LS load factors are used to stay consistent with factors used to calculate the driving forces. The factored resistance, R_r , is equal to:

$$R_r = [\gamma_{EV} (V_1 + V_2) + \gamma_{EH} (F_{V1}) + \gamma_{LS} (F_{V2})] \times \mu$$

$$R_r = [1.00 (45.00 + 10.12 \text{ k/ft}) + 1.50 (4.16 \text{ k/ft}) + 1.75 (0.57)] (0.577)$$

$$R_r = (55.12 + 6.24 + 1.00) (0.577) = 36.0 \text{ k/ft}$$

- 5) Compare factored sliding resistance, R_r , to the factored driving force, P_d , to check that resistance is greater. If the $CDR < 1.0$, increase the reinforcement length, L , and repeat the calculations. The sliding capacity demand ratio is:

$$CDR_s = \frac{R_r}{P_d} = \frac{36.0 \text{ k/ft}}{32.15 \text{ k/ft}} = 1.12 \quad \therefore \underline{\underline{\text{O.K.}}}$$

6.2 Eccentricity Limit Check

The system of forces for checking the eccentricity at the base of the wall is shown on Figure E1.6-2. The weight and width of the wall facing is neglected in the calculations.

Sum the factored moments about the centerline of the wall zone, with the loads as previously defined and moment arms as shown in Figure E1.6-1. This is a simple wall and, therefore, which load factor – minimum or maximum – is readily identified, see Figure 4-1, and are the same as used for the sliding check.

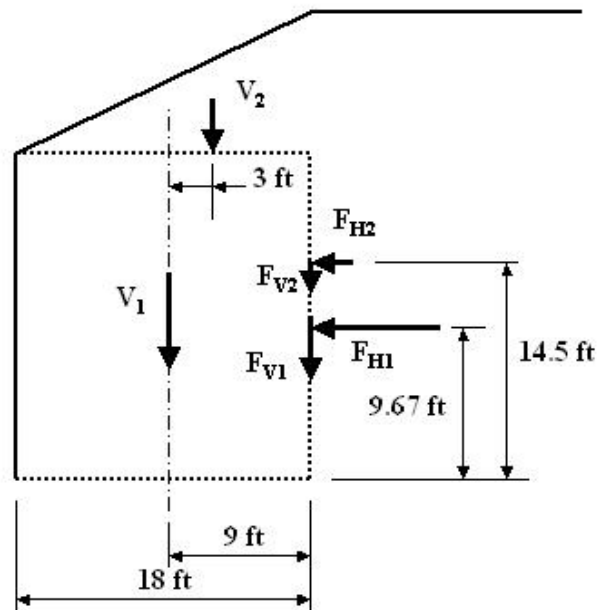


Figure E1.6-2 Forces for eccentricity check.

$$e = \frac{\sum M_O + \sum M_R}{\sum V}$$

$$e = \frac{\gamma_{EH-MAX} F_{H1} (9.67 \text{ ft}) + \gamma_{LS} F_{H2} (14.5 \text{ ft}) - \gamma_{EV-MIN} V_1 (0) - \gamma_{EV-MIN} V_2 (3 \text{ ft}) - (\gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}) (9 \text{ ft})}{\gamma_{EV-MIN} V_1 + \gamma_{EV-MIN} V_2 + \gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}}$$

$$e = \frac{1.50(18.46)(9.67 \text{ ft}) + 1.75(2.55)(14.5 \text{ ft}) - 1.00(45.00)(0) - 1.00(10.12)(3 \text{ ft}) - [1.50(4.16) + 1.75(0.57)](9 \text{ ft})}{1.00(45.00) + 1.00(10.12) + 1.50(4.16) + 1.75(0.57)}$$

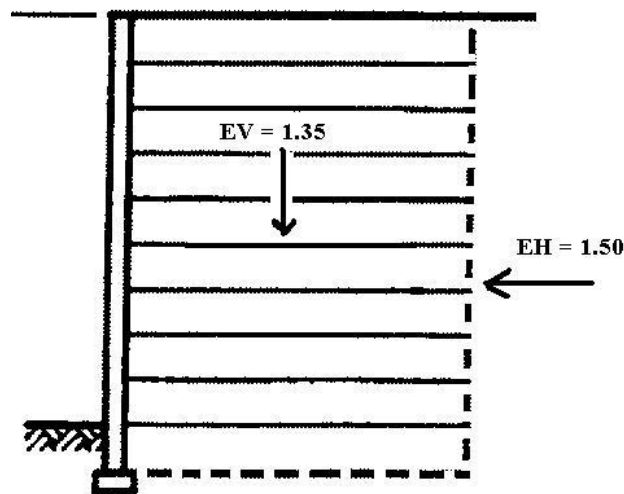
$$e = \frac{267.8 \text{ k} \cdot \text{ft} + 64.70 \text{ k} \cdot \text{ft} - 0 - 30.36 \text{ k} \cdot \text{ft} - 65.14 \text{ k} \cdot \text{ft}}{45.00 \text{ k} + 10.12 \text{ k} + 6.24 \text{ k} + 1.00 \text{ k}} = \frac{237.00 \text{ k} \cdot \text{ft}}{62.36 \text{ k}} = 3.80 \text{ ft}$$

Check that $e \leq L/4$:

$$\frac{L}{4} = \frac{18}{4} = 4.5 \text{ ft} \quad e = 3.80 \leq 4.50 \text{ ft} \quad \therefore \text{O.K.}$$

6.3 Evaluate Bearing on Foundation

This step, 6.3, requires a different computation of the eccentricity value computed in Step 6.2 because different, i.e., maximum in lieu of minimum, load factor(s) are used. This is a simple wall and, therefore, which load factor – minimum or maximum – is readily identified, see Figure E1.6-3 below (or Figure 4-1) for load factors for bearing check.



E.1.6-1 Typical load factors for bearing check.

- 1) Calculate the eccentricity, e_B , of the resulting force at the base of the wall. The e value from the eccentricity check, Step 6.a, cannot be used, calculate e_B with factored loads. The maximum load factors for γ_{EH} and γ_{EV} are used to be consistent with the computation for σ_v (below) where maximum load factors results in the maximum vertical stress.

$$e_B = \frac{\gamma_{EH-MAX} F_{H1} (9.67 \text{ ft}) + \gamma_{LS} F_{H2} (14.5 \text{ ft}) - \gamma_{EV-MAX} V_1 (0) - \gamma_{EV-MAX} V_2 (3 \text{ ft}) - (\gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}) (9 \text{ ft})}{\gamma_{EV-MAX} V_1 + \gamma_{EV-MAX} V_2 + \gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}}$$

$$e_B = \frac{1.50 (18.46) (9.67 \text{ ft}) + 1.75 (2.55) (14.5 \text{ ft}) - 1.35 (45.00) (0) - 1.35 (10.12) (3 \text{ ft}) - [1.50 (4.16) + 1.75 (0.57)] (9 \text{ ft})}{1.35 (45.00) + 1.35 (10.12) + 1.50 (4.16) + 1.75 (0.57)}$$

$$e_B = \frac{267.8 \text{ k} \cdot \text{ft} + 64.70 \text{ k} \cdot \text{ft} - 0 - 41.00 \text{ k} \cdot \text{ft} - 65.14 \text{ k} \cdot \text{ft}}{60.75 \text{ k} + 13.66 \text{ k} + 6.24 \text{ k} + 1.00 \text{ k}} = \frac{226.36 \text{ k} \cdot \text{ft}}{81.65 \text{ k}} = 2.77 \text{ ft}$$

- 2) Calculate the factored vertical stress σ_{V-F} at the base assuming Meyerhof-type distribution. Maintain consistency with loads and load factors used in the eccentricity calculation and corresponding bearing stress calculation.

$$\sigma_v = \frac{\sum V}{L - 2e_B}$$

For this wall with a broken backslope and traffic surcharge the factored bearing stress is:

$$q_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{EV-MAX} V_2 + \gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}}{L - 2e_B}$$

$$q_{V-F} = \frac{81.65 \text{ k/ft}}{18 \text{ ft} - 2(2.77 \text{ ft})} = 6.55 \text{ ksf}$$

- 3) Determine the nominal bearing resistance, q_n , see Eq. 4-22.

The nominal bearing resistance for strength limit state was provided. $q_{n-str} = 10.50 \text{ ksf}$

- 4) Compute the factored bearing resistance, q_R . The resistance factor, ϕ , for MSE walls is equal to 0.65 (see Table E1-5.2). The factored bearing resistance (q_R or q_{nf-str}) was given in Step 2 as equal to 10.50 ksf. (see Eq. 4-23) as:

$$q_R = \phi q_n$$

$$q_R = 10.50 \text{ ksf}$$

- 5) Compare the factored bearing resistance, q_R , to the factored bearing stress, σ_{V-F} , to check that the resistance is greater.

$$CDR_s = \frac{q_R}{\sigma_{V-F}} = \frac{10.50 \text{ ksf}}{6.55 \text{ ksf}} = 1.60 \quad \therefore \underline{\underline{\text{O.K.}}}$$

6.4 Settlement Estimate

Settlement is evaluated at Service I Limit State. From Step 2, the estimated settlement under a bearing stress of 7.50 ksf is 1.00 in. The bearing stress for Service I limit state is ____ ksf. Therefore, the settlement will be less than 1.00 in.

Step 7 EVALUATE INTERNAL STABILITY

7.1 Select Type of Soil Reinforcement

Geogrid soil reinforcement will be used. Three grades, or strengths, of geogrid may be used. The grades and ultimate tensile strengths of these geogrids are summarized in Table E1-7.1.

Table E1-7.1. Geogrid grades and strengths.

Name (Grade):	GG-I	GG-II	GG-III
Ultimate Tensile Strength (lb/ft):	6,000	9,000	12,000

7.2 Define Critical Slip Surface

The critical failure surface is approximately linear in the case of extensible, geogrid reinforcements (see Figure E1-7-1), and passes through the toe of the wall.

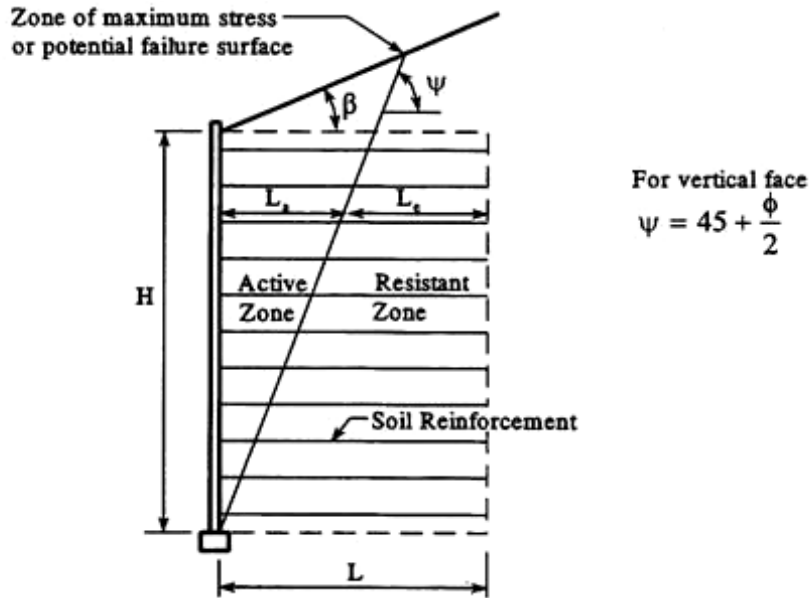


Figure E.7-1. Location of failure surface for internal stability of MSE walls and extensible reinforcements (from Figure 4-9).

7.3 Define Unfactored Loads

The relationship between the type of the reinforcement and the overburden stress is shown in Figure 4-10. The K_r/K_a ratio extensible (e.g., geogrid) reinforcement is a constant, and is equal to 1.0.

The lateral earth pressure coefficient K_r is determined by applying a multiplier to the active earth pressure coefficient. The active earth pressure coefficient is determined using a Coulomb earth pressure relationship, assuming no wall friction and a β angle equal to zero (i.e., equivalent to the Rankine earth pressure coefficient, see Eq. 4-25). With a reinforced fill friction angle of 34° , the active lateral earth pressure coefficient is:

$$K_a = \tan^2 \left(45 - \frac{\phi_r}{2} \right) = \tan^2 \left(45 - \frac{34^\circ}{2} \right) = 0.283$$

Therefore,

$$K_r = K_a \left(\frac{K_r}{K_a} \right) = 0.283(1.0) = 0.283$$

The stress, σ_2 , due to a sloping backfill on top of an MSE wall can be determined as shown in Figure 4-11. An equivalent soil height, S , is computed based upon the slope geometry. The value of S should not exceed the slope height for broken back sloping fills. A reinforcement length of $0.7H$ is used to compute the sloping backfill stress, σ_2 , on the soil reinforcement, as a greater length would only have minimal effect on the reinforcement. The vertical stress is equal to the product equivalent soil height and the reinforced fill unit weight, and is uniformly applied across the top of the MSE zone.

The equivalent uniform height of soil, S_{eq} , is equal to:

$$S_{eq} = \left(\frac{1}{2}\right)0.7H \tan\beta = \left(\frac{1}{2}\right)0.7(20 \text{ ft}) \tan 26.6^\circ = 3.51 \text{ ft}$$

7.4 Establish Vertical Layout of Soil Reinforcements

The MBW units are 8 inches tall. The geogrid soil reinforcement spacing is listed in Table E1-7.5. The upper layer of geogrid will be 8 inches below top of wall, and the bottom layer of geogrid will be 8 inches above the leveling pad. The grade of geogrid to use at each elevation will be determined by strength and connection requirements.

7.5 Calculate Factored Tensile Forces in the Reinforcement Layers

The factored horizontal stress, σ_H , at any depth Z below the top of wall is equal to (after equation 4-29):

$$\sigma_H = K_r [\gamma_r (Z + S_{eq}) \gamma_{EV-MAX}]$$

The maximum tension T_{MAX} in each reinforcement layer per unit width of wall based on the vertical spacing S_v (see Eq. 4-32a) is:

$$T_{MAX} = \sigma_H S_v$$

The term S_v is equal to the vertical reinforcement spacing for a layer where vertically adjacent reinforcements are equally spaced from the layer under consideration. In this case, σ_H , calculated at the level of the reinforcement, is at the center of the contributory height. The contributory height is defined as the midpoint between vertically adjacent reinforcement elevations, except for the top and bottom layers reinforcement. For the top and bottom layers

of reinforcement, S_v is the distance from top or bottom of wall, respectively, to the midpoint between the first and second layer (from top or bottom of wall, respectively) of reinforcement. S_v distances are illustrated in Figure 4-14.

The factored horizontal stress, vertical spacing, and maximum tension for all layers are summarized in Table E1-7.5. Example calculation, for layer #3 follows.

$$\begin{array}{lll} \text{For all layers:} & K_r = 0.283 & \gamma_r = 125 \text{ pcf} \\ & S_{eq} = 3.51 \text{ ft} & \gamma_{EV-MAX} = 1.35 \end{array}$$

$$\text{For Layer \#3:} \quad Z = 4.67 \text{ ft} \quad S_v = 2.0 \text{ ft}$$

$$\sigma_H = K_r [\gamma_r (Z + S_{eq}) \gamma_{EV-MAX}] = 0.283 [125 \text{ pcf} (4.67 + 3.51 \text{ ft}) (1.35)] = 391 \text{ lb/ft}^2$$

$$T_{MAX} = \sigma_H S_v = 391 \text{ lb/ft}^2 (2.0 \text{ ft}) = 781 \text{ lb/ft}$$

$$\begin{array}{lll} \text{For Layer \#1:} & Z = 0.67 \text{ ft} & S_v = 1.67 \text{ ft} \\ & S_{V-Top} = 0 \text{ ft} & S_{V-Bottom} = 1.67 \text{ ft} \quad Z_{Ave} = 0.835 \text{ ft} \end{array}$$

$$\sigma_H = K_r [\gamma_r (Z_{AVE} + S_{eq}) \gamma_{EV-MAX}] = 0.283 [125 \text{ pcf} (0.835 + 3.51 \text{ ft}) (1.35)] = 207 \text{ lb/ft}^2$$

$$T_{MAX} = \sigma_H S_v = 207 \text{ lb/ft}^2 (1.67 \text{ ft}) = 346 \text{ lb/ft}$$

Table E1-7.2 Maximum Tension in Geogrid Layers.

Layer #	Z (ft)	S_v (ft)	σ_H (lb/ft ²)	T_{MAX} (lb/ft)
1	0.67	1.67	115	346
2	2.67	2.0	203	590
3	4.67	2.0	298	781
4	6.67	2.0	394	972
5	8.67	2.0	489	1163
6	10.67	2.0	585	1354
7	12.67	2.0	681	1545
8	14.67	2.0	776	1736
9	16.67	2.0	872	1927
10	18.67	1.33	951	1366
11	19.33	1.0	1006	1091

7.6 Calculate Soil Reinforcement Resistance

The nominal geosynthetic reinforcement strength, T_{al} , per Eq. 3-12, is equal to:

$$T_{al} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_{ID} \times RF_{CR} \times RF_D}$$

The procedure and discussion on definition of nominal long-term reinforcement design strength (T_{al}), for both steel and geosynthetic reinforcements, are presented in Section 3.5 of this manual.

The factored soil resistance is the product of the nominal long-term strength and applicable resistance factor, ϕ . The resistance factors for tensile rupture of MSE wall soil reinforcements are summarized in Table 4-8. The resistance factor for geosynthetic reinforcement is 0.90. The factored soil reinforcement tensile resistance, T_r , is (per Eq. 4-33) equal to:

$$T_r = \phi T_{al}$$

The strength reduction factors, nominal resistance, and factored resistance for the three grades of geogrids are summarized in Table E1-7.3.

Table E1-7.3 Geogrid Nominal and Factored Resistance.

Geogrid:	GG-I	GG-II	GG-III
T_{ult} (lb/ft)	3,000	6,000	9,000
Creep, RF_{CR}	1.85	1.85	1.85
Durability, RF_D	1.15	1.15	1.15
Installation, RF_{ID}	1.3	1.3	1.2
T_{al} (lb/ft)	1,085	2,169	3,525
T_r	976	1,952	3,173

7.7 Select Grade of and/or Number of Soil Reinforcement Elements at Each Level

The soil reinforcement vertical layout, the factored tensile force at each reinforcement level, and the factored soil reinforcement resistance were defined in the previous three steps. Suitable grades (strength) of reinforcement for the defined vertical reinforcement layout is summarized in Table E1-7.4. The CRD for each layer is also listed.

Check this layout for pullout and connection resistance. Adjust layout if/as necessary.

Table E1-7.4 Geogrid Nominal and Factored Resistance.

Layer #	Z (ft)	S _v (ft)	T _{MAX} (lb/ft)	Geogrid		CDR
				Grade	T _r (lb/ft)	
1	0.67	1.67	346	GG-I	976	2.82
2	2.67	2.0	590	GG-I	976	1.65
3	4.67	2.0	781	GG-I	976	1.25
4	6.67	2.0	972	GG-I	976	1.00
5	8.67	2.0	1163	GG-II	1,952	1.68
6	10.67	2.0	1354	GG-II	1,952	1.44
7	12.67	2.0	1545	GG-II	1,952	1.26
8	14.67	2.0	1736	GG-II	1,952	1.12
9	16.67	2.0	1927	GG-II	1,952	1.01
10	18.67	1.33	1366	GG-II	1,952	1.43
11	19.33	1.0	1091	GG-II	1,952	1.79

7.8 Internal Stability with Respect to Pullout Failure

Therefore, the required embedment length in the resistance zone (i.e., beyond the potential failure surface) can be determined from (Eq. 4-36):

$$L_e \geq \frac{T_{MAX}}{\phi F^* \alpha \sigma_v C R_c} \geq 3 \text{ ft (1 m)}$$

where:

- L_e = Length of embedment in the resisting zone
- T_{MAX} = Maximum reinforcement tension
- φ = Resistance factor for soil reinforcement pullout, = 0.90
- F* = Pullout resistance factor, = 0.45 for these geogrids
- α = Scale correction factor, = 0.8 for these geogrids
- σ_v = Average (see Figure E.7-2), nominal (i.e., unfactored) vertical stress at the reinforcement level in the resistant zone
- C = 2 for geogrid type reinforcement
- R_c = Coverage ratio, = 1.0 for geogrid and 100% coverage

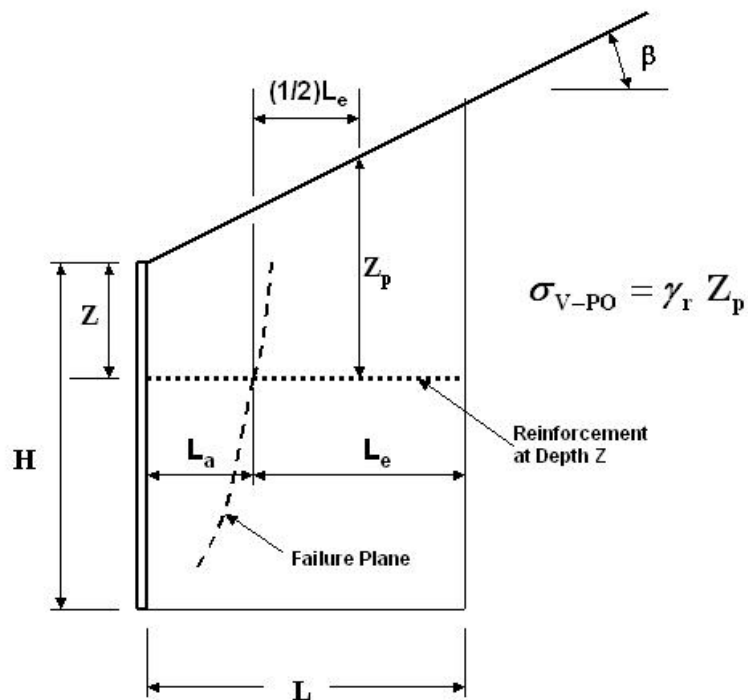


Figure E.7-2. Nominal vertical stress at the reinforcement level in the resistant zone, beneath a sloping backfill (also presented as Figure 4-15).

Table E1-7.5 Pullout Check.

Layer #	Z (ft)	L _a (ft)	Available L _e (ft)	T _{MAX} (lb/ft)	Z _P (ft)	Required L _e (ft)	CDR
1	0.67	10.28	7.72	346	7.74	0.55	14.0
2	2.67	9.22	8.78	590	9.47	0.77	11.4
3	4.67	8.16	9.84	781	11.21	0.86	11.4
4	6.67	7.09	10.91	972	12.94	0.93	11.8
5	8.67	6.03	11.97	1163	14.68	0.98	12.2
6	10.67	4.96	13.04	1354	16.41	1.02	12.8
7	12.67	3.90	14.10	1545	18.14	1.05	13.4
8	14.67	2.84	15.16	1736	19.88	1.08	14.1
9	16.67	1.77	16.23	1927	21.61	1.10	14.7
10	18.67	0.71	17.29	1366	23.35	0.72	23.9
11	19.33	0.36	17.64	1091	23.92	0.56	31.1

7.9 Check Connection Strength

The connection of the reinforcements with the facing, should be designed for T_{MAX} for all limit states. The resistance factors (ϕ) for the connectors is the same as for the reinforcement strength, i.e., $\phi = 0.90$ for geogrids.

The nominal long-term connection strengths, T_{alc} , based upon laboratory connection tests between these MBW units and geogrids, as a function of geogrid grade and normal pressure, are summarized in Table E1-7.6.

Table E1-7.6 Connection Strength Check.

Layer #	Geogrid Grade	T_{alc}^A (lb/ft)	ϕT_{alc} (lb/ft)	T_{MAX} (lb/ft)	CDR
1	GG-I	533	480	346	1.39
2	GG-I	733	660	590	1.12
3	GG-I	933	840	781	1.08
4	GG-I	1133	1020	972	1.05
5	GG-II	1333	1200	1163	1.03
6	GG-II	1533	1380	1354	1.02
7	GG-II	1733	1560	1545	1.01
8	GG-II	1933	1740	1736	1.00
9	GG-II	2150	1935	1927	1.00
10	GG-II	2450	2205 1952 ^B	1366	1.43
11	GG-II	2550	2295 1952 ^B	1091	1.79
Notes: A. T_{alc} values previously established when Agency placed this system on its approved wall systems list. B. The T_r value limits factored connection strength.					

7.10 Lateral Movements

The magnitude of lateral displacement depends on fill placement techniques, compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-facing connection details, and details of the wall facing. A rough estimate of probable lateral displacements of simple MSE walls that may occur during construction can be estimated based on empirical correlations (see Figure 2-15). It is assumed that experience with this type of MBW unit facing and wall fill material have demonstrated lateral movements are within acceptable limits.

7.11 Vertical Movement and Bearing Pads

Bearing pads are generally not used with MBW unit facings, and are not used with this example problem. The wall height is 20 feet, and is below the recommended maximum height of 32 ft without bearing pads (see 3.6.1).

Calculation of the external settlement was reviewed in Step 6.4. The reinforced wall fill is a well graded, granular soil and, therefore, the internal movement will be negligible with good compaction control during construction.

Step 8. Design of Facing Elements

Facing elements are designed to resist the horizontal forces developed in Step 7. With the modular concrete facing blocks (MBW), the maximum spacing between reinforcement layers should be limited to twice the front to back width, i.e., 24 in. The maximum depth of facing below the bottom reinforcement layer is 8 in., and is less than the MBW unit depth. The top row of reinforcement is 8 in. below top of wall, and is less than 1.5 the block depth. Sufficient inter-unit shear capacity exceeds the factored horizontal earth pressure at the facing.

Step 9. Assess Overall/Global Stability

This design step is performed to check the overall, or global, stability of the wall. Overall stability is determined using rotational or wedge analyses, as appropriate, to examine potential failure planes passing behind and under the reinforced zone. Analyses can be performed using a classical slope stability analysis method with standard slope stability computer programs. This step is not detailed in this example calculation, see Chapter 9.

Step 10. Assess Compound Stability

This design step is performed to check potential compound failure planes passing through the reinforced soil zone. Compound stability is determined using rotational or wedge analyses, as appropriate, performed with computer programs that directly incorporate reinforcement elements in the analyses. This step is not detailed in this example calculation, see Chapter 9.

Step 11. Wall Drainage Systems

Subsurface and surface drainage are important aspects in the design and specifying of MSE walls. The Agency should detail and specify drainage requirements for vendor designed walls. Furthermore, the Agency should coordinate the drainage design and detailing (e.g., outlets) within its own designers and with the vendor. This step is not detailed in this example calculation, see Chapter 5.

EXAMPLE E2
BEARING CHECK FOR EXAMPLE E1 MSE WALL
WITHOUT and WITH HIGH GROUNDWATER,
and WITH A SLOPING TOE

E2-1 INTRODUCTION

This example problem demonstrates the strength limit state bearing resistance analyses of an MSE wall with various foundation conditions. A flat bearing surface with and without a high groundwater condition, and a sloping toe without groundwater are examined. The MSE wall configuration to be analyzed is shown in Figure E.2-1 (and in Figure E.1-1).

This MSE wall is (assumed to be) a “simple” structure and, therefore, is analyzed with the load factors that typically control external stability analyses (see Figure 4-1). The design steps used in these calculations follow the basic design steps presented in Table 4-3, of which, the primary steps are presented in Table E1-1. Each of the steps and sub-steps are sequential. Therefore, if the design is revised at any step or sub-step the previous computations need to be re-examined. Each step and sub-step follow.

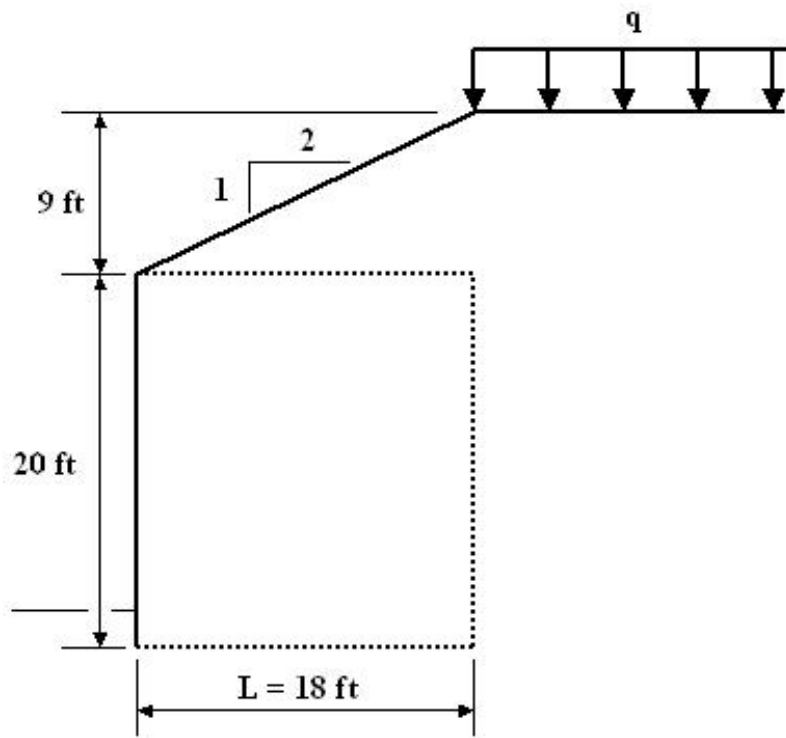


Figure E.2-1 Configuration of example problem E1.

Table E.2-1. Primary Design/Analysis Steps

Step 1.	Establish Project Requirements
Step 2.	Establish Project Parameters
Step 3.	Estimate Wall Embedment Depth, Design Height(s), and Reinforcement Length
Step 4.	Define unfactored loads
Step 5.	Summarize Load Combinations, Load Factors, and Resistance Factors
Step 6.	Evaluate External Stability
Step 7.	Evaluate Internal Stability
Step 8.	Design of Facing Elements
Step 9.	Assess Overall Global Stability
Step 10.	Assess Compound Stability
Step 11.	Design Wall Drainage Systems

See Example E1 for Steps 1 – 11.

However, for these computations, bearing resistances are computed (for several cases) in lieu of defined bearing resistances.

Step 2. Establish Project Parameters

- Foundation soil, $\phi_f^l = 30^\circ$, $\gamma_f = 125$ pcf
- Factored bearing resistance of foundation soil
 - For service limit consideration, $q_{nf-ser} = 7.5$ ksf for 1-inch of total settlement
 - For strength limit consideration, q_{nf-str} – is to be determined
- Reinforced wall fill, $\phi_r^l = 34^\circ$, $\gamma_r = 125$ pcf, $pH = 7.3$, maximum size $\frac{3}{4}$ -inch
- Retained backfill, $\phi_b^l = 30^\circ$, $\gamma_b = 125$ pcf

From Example E1:

Step 4. Define Unfactored Loads

Traffic Load

The traffic load is on the level surface of the retained backfill. For external stability, traffic load for walls parallel to traffic and more than 1-ft behind the backface of the MSE wall is represented by an equivalent height of soil, h_{eq} equal to 2.0 ft.

Unfactored Loads:

$$F_1 = \frac{1}{2} \gamma_b h^2 K_{ab} = \frac{1}{2} (125 \text{ pcf}) (29 \text{ ft})^2 (0.360) = 18,922 \text{ lb/ft} = 18.92 \text{ k/ft}$$

$$F_{H1} = F_1 \cos I = 18.92 \text{ k/ft} (\cos 12.7^\circ) = 18.46 \text{ k/ft}$$

$$F_{V1} = F_1 \sin I = 18.92 \text{ k/ft} (\sin 12.7^\circ) = 4.16 \text{ k/ft}$$

$$q = 2.0 \text{ ft} (125 \text{ pcf}) = 250 \text{ psf}$$

$$F_2 = q h K_{ab} = 250 \text{ psf} (29 \text{ ft}) (0.360) = 2,610 \text{ lb/ft} = 2.61 \text{ k/ft}$$

$$F_{H2} = F_2 \cos I = 2.61 \text{ k/ft} (\cos 12.7^\circ) = 2.55 \text{ k/ft}$$

$$F_{V2} = F_2 \sin I = 2.61 \text{ k/ft} (\sin 12.7^\circ) = 0.57 \text{ k/ft}$$

$$V_1 = \gamma_r H L = 125 \text{ pcf} (20 \text{ ft}) (18 \text{ ft}) = 45,000 \text{ lb/ft} = 45.0 \text{ k/ft}$$

$$V_2 = \frac{1}{2} \gamma_r L (h - H) = \frac{1}{2} (125 \text{ pcf}) (18 \text{ ft}) (29 \text{ ft} - 20 \text{ ft}) = 10,125 \text{ lb/ft} = 10.12 \text{ k/ft}$$

Step 5. Summarize Load Combinations, Load Factors, and Resistance Factors

The design requires checking Strength I limit state. This is a *simple* wall. Note that examination of only the critical loading combination, as described in Section 4.2, is sufficient for simple walls. Load factors typically used for MSE walls are listed in Tables 4-1 and 4-2. Load factors applicable to this problem are listed in Table E2-5.1. Bearing resistance factor for MSE walls is listed in Table E2-5.2

Table E2-5.1. Summary of applicable load factors

Load Combination	Load Factors		
	EV	ES	EH
Strength I (maximum)	1.35	1.50	1.50
Strength I (minimum)	1.00	0.75	0.90

Table E2-5.2. Bearing resistance factor

Item	Resistance Factor
Bearing resistance	$\phi_b = 0.65$

Step 6. Evaluate Bearing on Foundation

This step, 6.3, requires a different computation of the eccentricity value computed in Step 6.2 because different, i.e., maximum in lieu of minimum, load factor(s) are used. This is a simple wall and, therefore, which load factor – minimum or maximum – is readily identified for load factors for bearing check.

- 2) Calculate the factored vertical stress σ_{V-F} at the base assuming Meyerhof-type distribution. Maintain consistency with loads and load factors used in the eccentricity calculation and corresponding bearing stress calculation.

$$\sigma_v = \frac{\sum V}{L - 2e_B}$$

For this wall with a broken backslope and traffic surcharge the factored bearing stress is (see Example E1):

$$q_{V-F} = \frac{\gamma_{EV-MAX} V_1 + \gamma_{EV-MAX} V_2 + \gamma_{EH-MAX} F_{V1} + \gamma_{LS} F_{V2}}{L - 2e_B}$$

$$q_{V-F} = \frac{81.65 \text{ k/ft}}{18 \text{ ft} - 2(2.77 \text{ ft})} = 6.55 \text{ ksf}$$

E2-1 CALCULATIONS

It is not obvious whether the strength or the service (i.e., settlement) limit state controls. Therefore, check both.

2) (cont.)

Calculate e for service limit state:

$$e_{b-ser} = \frac{F_{H1}(9.67 \text{ ft}) + F_{H2}(14.5 \text{ ft}) - V_1(0) - V_2(3 \text{ ft}) - (F_{V1} + F_{V2})(9 \text{ ft})}{V_1 + V_2 + F_{V1} + F_{V2}}$$

$$e_{b-ser} = \frac{(18.46)(9.67 \text{ ft}) + (2.55)(14.5 \text{ ft}) - (45.00)(0) - (10.12)(3 \text{ ft}) - (4.16 + 0.57)(9 \text{ ft})}{(45.00) + (10.12) + (4.16) + (0.57)}$$

$$e_{b-ser} = \frac{178.51 \text{ k} - \text{ft} + 36.98 \text{ k} - \text{ft} - 0 - 30.36 \text{ k} - \text{ft} - 42.57 \text{ k} - \text{ft}}{59.85 \text{ k}} = \frac{142.55 \text{ k} - \text{ft}}{59.85 \text{ k}} = 2.38 \text{ ft}$$

Calculate service limit state bearing stress:

$$q_{v-service} = \frac{1.00(45 \text{ k/ft}) + 1.00(10.12 \text{ k/ft}) + (1.00)4.16 \text{ k/ft}}{18 \text{ ft} - 2(2.38 \text{ ft})} = \frac{59.85 \text{ k/ft}}{13.24 \text{ ft}} = 4.52 \text{ ksf}$$

The bearing stress of 4.52 ksf is less than the stated 7.5 ksf for a 1-inch total settlement. Therefore, less than 1-inch of settlement is anticipated and service limit state is O.K.

Note: See FHWA Soils and Foundations reference manual, FHWA NHI-06-089 (Samtani and Nowatzki, 2006) for settlement analysis and bearing pressure versus settlement plotting procedures.

- 3) Determine the nominal bearing resistance, q_n , see Eq. 4-22.

The nominal bearing resistance for strength I (max) limit state, with N_γ from Table 4-6, for $\phi'_f = 30^\circ$ is:

$$q_n = c_f N_c + 0.5 L' \gamma_f N_\gamma = 0 N_c + 0.5 (12.46 \text{ ft}) (125 \text{ pcf}) (22.4) = 17.44 \text{ ksf}$$

$$\text{where } L' = 18 \text{ ft} - 2 (2.77 \text{ ft}) = 12.46 \text{ ft}$$

- 4) Compute the factored bearing resistance, q_R . The resistance factor, ϕ , for MSE walls is equal to 0.65 (see Table E2-5.2). The factored bearing resistance (q_R) is given (see Eq. 4-23) as:

$$q_R = \phi q_n$$

For strength limit state:

$$q_R = 0.65 (17.44 \text{ ksf}) = 11.34 \text{ ksf}$$

- 5) Compare the factored bearing resistance, q_R , to the factored bearing stress, σ_{V-F} , to check that the resistance is greater.

For strength limit state:

$$\text{CDR}_{\text{Strength}} = \frac{q_R}{\sigma_{V-F}} = \frac{11.34 \text{ ksf}}{6.55 \text{ ksf}} = 1.73 \quad \therefore \underline{\underline{\text{O.K.}}}$$

E2-2 CALCULATIONS WITH GROUNDWATER NEAR SURFACE

Compute Strength I (max) limit state bearing resistance and CDR assuming groundwater is 12 ft below the ground surface, as illustrated in Figure E.2-2.

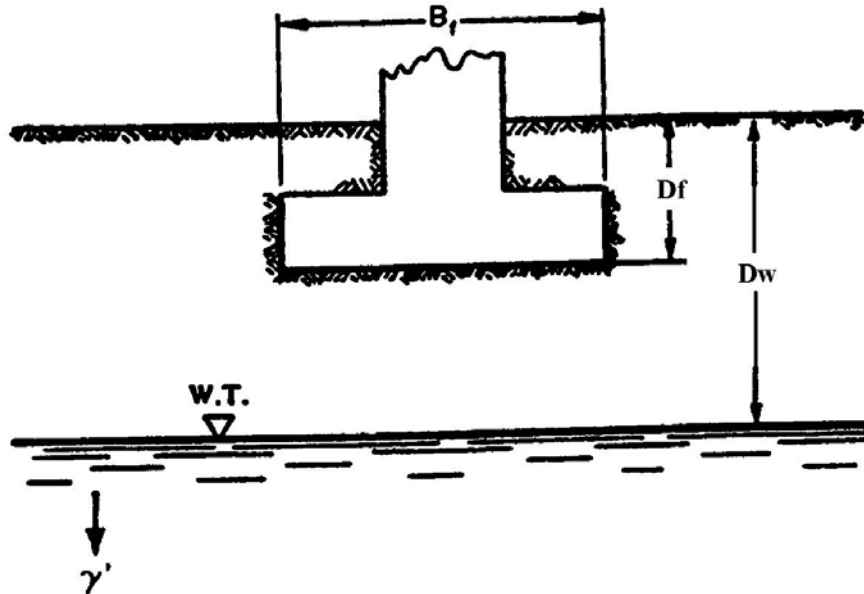


Figure E.2-2. Bearing groundwater influence terms for spread footing
(Note: $B_f = L'$ for MSE wall design).

With groundwater consideration, and no cohesion, the nominal bearing resistance (see Eq. 4-22 and AASHTO 10.6.3.1.2a-1) is equal to:

$$q_n = 0.5 L' \gamma_f N_\gamma C_{wy}$$

The term C_{wy} is defined in Table 10.6.3.1.2a-2 (AASHTO, 2007):

Coefficient C_{wy} for Various Groundwater Depths
(after AASHTO Table 10.6.3.1.2a-2)

D_w	C_{wy}
0.0	0.5
D_f	0.5
$> 1.5 L' + D_f$	1.0
Note: Interpolate between the values shown for intermediate positions of the groundwater table.	

Calculations:

Given: Moist unit weight, $\gamma_m = 125 \text{ lb/ft}^3$

$$D_w = 12 \text{ ft}$$

The buoyant unit weight should be used to compute the overburden pressure if the groundwater table is located with the potential failure zone.

$$1.5 L' + D_f = 1.5 [18 \text{ ft} - 2(2.77 \text{ ft})] + 2 \text{ ft} = 20.69 \text{ ft}$$

$$\text{At } D_w = D_f = 2.0 \text{ ft} \quad C_{w\gamma} = 0.5$$

$$\text{At } D_w = 1.5 L' + D_f = 20.7 \text{ ft} \quad C_{w\gamma} = 1.0$$

Interpolating to $D_w = 12 \text{ ft}$

$$C_{w\gamma} = 0.5 + 0.5 \left(\frac{12 \text{ ft} - 2 \text{ ft}}{20.69 \text{ ft} - 2 \text{ ft}} \right) = 0.77$$

The nominal bearing resistance for strength limit state, with $C_{w\gamma} = 0.77$, moist unit weight $\gamma' = 125 \text{ pcf}$, and N_γ from Table 4-6, is:

$$q_n = 0.5 L' \gamma_f N_\gamma C_{w\gamma} = 0.5 (12.46 \text{ ft}) (125 \text{ pcf}) (22.4) (0.77) = 13,432 \text{ psf} = 13.43 \text{ ksf}$$

The factored strength limit state bearing resistance is:

$$q_R = 0.65 (13.43 \text{ ksf}) = 8.73 \text{ ksf}$$

The capacity to demand ratio is:

$$CDR_{\text{Strength}} = \frac{q_R}{\sigma_{V-F}} = \frac{8.73 \text{ ksf}}{6.55 \text{ ksf}} = 1.33 \quad \therefore \underline{\underline{\text{O.K.}}}$$

E2-3 CALCULATIONS WITH TOE SLOPE AND WITHOUT GROUNDWATER

Compute Strength I (max) limit state bearing resistance and CDR assuming with sloping toe and no groundwater, as illustrated in Figure E.2-3, and with the following geometry:

$$\begin{aligned} B &= L - 2 e_B = 12.46 \text{ ft} \\ b &= 4 \text{ ft} \\ D_f &= \text{assume} = 0 \\ I &= 18.4^\circ (3H:1V) \end{aligned}$$

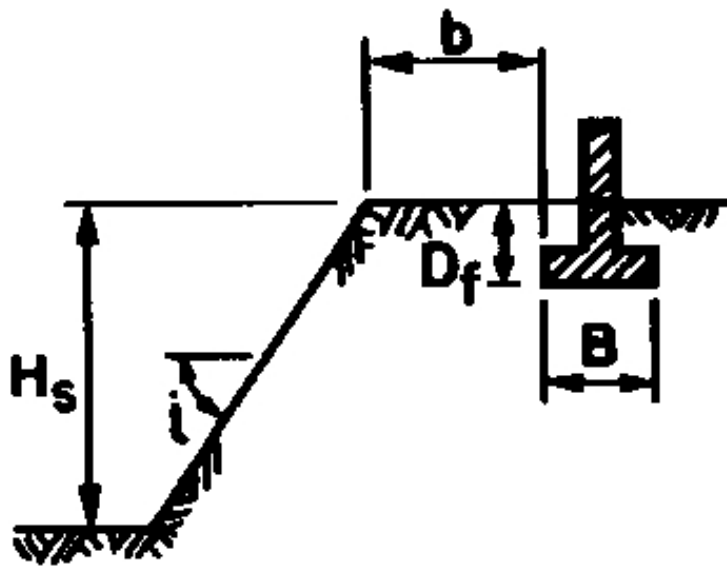


Figure E.2-3. Bearing sloping toe terms.

For footings bearing near a slope the term N_γ is replaced by $N_{\gamma q}$ (AASHTO 10.6.3.1.2c). The $N_{\gamma q}$ term is taken from AASHTO Figure 10.6.3.1.2c-2. The nominal bearing resistance, for a foundation soil with no cohesion, is equal to:

$$q_n = 0.5 L' \gamma_f N_{\gamma q}$$

From AASHTO Figure 10.6.3.1.2c-2 an $N_{\gamma q}$ value equal to approximately 18 is found for $\phi'_f = 30^\circ$, $b/B = 0.32$, and $\beta = 18.4^\circ$.

The nominal bearing resistance for strength limit state with $N_{\gamma q} = 20$ is:

$$q_n = 0.5 L' \gamma_f N_{\gamma q} = 0.5 (12.46 \text{ ft}) (125 \text{ pcf}) (18) = 14,017 \text{ psf} = 14.02 \text{ ksf}$$

The factored strength limit state bearing resistance is:

$$q_R = 0.65 (14.02 \text{ ksf}) = 9.11 \text{ ksf}$$

The capacity to demand ratio is:

$$CDR_{\text{Strength}} = \frac{q_R}{\sigma_{V-F}} = \frac{9.11 \text{ ksf}}{6.55 \text{ ksf}} = 1.39 \quad \therefore \underline{\underline{\text{O.K.}}}$$

EXAMPLE E3 **SEGMENTAL PRECAST PANEL MSE WALL WITH SLOPING BACKFILL** **SURCHARGE**

E3-1 INTRODUCTION

This example problem demonstrates the analysis of a MSE wall with a sloping backfill surcharge. The MSE wall is assumed to include a segmental precast panel face with ribbed steel strip reinforcements. The MSE wall configuration to be analyzed is shown in Figure E3-1. The analysis is based on various principles that were discussed in Chapter 4. Table E3-1 presents a summary of steps involved in the analysis. Each of the steps and sub-steps is sequential and if the design is revised at any step or sub-step then all the previous computations need to be re-visited. Each of the steps and the sub-steps in Table E3-1 is explained in detail herein.

Table E3-1. Summary of steps in analysis of MSE wall with sloping backfill

Step	Item
1	Establish project requirements
2	Establish project parameters
3	Estimate wall embedment depth and length of reinforcement
4	Estimate unfactored loads
5	Summarize applicable load and resistance factors
6	Evaluate external stability of MSE wall 6.1 Evaluation of sliding resistance 6.2 Evaluation of limiting eccentricity 6.3 Evaluation of bearing resistance 6.4 Settlement analysis
7	Evaluate internal stability of MSE wall 7.1 Estimate critical failure surface, variation of K_r and F^* for internal stability 7.2 Establish vertical layout of soil reinforcements 7.3 Calculate horizontal stress and maximum tension at each reinforcement level 7.4 Establish nominal and factored long-term tensile resistance of soil reinforcement 7.5 Establish nominal and factored pullout resistance of soil reinforcement 7.6 Establish number of soil reinforcing strips at each level of reinforcement
8	Design of facing elements
9	Check overall and compound stability at the service limit state.
10	Design wall drainage system

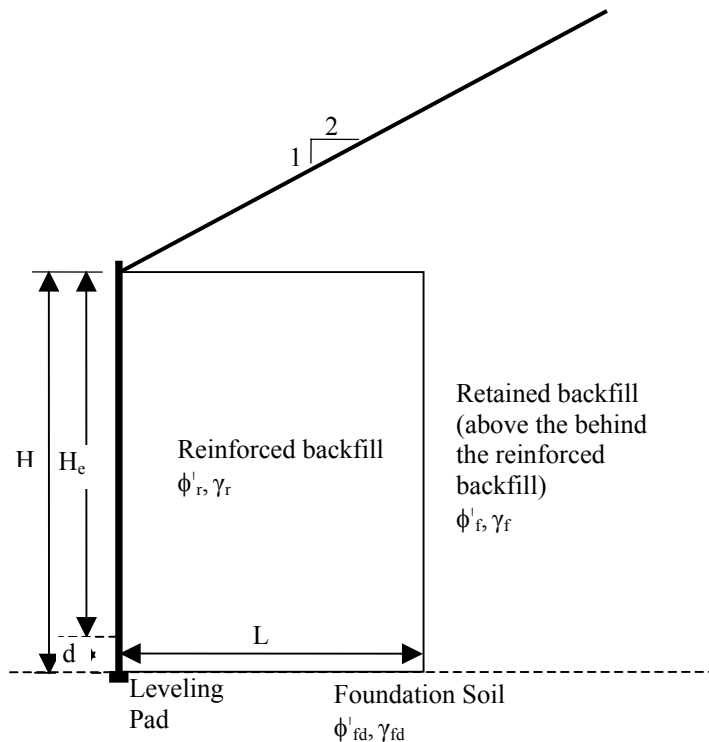


Figure E3-1. Configuration showing various parameters for analysis of a MSE wall with sloping backfill (not-to-scale).

STEP 1. ESTABLISH PROJECT REQUIREMENTS

- Exposed wall height, $H_e = 28$ ft
- Length of wall = 850 ft
- Design life = 75 years
- Precast panel units: 5 ft wide x 5 ft tall x 0.5 ft thick
- Type of reinforcement: Grade 65 ($F_y = 65$ ksi), 1.969 in. x 0.157 in. (50 mm wide x 4 mm) thick galvanized steel ribbed strips with zinc coating of 3.386 mils (86 μm).
- No seismic considerations

STEP 2. EVALUATE PROJECT PARAMETERS

- Reinforced backfill, $\phi_r = 34^\circ$, $\gamma_r = 125$ pcf, coefficient of uniformity, $C_u = 7.0$ and meeting the AASHTO (2007) requirements for electrochemical properties
- Retained backfill, $\phi_f = 30^\circ$, $\gamma_f = 125$ pcf
- Foundation soil, $\phi_{fd} = 30^\circ$, $\gamma_{fd} = 125$ pcf
 - Factored Bearing resistance of foundation soil

- o For service limit consideration, $q_{nf-ser} = 7.50$ ksf for 1-inch of total settlement
- o For strength limit consideration, $q_{nf-str} = 10.50$ ksf

Note: the above bearing resistance values are assumed values for the purpose of this example problem. In actual designs, the geotechnical engineer should develop appropriate project and wall specific values.

STEP 3. ESTIMATE DEPTH OF EMBEDMENT AND LENGTH OF REINFORCEMENT

Based on Table C.11.10.2.2.-1 of AASHTO (2007), the minimum embedment depth = $H/20$ for walls with horizontal ground in front of wall, i.e., 1.4 ft for exposed wall height of 28 ft. For this design, assume embedment, $d = 2.0$ ft. Thus, design height of the wall, $H = H_e + d = 28 \text{ ft} + 2.0 \text{ ft} = 30 \text{ ft}$.

Due to the 2H:1V backslope, the initial length of reinforcement is assumed to be $0.8H$ or 24 ft. This length will be verified as part of the design process. The length of the reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone because differential settlements could overstress the reinforcements.

STEP 4. ESTIMATE UNFACTORED LOADS

Tables E3-4.1 and E3-4.2 present the equations for unfactored loads and moment arms about Point A shown in Figure E3-2. The moments are a product of the respective forces and moment arms. Each force is assigned a designation representing the applicable load type as per Tables 3.4.1-1 and 3.4.1-2 of AASHTO (2007).

To compute the numerical values of various forces and moments, the parameters provided in Step 2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure for the retained fill is computed as follows:

Coefficient of active earth pressure per Eq. 3.11.5.3-1 of AASHTO (2007) is

$$K_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma \sin^2 \theta \sin(\theta - \delta)}$$

where per Eq. 3.11.5.3-2 of AASHTO (2007) the various parameters in above equation are as follows:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta) \sin(\phi'_f - \beta)}{\sin(\theta - \delta) \sin(\theta + \beta)}} \right]^2$$

δ = friction angle between fill and wall taken as specified

β = angle (nominal) of fill to horizontal

θ = angle of back face of wall to horizontal

ϕ'_f = effective angle of internal friction of retained backfill

For the case of level backfill with vertical backface, $\beta = \delta = 0^\circ$ and $\theta = 90^\circ$, the coefficient of active earth pressure is given as follows:

$$K_a = (1 - \sin \phi'_f) / (1 + \sin \phi'_f)$$

For this example problem, compute the coefficient of active earth pressure for the retained fill, K_{af} , using $\beta = 26.56^\circ$ (for the 2:1 backslope), vertical backface, $\theta = 90^\circ$, and $\delta = \beta$ as follows

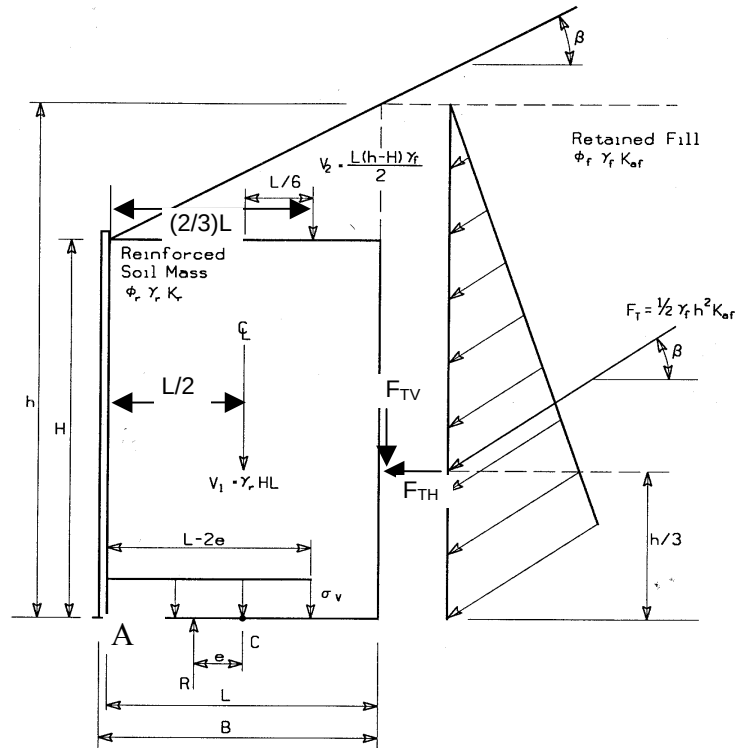
$$\Gamma = \left[1 + \sqrt{\frac{\sin(30^\circ + 26.56^\circ) \sin(30^\circ - 26.56^\circ)}{\sin(90^\circ - 26.56^\circ) \sin(90^\circ + 26.56^\circ)}} \right]^2 = \left[1 + \sqrt{\frac{(0.834)(0.060)}{(0.894)(0.894)}} \right]^2 = 1.563$$

$$K_{af} = \frac{\sin^2(\theta + \phi'_f)}{\Gamma \sin^2 \theta \sin(\theta - \delta)} = \frac{\sin^2(90^\circ + 30^\circ)}{1.563(\sin 90^\circ)^2 [\sin(90^\circ - 26.56^\circ)]} = \frac{0.750}{(1.563)(1.0)(0.894)} = 0.537$$

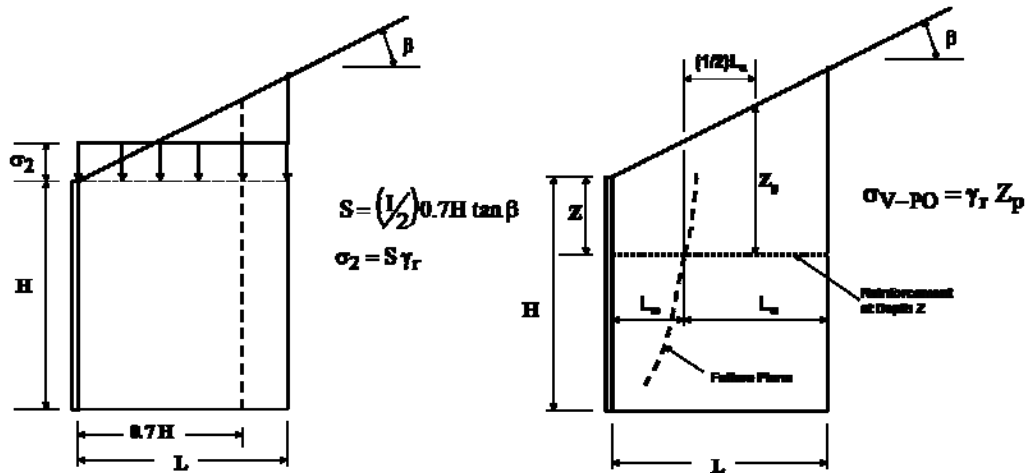
For the example problem, Tables E3-4.3 and E3-4.4 summarize the numerical values unfactored forces and moments, respectively, based on the equations, various dimensions and values of lateral earth pressure coefficients presented above. Refer to Figure E3-2 for notations of various forces.

The unfactored forces and moments in Tables E3-4.3 and E3-4.4 form the basis of all computations in this example problem. The unfactored forces and moments should be multiplied by the appropriate load factors based on the load types identified in the second column of the Tables E3-4.1 and E3-4.2 to perform the analysis for various load combinations such as Strength I, Service I, etc.

The load factors for various load types relevant to this example problem are discussed in Step 5.



(a)



(b)

Figure E3-2. Legend for computation of forces and moments (a) for external stability analysis, (b) for internal stability analysis (not-to-scale).

Table E3-4.1. Equations of computing unfactored vertical forces and moments

Vertical Force (Force/length units)	LRFD Load Type	Moment arm (Length units)
		@ Point A
$V_1 = (\gamma_r)(H)(L)$	EV	$L/2$
$V_2 = \left(\frac{1}{2}\right)(L)(L \tan \beta)(\gamma_f)$	EV	$(2/3)L$
$F_{TV} = (1/2)(\gamma_r)(h^2)(K_{af})(\sin \beta)$	EH	L

Note: $h = H + L \tan \beta$

Table E3-4.2. Equations of computing unfactored horizontal forces and moments

Horizontal Force (Force/length units)	LRFD Load Type	Moment arm (Length units)
		@ Point A
$F_{TH} = (1/2)(\gamma_r)(h^2)(K_{af})(\cos \beta)$	EH	$h/3$

Note: $h = H + L \tan \beta$

For this example problem, $\tan \beta = 0.5$, and $h = 30 \text{ ft} + 24 \text{ ft} (0.5) = 42.00 \text{ ft}$.

Table E3-4.3. Unfactored vertical forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment	Moment at Point A, k-ft/ft
$V_1 =$	90.00	12.00	$MV_1 =$	1080.00
$V_2 =$	18.00	16.00	$MV_2 =$	288.00
$F_{TV} =$	26.48	24.00	$MF_{TV} =$	635.44

Table E3-4.4. Unfactored horizontal forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment	Moment at Point A, k-ft/ft
$F_{TH} =$	52.95	14.00	$MF_{TH} =$	741.35

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table E3-5.1 summarizes the load factors for the various LRFD load type shown in second column of Tables E3-4.1 and E3-4.2. **Throughout the computations in this example problem, the forces and moments in Tables E3-4.1 and E3-4.2 should be multiplied by appropriate load factors.** For example, if computations are being done for Strength I (maximum) load combination, the forces and moments corresponding to load V_2 should be multiplied by 1.35 which is associated with load type EV assigned to load V_2 .

Table E3-5.1. Summary of applicable load factors

Load Combination	Load Factors (after AASHTO, 2007 Tables 3.4.1-1 and 3.4.1-2)	
	EV	EH
Strength I (maximum)	1.35	1.50
Strength I (minimum)	1.00	0.90
Service I	1.00	1.00

For computation of factored resistances during evaluation of strength limits states, appropriate resistance factors have to be used. Table E3-5.2 summarizes the applicable resistance factors. For service limit state, all resistance factors are equal to 1.0.

Table E3-5.2. Summary of applicable resistance factors for evaluation of resistances

Item	Resistance Factors	AASHTO (2007) Reference
Sliding of MSE wall on foundation soil	$\phi_s = 1.00$	Table 11.5.6-1
Bearing resistance	$\phi_b = 0.65$	Table 11.5.6-1
Tensile resistance (for steel strips)	$\phi_t = 0.75$	Table 11.5.6-1
Pullout resistance	$\phi_p = 0.90$	Table 11.5.6-1

STEP 6. EVALUATE EXTERNAL STABILITY OF MSE WALL

The external stability of MSE wall is a function of the various forces and moments that are shown in Figure E3-2a. In the LRFD context the forces and moments need to be categorized into various load types. For this example problem, the primary load types are soil loads (EV and EH).

6.1 Sliding Resistance at Base of MSE Wall

The purpose of these computations is to evaluate the sliding resistance at the base of the MSE wall. Since the computations are related to sliding resistance, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for sliding resistance at the base of the MSE wall are illustrated in Table E3-6.1. Note that sliding resistance is a strength limit state check and therefore service limit state calculations are not performed. Since the friction angle of foundation soil, ϕ_{fd} , is less than the friction angle for reinforced soil, ϕ_r , the sliding check will be performed using ϕ_{fd} . The critical values based on max/min result in the extreme force effect and govern the sliding mode of failure. The resistance to load ratio, CDR, based on critical values of max/min is 1.07 indicating that the choice of 24 ft long reinforcement is justified because lesser length would result in $CDR < 1.0$ which is not acceptable.

Table E3-6.1. Computations for evaluation of sliding resistance of MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Lateral load on the MSE wall, $H_m = F_{TH}$	k/ft	79.43	47.66	N/A
Vertical load at base of MSE wall, $V_{A1} = V_1 + V_2$	k/ft	145.80	108.00	N/A
Vertical load at base of MSE wall, $V_{A2} = F_{TV}$	k/ft	39.72	23.83	N/A
Total vertical load at base of MSE wall, $V_A = V_{A1} + V_{A2}$	k/ft	185.52	131.83	N/A
Nominal sliding resistance at base of MSE wall, $V_{Nm1} = \tan(\phi_{fd})(V_1 + V_2)$	k/ft	84.18	62.35	N/A
Nominal sliding resistance at base of MSE wall, $V_{Nm2} = \tan(\phi_{fd})(F_{TV})$	k/ft	22.93	13.76	N/A
Nominal sliding resistance at base of MSE wall, $V_{Nm} = V_{Nm1} + V_{Nm2}$	k/ft	107.11	76.11	N/A
Factored sliding resistance at base of MSE wall, $V_{Fm1} = \phi_s * V_{Nm1}$	k/ft	84.18	62.35	N/A
Factored sliding resistance at base of MSE wall, $V_{Fm2} = \phi_s * V_{Nm2}$	k/ft	22.93	13.76	N/A
Factored sliding resistance at base of MSE wall, $V_{Fm} = V_{Fm1} + V_{Fm2}$	k/ft	107.11	76.11	N/A
Is $V_{Fm} > H_m$?	-	Yes	Yes	N/A
Capacity:Demand Ratio (CDR) = $V_{Fm} : H_m$	dim	1.35	1.60	N/A
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_F (V_{Fmin})	k/ft	85.28*		
Maximum H_m (H_{mmax})	k/ft	79.43		
Is $V_{Fmin} > H_{mmax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = $V_{Fmin} : H_{mmax}$	dim	1.07		

Note: *85.28 = 62.35+22.93. This is to maintain consistency between the total inclined lateral force and its components.

6.2 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table E3-6.2. Limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. The critical values based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table E3-6.2. Computations for evaluation of limiting eccentricity for MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Vertical load at base of MSE wall, $V_{A1} = V_1 + V_2$	k/ft	145.80	108.00	N/A
Vertical load at base of MSE wall, $V_{A2} = F_{TV}$	k/ft	39.72	23.83	N/A
Total vertical load at base of MSE wall, $V_A = V_{A1} + V_{A2}$	k/ft	185.52	131.83	N/A
Resisting moments about Point A = M_{RA1} $= MV_1 + MV_2$	k-ft/ft	1846.80	1368.00	N/A
Resisting moments about Point A = $M_{RA2} = MF_{TV}$	k-ft/ft	953.17	571.90	N/A
Total resisting moment @ Point A, $M_{RA} = M_{RA1} + M_{RA2}$	k-ft/ft	2799.97	1939.90	N/A
Overturning moments about Point A = $M_{OA} = MF_{TH}$	k-ft/ft	1112.03	667.22	N/A
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft/ft	1687.94	1272.68	N/A
Location of the resultant force on base of MSE wall from Point A, $a = (M_{RA} - M_{OA})/V_A$	ft	9.10	9.65	N/A
Eccentricity at base of MSE wall, $e_L = L/2 - a$	ft	2.90	2.35	N/A
Limiting eccentricity, $e = L/4$ for strength limit state	ft	6.00	6.00	N/A
Is the resultant within limiting value of e_L ?	-	Yes	Yes	N/A
Calculated e_L/L	-	0.12	0.10	N/A
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point A, M_{OA-C}	k-ft/ft	1112.03		
Resisting moments about Point A, M_{RA-C}	k-ft/ft	2321.17*		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	1209.14		
Vertical force, V_{A-C}	k/ft	147.72**		
Location of resultant from Point A, $a_{nl} = M_{A-C}/V_{A-C}$	ft	8.19		
Eccentricity from center of wall base, $e_L = 0.5*L - a_{nl}$	ft	3.81		
Limiting eccentricity, $e = L/4$	ft	6.00		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	16.37		
Calculated e_L/L	-	0.16		

Notes: * $2321.17 = 1368.00 + 953.17$; ** $147.72 = 108.00 + 39.72$. This is to maintain consistency between the total inclined lateral force and its components.

6.3 Bearing Resistance at base of MSE Wall

The bearing stress at the base of the MSE wall can be computed as follows:

$$\sigma_v = \frac{\Sigma V}{L - 2e_L}$$

where $\Sigma V = R = V_1 + V_2 + F_{TV}$ is the resultant of vertical forces and the load eccentricity e_L is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

In LRFD, σ_v is compared with the factored bearing resistance when computed for strength limit state and used for settlement analysis when computed for service limit state. The various computations for evaluation of bearing resistance are presented in Table E3-6.3. The Strength I (max) load combination results in the extreme force effect in terms of maximum bearing stress and therefore governs the bearing resistance mode of failure. The Service I load combination is evaluated to compute the bearing stress for settlement analysis. Since the CDR ≈ 1.0 for Strength I (max) and Service I load combinations, shorter reinforcement lengths are not recommended.

6.4 Settlement Analysis

Settlement is evaluated at Service I Limit State. From Step 2, the estimated settlement under a bearing stress of 7.50 ksf is 1.00 in. From Table E3-6.3, the bearing stress for Service I limit state is 7.16 ksf. Therefore, the settlement will be less than 1.00 in.

Table E3-6.3. Computations for evaluation of bearing resistance for MSE wall

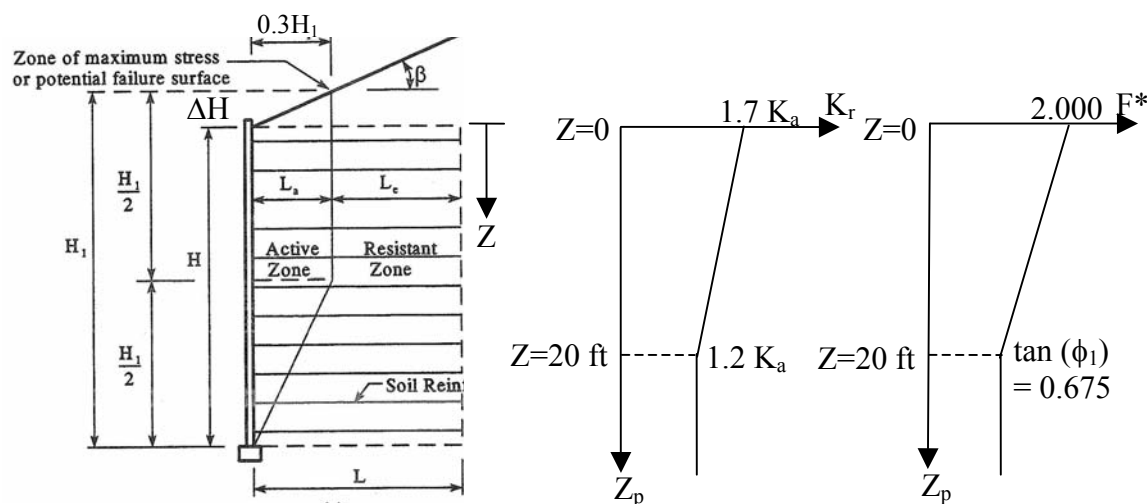
Item	Unit	Str I (max)	Str I (min)	Ser I
Vertical load at base of MSE wall, $V_{Ab1} = V_1 + V_2$	k/ft	145.80	108.00	108.00
Vertical load at base of MSE wall, $V_{Ab2} = F_{TV}$	k/ft	39.72	23.83	26.48
Total vertical load at base of MSE wall, $\Sigma V = R = V_{Ab1} + V_{Ab2}$	k/ft	185.52	131.83	134.48
Resisting moments about Point A, $M_{RA1} = MV_1 + MV_2$	k-ft/ft	1846.80	1368.00	1368.00
Resisting moments about Point A, $M_{RA2} = MF_{TV}$	k-ft/ft	953.17	571.90	635.44
Total resisting moment @ Point A, $M_{RA} = M_{RA1} + M_{RA2}$	k-ft/ft	2799.97	1939.90	2003.44
Overturning moments @ Point A, $M_{OA} = MF_{TH}$	k-ft/ft	1112.03	667.22	741.35
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft/ft	1687.94	1272.68	1262.09
Location of the resultant force on base of MSE wall from Point A, $a = (M_{RA} - M_{OA})/V_A$	ft	9.10	9.65	9.39
Eccentricity at base of MSE wall, $e_L = 0.5 \cdot L - a$	ft	2.90	2.35	2.61
Limiting eccentricity, $e = L/4$ for strength limit states and $e = L/6$ for service limit state	ft	6.00	6.00	4.00
Is the resultant within limiting value of e ?	-	Yes	Yes	Yes
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	18.20	19.31	18.77
Bearing stress due to MSE wall $= \Sigma V / (L - 2e_L) = \sigma_v$	ksf	10.19	6.83	7.16
Factored bearing resistance, (q_{nf-str} for strength) or (q_{nf-ser} for service) (given)	ksf	10.50	10.50	7.50
Is factored bearing stress less than the factored bearing resistance?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) $= q_{nf} \cdot \sigma_v$	dim	1.03	1.54	1.05
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point A, M_{OA-C}	k-ft/ft	1112.03		
Resisting moments about Point A, M_{RA-C}	k-ft/ft	2321.17*		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	1209.14		
Vertical force, ΣV_C	k/ft	147.72**		
Location of resultant from Point A, $a = M_{A-C} / \Sigma V_C$	ft	8.19		
Eccentricity from center of wall, $e_L = 0.5 \cdot L - a$	ft	3.81		
Limiting eccentricity, $e = L/4$	ft	6.00		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	16.37		
Bearing stress, $\Sigma V_C / (L - 2e_L) = \sigma_{v-c}$	ksf	9.02***		
Factored bearing resistance, q_{nf-str} (given)	ksf	10.50		
Is bearing stress < factored bearing resistance?	dim	Yes		
Capacity:Demand Ratio (CDR) $= q_{nf-str} \cdot \sigma_{v-c}$	dim	1.16		

Notes: * 2321.17 = 1368.00 + 953.17; **147.72 = 108.00+39.72; ***9.02=147.72/16.37. This is to maintain consistency between the total inclined lateral force and its components.

STEP 7: EVALUATE INTERNAL STABILITY ANALYSIS OF MSE WALL

7.1 Estimate critical failure surface, variation of K_r and F^* for internal stability

For the case of inextensible steel ribbed strips, the profile of the critical failure surface, the variation of internal lateral horizontal stress coefficient, K_r , and the variation of the pullout resistance factor, F^* , are as shown in Figure E3-5 wherein other definitions such as measurement of depths Z and Z_p as well as heights H and H_1 are also shown. It should be noted that the variation of K_r and F^* are with respect to depth Z that is measured from the top of the reinforced soil zone. For the computation of K_r , the value of K_a is based on the angle of internal friction of the reinforced backfill, ϕ_r , and the assumption that the backslope angle $\beta = 0$; thus, $K_a = \tan^2(45^\circ - 34^\circ/2) = 0.283$. Hence, the value of K_r varies from $1.7(0.283) = 0.481$ at $Z = 0$ ft to $1.2(0.283) = 0.340$ at $Z = 20$ ft. For steel strips, $F^* = 1.2 + \log_{10} C_u$. Using $C_u = 7.0$ as given in Step 2, $F^* = 1.2 + \log_{10}(7.0) = 2.045 > 2.000$. Therefore, use $F^* = 2.000$.



$$\Delta H = \frac{(\tan \beta)(0.3H)}{1 - 0.3 \tan \beta} \quad H_1 = H + \Delta H$$

Z_p at start of resistant zone, $Z_{p-s} = Z + L_a \tan \beta$

Z_p at end of resistant zone, $Z_{p-e} = Z + L \tan \beta$

Use average Z_p over the resistance zone, Z_{p-ave} , for computing pullout resistance

$$Z_{p-ave} = Z + 0.5(L_a \tan \beta + L \tan \beta) = Z + 0.5 \tan \beta (L_a + L)$$

K_a is computed assuming that the backslope angle is zero, i.e., $\beta = 0$ per Article C11.10.6.2.1 of AASHTO (2007)

Figure E3-5. Geometry definition, location of critical failure surface and variation of K_r and F^* parameters for steel ribbed strips.

7.2 Establish vertical layout of soil reinforcements

Using the definition of depth Z as shown in Figure E3-5 the following vertical layout of the soil reinforcements is chosen.

$Z =$ 1.25 ft, 3.75 ft, 6.25 ft, 8.75 ft, 11.25 ft, 13.75 ft, 16.25 ft, 18.75 ft, 21.25 ft, 23.75 ft, 26.25 ft, and 28.75 ft.

The above layout leads to 12 levels of reinforcements. The vertical spacing was chosen based on a typical vertical spacing, S_v , of approximately 2.5 ft that is commonly used in the industry for steel ribbed strip reinforcement. The vertical spacing near the top and bottom of the walls are locally adjusted as necessary to fit the height of the wall.

For internal stability computations, each layer of reinforcement is assigned a tributary area, A_{trib} as follows

$$A_{trib} = (w_p)(S_{vt})$$

where w_p is the panel width of the precast facing element and S_{vt} is the vertical tributary spacing of the reinforcements based on the location of the reinforcements above and below the level of the reinforcement under consideration. The computation of S_{vt} is summarized in Table E4-7.1 wherein $S_{vt} = Z^+ - Z^-$. Note that $w_p = 5.00$ ft per Step 2.

Table E3-7.1. Summary of computations for S_{vt}

Level	Z (ft)	Z^- (ft)	Z^+ (ft)	S_{vt} (ft)
1	1.25	0	$1.25 + 0.5(3.75 - 1.25) = 2.50$	2.50
2	3.75	$3.75 - 0.5(3.75 - 1.25) = 2.50$	$3.75 + 0.5(6.25 - 3.75) = 5.00$	2.50
3	6.25	$6.25 - 0.5(6.25 - 3.75) = 5.00$	$6.25 + 0.5(8.75 - 6.25) = 7.50$	2.50
4	8.75	$8.75 - 0.5(8.75 - 6.25) = 7.50$	$8.75 + 0.5(11.25 - 8.75) = 10.00$	2.50
5	11.25	$11.25 - 0.5(11.25 - 8.75) = 10.00$	$11.25 + 0.5(13.75 - 11.25) = 12.50$	2.50
6	13.75	$13.75 - 0.5(13.75 - 11.25) = 12.50$	$13.75 + 0.5(16.25 - 13.75) = 15.00$	2.50
7	16.25	$16.25 - 0.5(16.25 - 13.75) = 15.00$	$16.25 + 0.5(18.75 - 16.25) = 17.50$	2.50
8	18.75	$18.75 - 0.5(18.75 - 16.25) = 17.50$	$18.75 + 0.5(21.25 - 18.75) = 20.00$	2.50
9	21.25	$21.25 - 0.5(21.25 - 18.75) = 20.00$	$21.25 + 0.5(23.75 - 21.25) = 22.50$	2.50
10	23.75	$23.75 - 0.5(23.75 - 21.25) = 22.50$	$23.75 + 0.5(26.25 - 23.75) = 25.00$	2.50
11	26.25	$26.25 - 0.5(26.25 - 23.75) = 25.00$	$26.25 + 0.5(28.75 - 26.25) = 27.50$	2.50
12	28.75	$28.75 - 0.5(28.75 - 26.25) = 27.50$	30.00	2.50

7.3 Calculate horizontal stress and maximum tension at each reinforcement level

The horizontal spacing of the reinforcements is based on the maximum tension ($T_{\max}$) at each level of reinforcements which requires computation of the horizontal stress, σ_H , at each reinforcement level. The reinforcement tensile and pullout resistances are then compared with $T_{\max}$ and an appropriate reinforcement pattern is adopted. This section demonstrates the calculation of horizontal stress, σ_H , and maximum tension, $T_{\max}$.

The horizontal stress, σ_H , at any depth within the MSE wall is based on only the soil load as summarized in Table E3-7.2.

$$\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}}$$

Table E3-7.2. Summary of load components leading to horizontal stress

Load Component	Load Type	Horizontal Stress
Soil load from reinforced mass, $\sigma_{v\text{-soil}}$	EV	$\sigma_{H\text{-soil}} = [K_r \sigma_{v\text{-soil}}] \gamma_{P\text{-EV}}$
Surcharge load due to backslope, σ_2	EV	$\sigma_{H\text{-surcharge}} = [K_r \sigma_2] \gamma_{P\text{-EV}}$

Using the unit weight of the reinforced soil mass and heights Z and S as shown in Figure E3-2b, the equation for horizontal stress at any depth Z within the MSE wall can be written as follows (also see Chapter 4):

$$\sigma_H = K_r (\gamma_r Z) \gamma_{P\text{-EV}} + K_r (\gamma_r S) \gamma_{P\text{-EV}} = K_r [\gamma_r (Z + S) \gamma_{P\text{-EV}}]$$

Once the horizontal stress is computed at any given level of reinforcement, the maximum tension, $T_{\max}$, is computed as follows:

$$T_{\max} = (\sigma_H)(A_{\text{trib}})$$

where A_{trib} is the tributary area for the soil reinforcement at a given level as discussed in Section 7.2

The computations for $T_{\max}$ are illustrated at $z = 8.75$ ft which is Level 4 in the assumed vertical layout of reinforcement. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E3-5.1.

- At $Z = 8.75$ ft, the following depths are computed
 $Z^- = 7.50$ ft (from Table E3-7.1)
 $Z^+ = 10.00$ ft (from Table E3-7.1)

- Obtain K_r by linear interpolation between $1.7K_a = 0.481$ at $Z = 0.00$ ft and $1.2K_a = 0.340$ at $Z = 20.00$ ft as follows:
 At $Z^- = 7.50$ ft, $K_{r(Z^-)} = 0.340 + (20.00 \text{ ft} - 7.50 \text{ ft})(0.481 - 0.340)/20.00 \text{ ft} = 0.428$
 At $Z^+ = 10.00$ ft, $K_{r(Z^+)} = 0.340 + (20.00 \text{ ft} - 10.00 \text{ ft})(0.481 - 0.340)/20.00 \text{ ft} = 0.411$
- Compute $\sigma_{H\text{-soil}} = [K_r \sigma_{v\text{-soil}}]\gamma_{P\text{-EV}}$ as follows:
 $\gamma_{P\text{-EV}} = 1.35$ from Table E3-5.1
 At $Z^- = 7.50$ ft,
 $\sigma_{v\text{-soil}(Z^-)} = (0.125 \text{ kcf})(7.50 \text{ ft}) = 0.94 \text{ ksf}$
 $\sigma_{H\text{-soil}(Z^-)} = [K_{r(Z^-)}\sigma_{v\text{-soil}(Z^-)}]\gamma_{P\text{-EV}} = (0.428)(0.94 \text{ ksf})(1.35) = 0.54 \text{ ksf}$
 At $Z^+ = 10.00$ ft,
 $\sigma_{v\text{-soil}(Z^+)} = (0.125 \text{ kcf})(10.00 \text{ ft}) = 1.25 \text{ ksf}$
 $\sigma_{H\text{-soil}(Z^+)} = [K_{r(Z^+)}\sigma_{v\text{-soil}(Z^+)}]\gamma_{P\text{-EV}} = (0.411)(1.25 \text{ ksf})(1.35) = 0.69 \text{ ksf}$
 $\sigma_{H\text{-soil}} = 0.5(0.54 \text{ ksf} + 0.69 \text{ ksf}) = 0.62 \text{ ksf}$
- Compute $\sigma_{H\text{-surcharge}} = [K_r \sigma_2]\gamma_{P\text{-EV}}$ as follows:
 $\sigma_2 = (1/2)(0.7H\tan\beta)(\gamma_f)$ (from Figure E3-2b)
 $\sigma_2 = (1/2)(0.7*30 \text{ ft})[\tan(26.56^\circ)](0.125 \text{ kcf}) = 0.656 \text{ ksf}$
 $\gamma_{P\text{-EV}} = 1.35$ from Table E3-5.1
 At $Z^- = 7.50$ ft,
 $\sigma_{H\text{-surcharge}} = [K_{r(Z^-)} \sigma_2]\gamma_{P\text{-EV}} = (0.428)(0.656 \text{ ksf})(1.35) = 0.38 \text{ ksf}$
 At $Z^+ = 10.00$ ft and $Z_p^+ = 15.29$ ft,
 $\sigma_{H\text{-surcharge}} = [K_{r(Z^+)} \sigma_2]\gamma_{P\text{-EV}} = (0.411)(0.656 \text{ ksf})(1.35) = 0.36 \text{ ksf}$
 $\sigma_{H\text{-surcharge}} = 0.5(0.38 \text{ ksf} + 0.36 \text{ ksf}) = 0.37 \text{ ksf}$
- Compute $\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}}$ as follows:
 $\sigma_H = 0.62 \text{ ksf} + 0.37 \text{ ksf} = 0.99 \text{ ksf}$
- Based on Table E3-7.1, the vertical tributary spacing at Level 4 is $S_{vt} = 2.50$ ft
- The panel width, w_p , is 5.00 ft (given in Step 1)
- The tributary area, A_{trib} , is computed as follows:
 $A_{trib} = (2.50 \text{ ft})(5.00 \text{ ft}) = 12.50 \text{ ft}^2$

- The maximum tension at Level 4 is computed as follows:

$$T_{\max} = (\sigma_h)(A_{\text{trib}}) = (0.99 \text{ ksf})(12.50 \text{ ft}^2) = 12.37 \text{ k for panel of 5-ft width}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

7.4 Establish nominal and factored long-term tensile resistance of soil reinforcement

The nominal tensile resistance of galvanized steel ribbed strip soil reinforcement is based on the design life and estimated loss of steel over the design life during corrosion.

As per Step 1, the soil reinforcement for this example is assumed to be Grade 65 ($F_y = 65$ ksi), 1.969 in. x 0.157 in. (50 mm wide x 4 mm) thick galvanized steel ribbed strips with zinc coating of 3.386 mils (86 μm). As per Step 2, the reinforced backfill meets the AASHTO (2007) requirements for electrochemical properties. For this reinforced backfill, the basis for calculating the thickness losses due to corrosion is as follows per Article 11.10.6.4.2a of AASHTO (2007):

Zinc loss = 0.58 mil for first 2 years and 0.16 mil per year thereafter

Steel loss = 0.47 mil/year/side

Based on the above corrosion rates, the following can be calculated:

$$\text{Life of zinc coating (galvanization)} = 2 \text{ years} + (3.386 - 2 \times 0.58)/0.16 \approx 16 \text{ years}$$

As per Step 1, the design life is 75 years. The base carbon steel will lose thickness for 75 years – 16 years = 59 years at a rate of 0.47 mil/year/side. Therefore, the anticipated thickness loss is calculated as follows:

$$E_R = (0.47 \text{ mil/year/side}) (59 \text{ years}) (2 \text{ sides}) = 55.46 \text{ mils} = 0.055 \text{ in.}, \text{ and}$$

$$E_C = 0.157 \text{ in.} - 0.055 \text{ in.} = 0.102 \text{ in.}$$

Based on a 1.969 wide strip, the cross-sectional area at the end of 75 years will be equal to $(1.969 \text{ in.}) (0.102 \text{ in.}) = 0.200 \text{ in}^2$.

For Grade 65 steel with $F_y = 65$ ksi, the nominal tensile resistance at end of 75 year design life will be $T_n = 65 \text{ ksi} (0.200 \text{ in}^2) = 13.00 \text{ k/strip}$. Using the resistance factor, $\phi_t = 0.75$ as listed in Table E3-5.2, the factored tensile resistance, $T_r = 13.00 \text{ k/strip} (0.75) = 9.75 \text{ k/strip}$.

7.5 Establish nominal and factored pullout resistance of soil reinforcement

The nominal pullout resistance, P_r , of galvanized steel ribbed strip soil reinforcement is based on various parameters in the following equation:

$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_v)(\gamma_{P-EV})]$$

For this example problem, the following parameters are constant at levels of reinforcements:

$$b = 1.969 \text{ in.} = 0.164 \text{ ft}$$

$\alpha = 1.0$ for inextensible reinforcement per Table 11.10.6.3.2-1 of AASHTO (2007)

The computations for P_r are illustrated at $z = 8.75$ ft which is Level 4 as measured from top of the wall. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E3-5.1.

- Compute effective (resisting) length, L_e , as follows:

Since $Z < H_1/2$, active length $L_a = 0.3(H_1)$ and $L_e = L - L_a = L - 0.3(H_1)$

$$H_1 = H + \Delta H$$

$$\Delta H = \frac{(\tan \beta)(0.3H)}{1 - 0.3 \tan \beta} = \frac{(0.5)(0.3 \times 30 \text{ ft})}{1 - 0.3(0.5)} = 5.29 \text{ ft}$$

$$H_1 = H + \Delta H = 30.00 \text{ ft} + 5.29 \text{ ft} = 35.29 \text{ ft}$$

$$\text{Active length, } L_a = 0.3(35.29 \text{ ft}) = 10.59 \text{ ft}$$

$$\text{Effective (resisting) length, } L_e = 24.00 \text{ ft} - 10.59 \text{ ft} = 13.41 \text{ ft}$$

- Compute $(\sigma_v)(\gamma_{P-EV})$

As per Figure E3-2b, $\sigma_v = \gamma_r(Z_{p-\text{ave}})$

$$Z_{p-\text{ave}} = Z + 0.5 \tan \beta (L_a + L) = 8.75 \text{ ft} + 0.5[\tan(26.56^\circ)](10.59 \text{ ft} + 24.00 \text{ ft}) = 17.40 \text{ ft}$$

Per Article 11.10.6.3.2 of AASHTO (2007), use unfactored vertical stress for pullout resistance. Thus,

$$\gamma_{P-EV} = 1.00$$

$$\sigma_v(\gamma_{P-EV}) = (0.125 \text{ kcf})(17.40 \text{ ft})(1.00) = 2.175 \text{ ksf}$$

- Obtain F^* at $Z = 8.75$ ft

Obtain F^* by linear interpolation between 2.000 at $Z = 0$ and 0.675 at $Z = 20.00$ ft as follows:

$$F^* = 0.675 + (20.00 \text{ ft} - 8.75 \text{ ft})(2.000 - 0.675)/20 \text{ ft} = 1.420$$

- Compute nominal pullout resistance as follows:

$$P_r = \alpha(F^*)(2)(b)(L_e)[(\sigma_{v-\text{soil}})(\gamma_{P-EV})]$$

$$P_r = (1.0)(1.420)(2)(0.164 \text{ ft})(13.41 \text{ ft})(2.175 \text{ ksf}) = 13.58 \text{ k/strip}$$

- Compute factored pullout resistance as follows:

$$P_{rr} = \phi P_r = (0.90)(13.58 \text{ k/strip}) = 12.23 \text{ k/strip}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

7.6 Establish number of soil reinforcing strips at each level of reinforcement

Based on T_{max} , T_r and P_{rr} , the number of strip reinforcements at any given level of reinforcements can be computed as follows:

- Based on tensile resistance considerations, the number of strip reinforcements, N_t , is computed as follows:

$$N_t = T_{max}/T_r$$

- Based on pullout resistance considerations, the number of strip reinforcements, N_p , is computed as follows:

$$N_p = T_{max}/P_{rr}$$

Using the Level 4 reinforcement at $Z = 8.75 \text{ ft}$, the number of strip reinforcements can be computed as follows:

- $T_{max} = 13.13 \text{ k}$ for panel of 5-ft width, $T_r = 9.75 \text{ k/strip}$, $P_{rr} = 12.23 \text{ k/strip}$
- $N_t = T_{max}/T_r = (13.13 \text{ k for panel of 5 ft width})/(9.75 \text{ k/strip}) = 1.35 \text{ strips for panel of 5-ft width}$
- $N_p = T_{max}/P_{rr} = (13.13 \text{ k for panel of 5 ft width})/(12.23 \text{ k/strip}) = 1.07 \text{ strips for panel of 5-ft width}$
- Since $N_t > N_p$, tension breakage is the governing criteria and therefore the governing value, N_g , is 1.35. Round up to select 2 strips at Level 4 for each panel of 5 ft width.

The computations in Sections 7.4 to 7.6 are repeated at each level of reinforcement. Table E3-7.3 presents the computations at all levels of reinforcement for Strength I (max) load

combination. The last column of Table E3-7.3 provides horizontal spacing of the reinforcing strips which is obtained by dividing the panel width, w_p , by the governing number of strips, N_g . Similar computations can be performed for Strength I (min) and Service I load combination but they will not govern the design because the load factors for these two load combinations are less than those for Strength I (max) load combination. The facing design (Step 8) may necessitate more reinforcement per level.

Note to users: All the long-form step-by-step calculations illustrated in Step 8 were based on hand-calculations in which numbers were rounded to the third or fourth significant digit as appropriate in each step. Table E3-7.3 was generated using a spreadsheet in which numbers at all calculation steps were not rounded. Thus, the end result in Table E3-7.3 may be somewhat different when compared to long-form hand calculations. However, the difference should be less than 0.2 in most cases.

Table E3-7.3. Summary of internal stability computations for Strength I (max) load combination

Level	Z ft	Z _{p-ave} dim	σ_H ksf	T _{max} k/5 ft wide panel	F* dim	L _e ft	$\phi_p(P_r)$ k/strip	$\phi_s(T_n)$ k/strip	N _p -	N _t -	N _g -	S _h ft
1	1.25	9.90	0.52	6.46	1.917	13.41	9.40	9.75	0.7	0.7	2	2.50
2	3.75	12.40	0.69	8.63	1.751	13.41	10.76	9.75	0.8	0.9	2	2.50
3	6.25	14.90	0.85	10.58	1.586	13.41	11.70	9.75	0.9	1.1	2	2.50
4	8.75	17.40	0.99	12.35	1.420	13.41	12.23	9.75	1.0	1.3	2	2.50
5	11.25	19.90	1.12	13.96	1.254	13.41	12.36	9.75	1.1	1.4	2	2.50
6	13.75	22.19	1.23	15.40	1.089	14.25	12.70	9.75	1.2	1.6	2	2.50
7	16.25	24.31	1.33	16.59	0.923	15.75	13.05	9.75	1.3	1.7	2	2.50
8	18.75	26.44	1.41	17.60	0.757	17.25	12.76	9.75	1.4	1.8	2	2.50
9	21.25	28.56	1.52	18.98	0.675	18.75	13.33	9.75	1.4	1.9	3	1.67
10	23.75	30.69	1.66	20.77	0.675	20.25	15.50	9.75	1.3	2.1	3	1.67
11	26.25	32.81	1.81	22.56	0.675	21.75	17.79	9.75	1.3	2.3	3	1.67
12	28.75	34.94	1.95	24.36	0.675	23.25	20.24	9.75	1.2	2.5	3	1.67
Note 1: Based on pullout and tension breakage considerations only 1 strip is required at Level 1 and 2. However, a minimum 2 strips at a horizontal spacing not exceeding 2.5 ft should be provided as per the criteria in Chapter 4.												

STEP 8: DESIGN OF FACING ELEMENTS

The precast facing elements must be designed as structural elements with appropriate connection strength as discussed in Chapter 4. Depending on the design of the facing panel, the number of strips at each level may have to be increased.

STEP 9: CHECK OVERALL AND COMPOUND STABILITY AT SERVICE LIMIT STATE

From Step 2, it is given that the foundation soil is dense clayey sand that has $\phi_f = 30^\circ$, $\gamma_f = 125$ pcf. Furthermore, the ground in front of the wall is horizontal and the foundation soil has no water table. Therefore, based on observation, overall stability is adequate. For actual projects, overall stability should be investigated at the Service I load combination and a resistance factor of 0.65.

STEP 10: DESIGN WALL DRAINAGE SYSTEMS

Drains are detailed on construction drawings. For a MSE wall with sloping backfill, the drainage system for the MSE wall must be carefully integrated with the other hillside drain systems as appropriate.

EXAMPLE E4

SEGMENTAL PRECAST PANEL MSE WALL WITH LEVEL BACKFILL AND LIVE LOAD SURCHARGE

E4-1 INTRODUCTION

This example problem demonstrates the analysis of a MSE wall with a level backfill and live load surcharge. The MSE wall is assumed to include a segmental precast panel face with steel grid (bar mat) reinforcements. The MSE wall configuration to be analyzed is shown in Figure E4-1. The analysis is based on various principles that were discussed in Chapter 4. Table E4-1 presents a summary of steps involved in the analysis. Each of the steps and sub-steps is sequential and if the design is revised at any step or sub-step then all the previous computations need to be re-visited. Each of the steps and the sub-steps in Table E4-1 is explained in detail herein. Practical considerations are presented in Section E4-2 after the illustration of the step-by-step procedures.

Table E4-1. Summary of steps in analysis of MSE wall with level backfill and live load surcharge

Step	Item
1	Establish project requirements
2	Establish project parameters
3	Estimate wall embedment depth and length of reinforcement
4	Estimate unfactored loads
5	Summarize applicable load and resistance factors
6	Evaluate external stability of MSE wall 6.1 Evaluation of sliding resistance 6.2 Evaluation of limiting eccentricity 6.3 Evaluation of bearing resistance 6.4 Settlement analysis
7	Evaluate internal stability of MSE wall 7.1 Estimate critical failure surface, variation of K_r and F^* for internal stability 7.2 Establish vertical layout of soil reinforcements 7.3 Calculate horizontal stress and maximum tension at each reinforcement level 7.4 Establish nominal and factored long-term tensile resistance of soil reinforcement 7.5 Establish nominal and factored pullout resistance of soil reinforcement 7.6 Establish number of soil reinforcing elements at each level of reinforcement
8	Design of facing elements
9	Check overall and compound stability at the service limit state.
10	Design wall drainage system

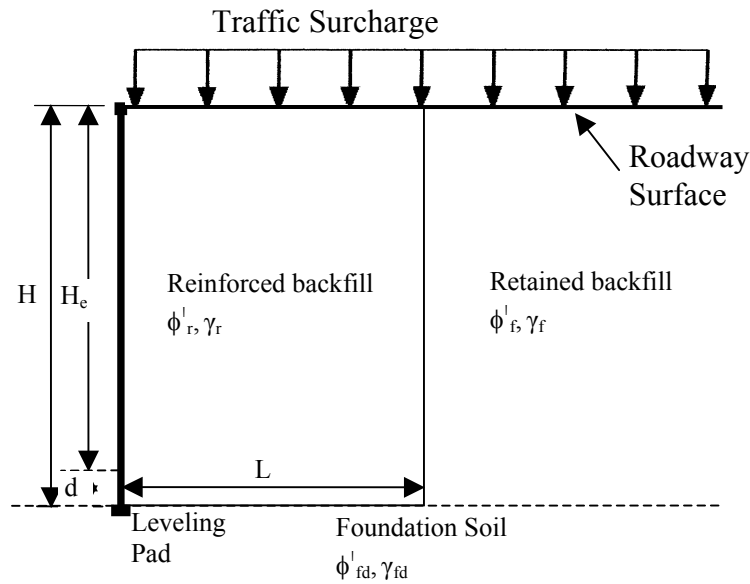


Figure E4-1. Configuration showing various parameters for analysis of a MSE wall with level backfill and live load surcharge (not-to-scale).

STEP 1. ESTABLISH PROJECT REQUIREMENTS

- Exposed wall height, $H_e = 23.64$ ft
- Length of wall = 850 ft
- Design life = 75 years
- Precast panel units: 5 ft wide x 5 ft tall x 0.5 ft thick
- Type of reinforcement: Grade 65 ($F_y = 65$ ksi), steel bar mat with W15 and W11 wires. Assume wires to be galvanized with zinc coating of 3.386 mils (86 μm).
- No seismic considerations

STEP 2. EVALUATE PROJECT PARAMETERS

- Reinforced backfill, $\phi_r^l = 34^\circ$, $\gamma_r = 125$ pcf, coefficient of uniformity, $C_u = 7.0$ and meeting the AASHTO (2007) requirements for electrochemical properties
- Retained backfill, $\phi_f^l = 30^\circ$, $\gamma_f = 125$ pcf
- Foundation soil, dense clayey sand with $\phi_{fd}^l = 30^\circ$, $\gamma_{fd} = 125$ pcf
- Factored Bearing resistance of foundation soil
 - For service limit consideration, $q_{nf-ser} = 7.50$ ksf for 1-inch of total settlement
 - For strength limit consideration, $q_{nf-str} = 10.50$ ksf
- Live load surcharge, $h_{eq} = 2$ ft of soil per Table 3.11.6.4-2 of AASHTO (2007)

STEP 3. ESTIMATE DEPTH OF EMBEDMENT AND LENGTH OF REINFORCEMENT

Based on Table C.11.10.2.2.-1 of AASHTO (2007), the minimum embedment depth = $H/20$ for walls with horizontal ground in front of wall, i.e., 1.2 ft for exposed wall height of 23.64 ft. For this design, assume embedment, $d = 2.0$ ft. Thus, design height of the wall, $H = H_e + d = 23.64 \text{ ft} + 2.0 \text{ ft} = 25.64 \text{ ft}$.

Due to the level backfill, the minimum initial length of reinforcement is assumed to be $0.7H$ or 18 ft. This length will be verified as part of the design process. The length of the reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone because differential settlements could overstress the reinforcements.

STEP 4. ESTIMATE UNFACTORED LOADS

Tables E4-4.1 and E4-4.2 present the equations for unfactored loads and moment arms about Point A shown in Figure E4-2. The moments are a product of the respective forces and moment arms. Each force is assigned a designation representing the applicable load type as per Tables 3.4.1-1 and 3.4.1-2 of AASHTO (2007).

To compute the numerical values of various forces and moments, the parameters provided in Step 2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure are as follows:

$$K_{ar} = (1 - \sin 34^\circ) / (1 + \sin 34^\circ) = 0.283$$

$$K_{af} = (1 - \sin 30^\circ) / (1 + \sin 30^\circ) = 0.333$$

For the example problem, Tables E4-4.3 and E4-4.4 summarize the numerical values unfactored forces and moments, respectively, based on the equations, various dimensions and values of lateral earth pressure coefficients presented above. Refer to Figure E4-2 for notations of various forces.

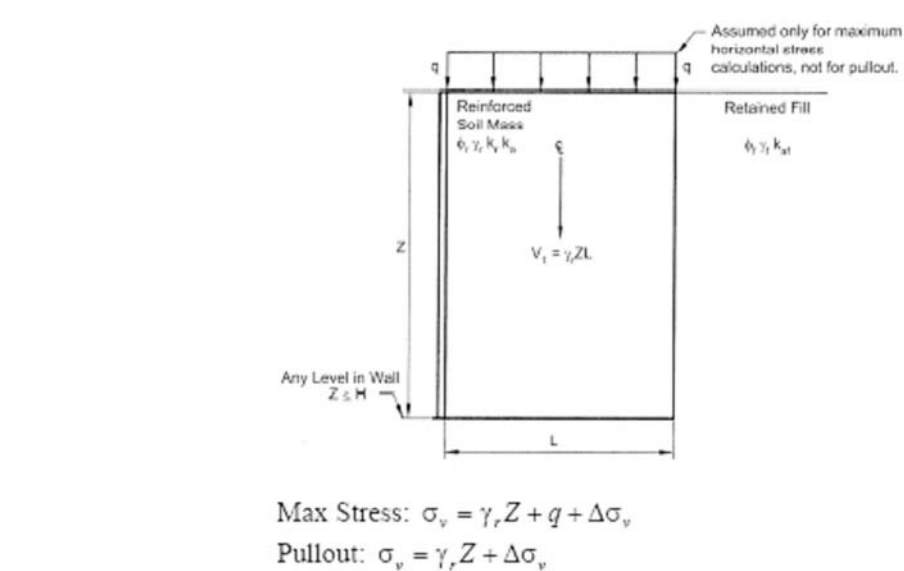
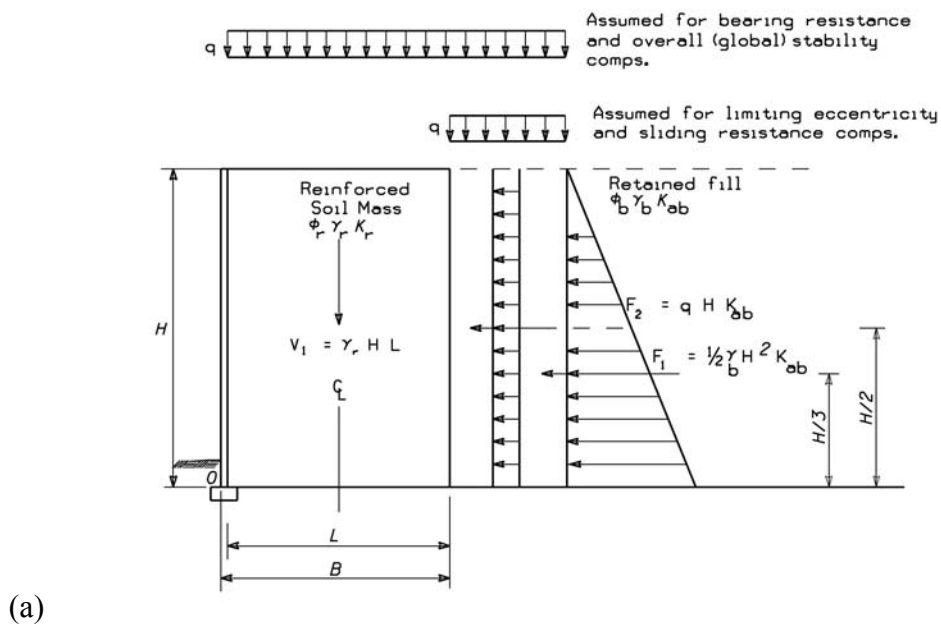


Figure E4-2. Legend for computation of forces and moments (not-to-scale).

The unfactored forces and moments in Tables E4-4.3 and E4-4.4 form the basis of all computations in this example problem. The unfactored forces and moments should be multiplied by the appropriate load factors based on the load types identified in the second column of the Tables E4-4.1 and E4-4.2 to perform the analysis for various load combinations such as Strength I, Service I, etc.

The load factors for various load types relevant to this example problem are discussed in Step 5.

Table E4-4.1. Equations of computing unfactored vertical forces and moments

Vertical Force (Force/length units)	LRFD Load Type	Moment arm (Length units)
		@ Point A
$V_1 = (\gamma_r)(H)(L)$	EV	L/2
$V_s = (\gamma_f)(h_{eq})(L) = (q)L$	LL	L/2
Note: h_{eq} is the equivalent height of soil such that $q = (\gamma_f)(h_{eq})$		

Table E4-4.2. Equations of computing unfactored horizontal forces and moments

Horizontal Force (Force/length)	LRFD Load Type	Moment arm (Length units)
		@ Point A
$F_1 = \frac{1}{2}(K_{af})(\gamma_f)H^2$	EH	H/3
$F_2 = (K_{af})[(\gamma_f)(h_{eq})](H)$	LL	H/2

Table E4-4.3. Unfactored vertical forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment	Moment at Point A, k-ft/ft
$V_1 =$	57.69	9.00	$MV_1 =$	519.21
$V_s =$	4.50	9.00	$MV_s =$	40.50
Note: V_s is based on h_{eq} of 2 ft per Table 3.11.6.4-1 of AASHTO (2007).				

Table E4-4.4. Unfactored horizontal forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment	Moment at Point A, k-ft/ft
$F_1 =$	13.68	8.55	$MF_1 =$	116.94
$F_2 =$	2.13	12.82	$MF_2 =$	27.36

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table E4-5.1 summarizes the load factors for the various LRFD load type shown in second column of Tables E4-4.1 and E4-4.2. **Throughout the computations in this example problem, the forces and moments in Tables E4-4.1 and E4-4.2 should be multiplied by appropriate load factors.** For example, if computations are being done for Strength I (maximum) load combination, the forces and moments corresponding to load V_1 should be multiplied by 1.35 which is associated with load type EV assigned to load V_1 .

Table E4-5.1. Summary of applicable load factors

Load Combination	Load Factors (after AASHTO, 2007 Tables 3.4.1-1 and 3.4.1-2)		
	EV	EH	LL
Strength I (maximum)	1.35	1.50	1.75
Strength I (minimum)	1.00	0.90	1.75
Service I	1.00	1.00	1.00

For computation of factored resistances during evaluation of strength limits states, appropriate resistance factors have to be used. Table E4-5.2 summarizes the applicable resistance factors. For service limit state, all resistance factors are equal to 1.0.

Table E4-5.2. Summary of applicable resistance factors for evaluation of resistances

Item	Resistance Factors	AASHTO (2007) Reference
Sliding of MSE wall on foundation soil	$\phi_s = 1.00$	Table 11.5.6-1
Bearing resistance	$\phi_b = 0.65$	Table 11.5.6-1
Tensile resistance (for steel bar mats)	$\phi_t = 0.65$	Table 11.5.6-1
Pullout resistance	$\phi_p = 0.90$	Table 11.5.6-1

STEP 6. EVALUATE EXTERNAL STABILITY OF MSE WALL

The external stability of MSE wall is a function of the various forces and moments shown in Figure E4-2. In the LRFD context the forces and moments need to be categorized into various load types. The primary load types for this example problem are soil loads (EV, EH) and live load (LL).

6.1 Sliding Resistance at Base of MSE Wall

The purpose of these computations is to evaluate the sliding resistance at the base of the MSE wall. Since the computations are related to sliding resistance, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for sliding resistance at the base of the MSE wall are illustrated in Table E4-6.1. Note that sliding resistance is a strength limit state check and therefore service limit state calculations are not performed. Since the friction angle of foundation soil, ϕ'_{fd} , is less than the friction angle for reinforced soil, ϕ'_r , the sliding check will be performed using ϕ'_{fd} . The critical values based on max/min result in the extreme force effect and govern the sliding mode of failure.

Table E4-6.1. Computations for evaluation of sliding resistance of MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Lateral load on the MSE wall, $H_m = F_1 + F_2$	k/ft	24.26	16.05	NA
Vertical load at base of MSE wall without LL surcharge = V_1	k/ft	77.88	57.69	NA
Nominal sliding resistance at base of MSE wall, $V_{Nm} = \tan(\phi'_{fd})(V_1)$	k/ft	44.96	33.31	NA
Sliding resistance at base of MSE wall, $V_{Fm} = \phi_s * V_{Nm}$	k/ft	44.96	33.31	NA
Is $V_{Fm} > H_m$?	-	Yes	Yes	NA
Capacity:Demand Ratio (CDR) = $V_{Fm} : H_m$	dim	1.85	2.08	NA
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_{Fm} (V_{Fmmin})	k/ft	33.31		
Maximum H_m (H_{mmax})	k/ft	24.26		
Is $V_{Fmmin} > H_{mmax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = $V_{Fmmin} : H_{mmax}$	dim	1.37		

6.2 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table E4-6.2. Limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. The critical values based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table E4-6.2. Computations for evaluation of limiting eccentricity for MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Total vertical load at base of MSE wall without LL, $V_A = V_1$	k/ft	77.88	57.69	N/A
Resisting moments about Point A without LL surcharge= $M_{RA} = MV_1$	k-ft/ft	700.93	519.21	N/A
Overturning moments about Point A = $M_{OA} =$ $MF_1 + MF_2$	k-ft/ft	223.30	153.13	N/A
Net moment about Point A = $M_A = M_{RA} - M_{OA}$	k-ft/ft	477.64	366.08	N/A
Location of the resultant force on base of MSE wall from Point A, $a = M_A/V_A$	ft	6.13	6.35	N/A
Eccentricity at base of MSE wall, $e_L = L/2 - a$	ft	2.87	2.65	N/A
Limiting eccentricity, $e = L/4$ for strength limit state	ft	4.50	4.50	N/A
Is the resultant within limiting value of e ?	-	Yes	Yes	N/A
Calculated e_L/L	-	0.16	0.15	N/A
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point A, M_{OA-C}	k-ft/ft	223.30		
Resisting moments about Point A, M_{RA-C}	k-ft/ft	519.21*		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	295.91		
Vertical force, V_{A-C}	k/ft	57.69*		
Location of resultant from Point A, $a_{nl} = M_{A-C}/V_{A-C}$	ft	5.13		
Eccentricity from center of wall base, $e_L = 0.5*L - a_{nl}$	ft	3.87		
Limiting eccentricity, $e = L/4$	ft	4.50		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	10.26		
Calculated e_L/L	-	0.22		

Note: *519.21 and 57.69 are consistent values based on the mass of reinforced soil block

6.3 Bearing Resistance at base of MSE Wall

For bearing resistance computations, the effect of live load is included since it creates larger bearing stresses. The bearing stress at the base of the MSE wall can be computed as follows:

$$\sigma_v = \frac{\Sigma V}{L - 2e_L}$$

where $\Sigma V = R = V_1 + V_s$ is the resultant of vertical forces and the load eccentricity e_L is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

In LRFD, σ_v is compared with the factored bearing resistance when computed for strength limit state and used for settlement analysis when computed for service limit state. The

various computations for evaluation of bearing resistance are presented in Table E4-6.3. The Strength I (max) load combination results in the extreme force effect in terms of maximum bearing stress and therefore governs the bearing resistance mode of failure. The Service I load combination is evaluated to compute the bearing stress for settlement analysis.

Table E4-6.3. Computations for evaluation of bearing resistance for MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Vertical load at base of MSE wall including LL on top, $\Sigma V = R = V_1 + V_S$	k/ft	85.76	65.57	62.19
Resisting moments @ Point A on the MSE wall, $M_{RA} = MV_1 + MV_S$	k-ft/ft	771.81	590.09	559.71
Overturning moments @ Point A on the MSE wall, $M_{OA} = MF_1 + MF_2$	k-ft/ft	223.30	153.13	144.30
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft/ft	548.51	436.95	415.41
Location of Resultant from Point A, $a = M_A / \Sigma V$	ft	6.40	6.66	6.68
Eccentricity from center of wall base, $e_L = 0.5 * L - a$	ft	2.60	2.34	2.32
Limiting eccentricity, $e = L/4$ for strength limit states and $e = L/6$ for service limit state	ft	4.50	4.50	3.00
Is the resultant within limiting value of e_L ?	-	Yes	Yes	Yes
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	12.79	13.33	13.36
Bearing stress due to MSE wall $= \Sigma V / (L - 2e_L) = \sigma_v$	ksf	6.70	4.92	4.66
Bearing resistance, (q_{nf-str} for strength) or (q_{nf-ser} for service) (given)	ksf	10.50	10.50	7.50
Is bearing stress less than the bearing resistance?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) $= q_{nf} \cdot \sigma_v$	dim	1.57	2.13	1.61
CRITICAL VALUES BASED ON MAX/MIN				
Resisting moments about Point A, M_{RA-C}	k-ft/ft	590.09*		
Overturning moments about Point A, M_{OA-C}	k-ft/ft	223.30		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	366.79		
Vertical force, ΣV_C	k/ft	65.57*		
Location of resultant from Point A, $a = M_{A-C} / \Sigma V_C$	ft	5.59		
Eccentricity from center of wall base, $e_L = 0.5 * L - a$	ft	3.41		
Limiting eccentricity, $e = L/4$	ft	4.50		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	11.19		
Bearing stress, $\Sigma V_C / (L - 2e_L) = \sigma_{v-c}$	ksf	5.86		
Bearing resistance, q_{nf-str} (given)	ksf	10.50		
Is bearing stress < bearing resistance?	dim	Yes		
Capacity:Demand Ratio (CDR) $= q_{nf-str} \cdot \sigma_{v-c}$	dim	1.79		

Note: *590.09 and 65.57 are consistent values based on the mass of reinforced soil block

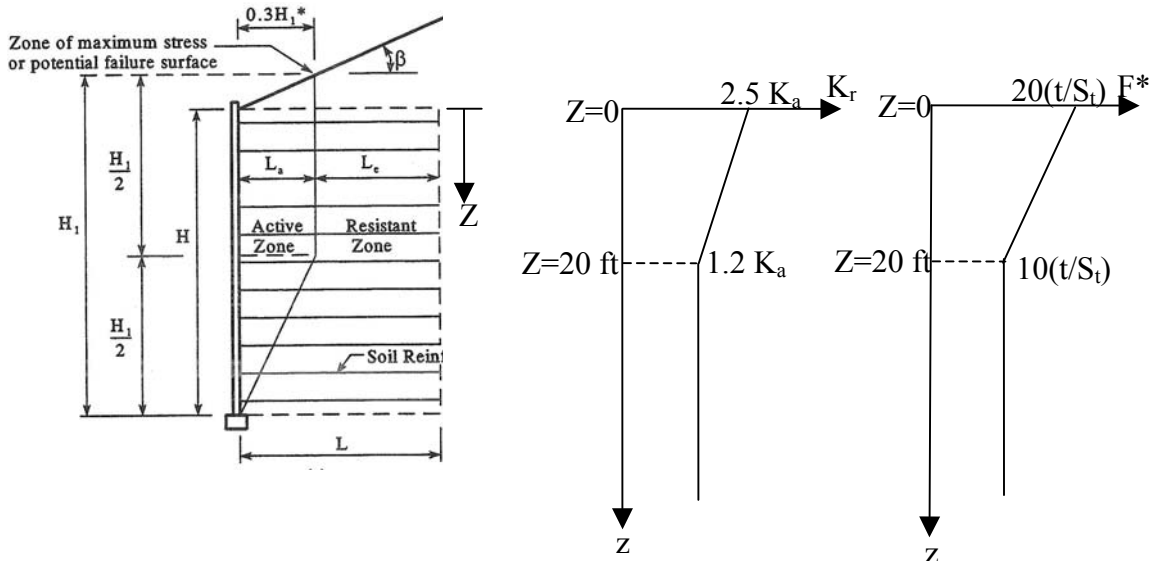
6.4 Settlement Analysis

Settlement is evaluated at Service I Limit State. From Step 2, the estimated settlement under a bearing stress of 7.50 ksf is 1.00 in. From Table E1-6.3, the bearing stress for Service I limit state is 4.66 ksf. Therefore, the settlement will be less than 1.00 in.

STEP 7: EVALUATE INTERNAL STABILITY ANALYSIS OF MSE WALL

7.1 Estimate critical failure surface, variation of K_r and F^* for internal stability

For the case of inextensible steel bar mats, the profile of the critical failure surface, the variation of internal lateral horizontal stress coefficient, K_r , and the variation of the pullout resistance factor, F^* , are as shown in Figure E4-5 wherein other definitions such as measurement of depth, Z , and height, H , are also shown. It should be noted that the variation of K_r and F^* are with respect to depth Z that is measured from top of the reinforced soil zone. The value of K_a is based on the angle of internal friction of the reinforced backfill, ϕ_r , which is equal to $K_a = 0.283$ calculated in Step 4. Thus, the value of K_r varies from $2.5(0.283) = 0.707$ at $Z=0$ to $1.2(0.283) = 0.340$ at $Z=20$ ft. The value of F^* is a function of the transverse wire configuration and is calculated later.



Note: In this example problem, the backfill is level, i.e., $\beta=0$. Therefore, the K_r and F^* profiles start at $Z = 0$ where Z is the depth below the top of the reinforced soil zone as shown in the figure.

Figure E4-5. Geometry definition, location of critical failure surface and variation of K_r and F^* parameters for steel bar mats.

7.2 Establish vertical layout of soil reinforcements

Using the definition of depth Z as shown in Figure E4-5 the following vertical layout of the soil reinforcements is chosen.

$Z = 1.87 \text{ ft}, 4.37 \text{ ft}, 6.87 \text{ ft}, 9.37 \text{ ft}, 11.87 \text{ ft}, 14.37 \text{ ft}, 16.87 \text{ ft}, 19.37 \text{ ft}, 21.87 \text{ ft}, \text{ and } 24.37 \text{ ft}$

The above layout leads to 10 levels of reinforcements. The vertical spacing was chosen based on a typical vertical spacing, S_v , of approximately 2.5 ft that is commonly used in the industry for steel grid (bar mat) reinforcement. The vertical spacing near the top and bottom of the walls are locally adjusted as necessary to fit the height of the wall.

For internal stability computations, each layer of reinforcement is assigned a tributary area, A_{trib} as follows

$$A_{\text{trib}} = (w_p)(S_{vt})$$

where w_p is the panel width of the precast facing element and S_{vt} is the vertical tributary spacing of the reinforcements based on the location of the reinforcements above and below the level of the reinforcement under consideration. The computation of S_{vt} is summarized in Table E4-7.1 wherein $S_{vt} = z^+ - z^-$. Note that $w_p = 5.00 \text{ ft}$ per Step 1.

Table E4-7.1. Summary of computations for S_{vt}

Level	Z (ft)	Z^- (ft)	Z^+ (ft)	S_{vt} (ft)
1	1.87	0	$1.87+0.5(4.37-1.87)=3.12$	3.12
2	4.37	$4.37-0.5(4.37-1.87)=3.12$	$4.37+0.5(6.87-4.37)=5.62$	2.50
3	6.87	$6.87-0.5(6.87-4.37)=5.62$	$6.87+0.5(9.37-6.87)=8.12$	2.50
4	9.37	$9.37-0.5(9.37-6.87)=8.12$	$9.37+0.5(11.87-9.37)=10.62$	2.50
5	11.87	$11.87-0.5(11.87-9.37)=10.62$	$11.87+0.5(14.37-11.87)=13.12$	2.50
6	14.37	$14.37-0.5(14.37-11.87)=13.12$	$14.37+0.5(16.87-14.37)=15.62$	2.50
7	16.87	$16.87-0.5(16.87-14.37)=15.62$	$16.87+0.5(19.37-16.87)=18.12$	2.50
8	19.37	$19.37-0.5(19.37-16.87)=18.12$	$19.37+0.5(21.87-19.37)=20.62$	2.50
9	21.87	$21.87-0.5(21.87-19.37)=20.62$	$21.87+0.5(24.37-21.87)=23.12$	2.50
10	24.37	$24.37-0.5(24.37-21.87)=23.12$	25.64	2.52

7.3 Calculate horizontal stress and maximum tension at each reinforcement level

The horizontal spacing of the reinforcements is based on the maximum tension ($T_{\max}$) at each level of reinforcements which requires computation of the horizontal stress, σ_H , at each reinforcement level. The reinforcement tensile and pullout resistances are then compared with $T_{\max}$ and an appropriate reinforcement pattern is adopted. This section demonstrates the calculation of horizontal stress, σ_H , and maximum tension, $T_{\max}$.

The horizontal stress, σ_H , at any depth within the MSE wall is based on only the soil load as summarized in Table E4-7.2.

$$\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}}$$

Table E4-7.2. Summary of load components leading to horizontal stress

Load Component	Load Type	Horizontal Stress
Soil load from reinforced mass, $\sigma_{v\text{-soil}}$	EV	$\sigma_{H\text{-soil}} = [K_r \sigma_{v\text{-soil}}] \gamma_{P\text{-EV}}$
Surcharge traffic live load, q	EV	$\sigma_{H\text{-surcharge}} = [K_r q] \gamma_{P\text{-EV}}$

Using the unit weight of the reinforced soil mass and heights Z and h_{eq} , the equation for horizontal stress at any depth Z within the MSE wall can be written as follows (also see Chapter 4):

$$\sigma_H = K_r (\gamma_r Z) \gamma_{P\text{-EV}} + K_r (\gamma_r h_{eq}) \gamma_{P\text{-EV}} = K_r [\gamma_r (Z + h_{eq}) \gamma_{P\text{-EV}}]$$

Once the horizontal stress is computed at any given level of reinforcement, the maximum tension, $T_{\max}$, is computed as follows:

$$T_{\max} = (\sigma_H)(A_{\text{trib}})$$

where A_{trib} is the tributary area for the soil reinforcement at a given level as discussed in Section 7.2.

The computations for $T_{\max}$ are illustrated at $z_o = 9.37$ ft which is Level 4 in the assumed vertical layout of reinforcement. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E4-5.1.

- At $Z = 9.37$ ft, the following depths are computed
 $Z^- = 8.12$ ft (from Table E4-7.1)
 $Z^+ = 10.62$ ft (from Table E4-7.1)

- Obtain K_r by linear interpolation between $1.7K_a = 0.707$ at $Z = 0$ and $1.2K_a = 0.340$ at $Z = 20.00$ ft as follows:

$$\text{At } Z^- = 8.12 \text{ ft, } K_{r(z^-)} = 0.340 + (20.00 \text{ ft} - 8.12 \text{ ft})(0.707 - 0.340)/20.00 \text{ ft} = 0.558$$

$$\text{At } Z^+ = 10.62 \text{ ft, } K_{r(z^+)} = 0.340 + (20.00 \text{ ft} - 10.62 \text{ ft})(0.707 - 0.340)/20.00 \text{ ft} = 0.512$$

- Compute $\sigma_{H\text{-soil}} = [k_r \sigma_{v\text{-soil}}] \gamma_{P\text{-EV}}$ as follows:

$$\gamma_{P\text{-EV}} = 1.35 \text{ from Table E4-5.1}$$

$$\text{At } Z^- = 8.12 \text{ ft,}$$

$$\sigma_{v\text{-soil}(Z^-)} = (0.125 \text{ kcf})(8.12 \text{ ft}) = 1.02 \text{ ksf}$$

$$\sigma_{H\text{-soil}(Z^-)} = [K_{r(z^-)} \sigma_{v\text{-soil}(z^-)}] \gamma_{P\text{-EV}} = (0.558)(1.02 \text{ ksf})(1.35) = 0.76 \text{ ksf}$$

$$\text{At } Z^+ = 10.62 \text{ ft,}$$

$$\sigma_{v\text{-soil}(Z^+)} = (0.125 \text{ kcf})(10.62 \text{ ft}) = 1.33 \text{ ksf}$$

$$\sigma_{H\text{-soil}(Z^+)} = [K_{r(z^+)} \sigma_{v\text{-soil}(z^+)}] \gamma_{P\text{-EV}} = (0.512)(1.33 \text{ ksf})(1.35) = 0.92 \text{ ksf}$$

$$\sigma_{H\text{-soil}} = 0.5(0.76 \text{ ksf} + 0.92 \text{ ksf}) = 0.84 \text{ ksf}$$

- Compute $\sigma_{H\text{-surcharge}} = [K_r q] \gamma_{P\text{-EV}}$ as follows:

$$\gamma_{P\text{-EV}} = 1.35 \text{ from Table E4-5.1}$$

$$q = (\gamma_f)(h_{eq}) = (0.125 \text{ kcf})(2.00 \text{ ft}) = 0.25 \text{ ksf}$$

$$\text{At } Z^- = 8.12 \text{ ft, } \sigma_{H\text{-surcharge}(Z^-)} = [K_{r(z^-)} q] \gamma_{P\text{-EV}} = (0.558)(0.25 \text{ ksf})(1.35) = 0.19 \text{ ksf}$$

$$\text{At } Z^+ = 10.62 \text{ ft, } \sigma_{H\text{-surcharge}(Z^+)} = [K_{r(z^+)} q] \gamma_{P\text{-EV}} = (0.512)(0.25 \text{ ksf})(1.35) = 0.17 \text{ ksf}$$

$$\sigma_{H\text{-surcharge}} = 0.5(0.19 \text{ ksf} + 0.17 \text{ ksf}) = 0.18 \text{ ksf}$$

- Compute $\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}}$ as follows:

$$\sigma_H = 0.84 \text{ ksf} + 0.18 \text{ ksf} = 1.02 \text{ ksf}$$

- Based on Table E4-7.1, the vertical tributary spacing at Level 4 is $S_{vt} = 2.50$ ft

- The panel width, w_p , is 5.00 ft (given in Step 1)

- The tributary area, A_{trib} , is computed as follows:

$$A_{trib} = (2.50 \text{ ft})(5.00 \text{ ft}) = 12.50 \text{ ft}^2$$

- The maximum tension at Level 4 is computed as follows:

$$T_{max} = (\sigma_h)(A_{trib}) = (1.02 \text{ ksf})(12.50 \text{ ft}^2) = 12.75 \text{ k/panel of 5-ft width}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

7.4 Establish nominal and factored long-term tensile resistance of soil reinforcement

The nominal tensile resistance of galvanized steel bar mat soil reinforcement is based on the design life and estimated loss of steel over the design life during corrosion.

The nominal diameters, D , of W15 and W11 wires are as follows:

For W15 wires, $D = 0.437 \text{ in.} = 11.10 \text{ mm} = 11,100 \mu\text{m}$

For W11 wires, $D = 0.374 \text{ in.} = 9.50 \text{ mm} = 9,500 \mu\text{m}$

As per Step 2, the reinforced backfill meets the AASHTO (2007) requirements for electrochemical properties. For this reinforced backfill, the basis for calculating the thickness losses due to corrosion is as follows per Article 11.10.6.4.2a of AASHTO (2007):

Zinc loss = 0.58 mil for first 2 years and 0.16 mil per year thereafter

Steel loss = 0.16 mil/year/side

Based on the above corrosion rates, the following can be calculated:

$$\text{Life of zinc coating (galvanization)} = 2 \text{ years} + (3.386 - 2 \times 0.58) / 0.16 \approx 16 \text{ years}$$

As per Step 1, the design life is 75 years. The base carbon steel will lose thickness for 75 years – 16 years = 59 years at a rate of 0.47 mil/year/side. Therefore, the anticipated diameter and area after 75 years for W15 and W11 is calculated as follows:

For W15 wires

$$D_{75} = 0.437 \text{ in.} - (0.47 \text{ mil/year/side}) (59 \text{ years}) (2 \text{ sides}) / 1000 = 0.437 \text{ in.} - 0.056 \text{ in.}$$

$$D_{75} = 0.381 \text{ in.}$$

Based on a 0.381 in. diameter wire, the cross-sectional area at the end of 75 years will be equal to $(\pi)(0.381 \text{ in.})^2/4 = 0.1142 \text{ in}^2/\text{wire}$

For W11 wires

$$D_{75} = 0.374 \text{ in.} - (0.47 \text{ mil/year/side}) (59 \text{ years}) (2 \text{ sides}) = 0.374 \text{ in.} - 0.056 \text{ in.}$$

$$D_{75} = 0.318 \text{ in.}$$

Based on a 0.318 in. diameter wire, the cross-sectional area at the end of 75 years will be equal to $(\pi)(0.318 \text{ in.})^2/4 = 0.0795 \text{ in}^2/\text{wire}$

For Grade 65 steel with $F_y = 65 \text{ ksi}$, the nominal tensile resistance, T_n , and factored tensile resistance, T_r , for the W15 and W11 wires will be as follows:

For W15 wires

$$T_n = 65 \text{ ksi} (0.1142 \text{ in}^2) = 7.42 \text{ k/wire.}$$

Using the resistance factor, $\phi_t = 0.65$ as listed in Table E4-5.2, the factored tensile resistance, $T_r = 7.42 \text{ k/wire} (0.65) = 4.82 \text{ k/wire}$.

For W11 wires

$$T_n = 65 \text{ ksi} (0.0795 \text{ in}^2) = 5.17 \text{ k/wire.}$$

Using the resistance factor, $\phi_t = 0.65$ as listed in Table E4-5.2, the factored tensile resistance, $T_r = 5.17 \text{ k/wire} (0.65) = 3.36 \text{ k/wire}$.

7.5 Establish nominal and factored pullout resistance of soil reinforcement

The nominal pullout resistance, P_r , of galvanized steel bar mat (grid) reinforcement is based on various parameters in the following equation:

$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}})]$$

In the above equation, the contribution of live load is not included as per Figure E4-2b. Since the steel bar mat has welded connections, it can be considered inextensible with $\alpha = 1$.

Assume a W11 transverse wire which has a nominal diameter of 0.374 in. The transverse spacing of transverse wires, S_t , is varied depending on the level of reinforcement to optimize the design from an economical perspective. For this example problem, assume that the spacing of the transverse wires, $S_t = 6 \text{ in.}$, 12 in. and 18 in. Based on these spacing, the value of t/S_t is as follows:

$$\text{For, } S_t = 6 \text{ in., } t/S_t = 0.374 \text{ in.}/6 \text{ in.} = 0.0623$$

$$\text{For, } S_t = 12 \text{ in., } t/S_t = 0.374 \text{ in.}/12 \text{ in.} = 0.0312$$

$$\text{For, } S_t = 18 \text{ in., } t/S_t = 0.374 \text{ in.}/18 \text{ in.} = 0.0208$$

Based on the value of t/S_t , the F^* parameter varies from $20(t/S_t)$ at $z = 0$ ft to $10(t/S_t)$ at $z \geq 20$ ft and greater as shown in Figure E4-5. For the three value of t/S_t the variation of F^* is as follows:

For $t/S_t = 0.0623$, $F^* = 1.2460$ at $Z = 0$ ft and $F^* = 0.623$ at $Z \geq 20$ ft

For $t/S_t = 0.0312$, $F^* = 0.623$ at $Z = 0$ ft and $F^* = 0.312$ at $Z \geq 20$ ft

For $t/S_t = 0.0208$, $F^* = 0.416$ at $Z = 0$ ft and $F^* = 0.208$ at $Z \geq 20$ ft

Assume bar mat width, $b = 1$ ft for computing pullout resistance on a per foot width basis. The actual bar mat width will be computed based on comparison of the pullout resistance with $T_{\max}$.

For this example problem, assume the layout of longitudinal and transverse wires as shown in Table E4-7.3. The number of longitudinal wires and thus the width of the bar mats will be determined in Section 7.6.

Table E4-7.3. Assumed bar mat configuration for internal stability analysis

Level	Longitudinal wire	Transverse wire	Spacing of transverse wires, S_t
1 to 4	W11	W11	6 in.
5 to 7	W15	W11	12 in.
8 to 10	W15	W11	18 in.

The computations for P_r are illustrated at $z = 9.37$ ft which is Level 4 as measured from top of the wall. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E4-5.1.

- Obtain F^* at $z = 9.37$ ft by linear interpolation between 1.2460 at $Z = 0$ and 0.623 at $Z = 20$ ft as follows:

$$F^* = 0.623 + (20.00 \text{ ft} - 9.37 \text{ ft})(1.246 - 0.623)/20 \text{ ft} = 0.955$$

- Compute effective length L_e as follows:

$$\text{Since } Z < H/2, L_e = L - 0.3(H)$$

$$L_e = 18 \text{ ft} - 0.3(25.64 \text{ ft}) = 10.31 \text{ ft}$$

- Compute $(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}})$

Per Article 11.10.6.3.2 of AASHTO (2007), use unfactored vertical stress for pullout resistance. Thus,

$$\gamma_{P\text{-EV}} = 1.00$$

$$(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}}) = (0.125 \text{ kcf})(9.37 \text{ ft})(1.00) = 1.171 \text{ ksf}$$

- Compute nominal pullout resistance as follows:

$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}})]$$

$$P_r = (1.0)(0.955)(2)(1.00 \text{ ft})(10.31 \text{ ft})(1.171 \text{ ksf}) = 23.06 \text{ k/ft}$$

- Compute factored pullout resistance as follows:

$$P_{rr} = \phi P_r = (0.90)(23.06 \text{ k/ft}) = 20.75 \text{ k/ft}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

7.6 Establish number of longitudinal wires at each level of reinforcement

Based on $T_{\max}$, T_r and P_{rr} , the number of longitudinal wires at any given level of reinforcements can be computed as follows:

- Assume spacing of the longitudinal wires, $S_l = 6 \text{ in.} = 0.5 \text{ ft}$
- Based on tensile resistance considerations, the number of longitudinal, N_t , is computed as follows:

$$N_t = T_{\max}/T_r$$

- Based on pullout resistance considerations, the number of longitudinal wires, N_p , is computed as follows:

$$N_p = 1 + (T_{\max}/P_{rr})/(S_l)$$

Using the Level 4 reinforcement at $Z = 9.37 \text{ ft}$, the number of W11 longitudinal wires for 5 ft wide panel can be computed as follows:

- $T_{\max} = 12.75 \text{ k/panel of 5 ft width}$, $T_r = 3.36 \text{ k/wire}$, $P_{rr} = 20.75 \text{ k/ft}$
- $N_t = T_{\max}/T_r = (12.75 \text{ k/panel of 5 ft width})/(3.36 \text{ k/wire}) = 3.8 \text{ longitudinal wires/panel of 5 ft width}$
- $N_p = 1 + (T_{\max}/P_{rr})/(S_l) = 1 + [(12.75 \text{ k/panel of 5 ft width})/(20.75 \text{ k/ft})]/(0.5 \text{ ft}) = 2.2 \text{ longitudinal wires/panel}$

- Since $N_t > N_p$, tension breakage is the governing criteria and therefore the governing value, N_g , is 3.8. Select 4 longitudinal wires at Level 4 for each panel of 5 ft width.

Thus, the steel bar mat configuration at Level 4 is 4W11 + W11x0.5' which means a bar mat with 4 W11 longitudinal wires spaced at 0.5 ft on centers with W11 transverse wires spaced at 0.5 ft on centers.

The computations in Sections 7.4 to 7.6 are repeated at each level of reinforcement. Table E4-7.4 presents the computations at all levels of reinforcement for Strength I (max) load combination. Similar computations can be performed for Strength I (min) and Service I load combination but they will not govern the design because the load factors for these two load combinations are less than those for Strength I (max) load combination.

Note to users: All the long-form step-by-step calculations illustrated in Step 7 were based on hand-calculations in which numbers were rounded to the third or fourth significant digit as appropriate in each step. Table E4-7.4 was generated using a spreadsheet in which numbers at all calculation steps were not rounded. Thus, the end result in Table E4-7.4 may be somewhat different when compared to long-form hand calculations. However, the difference should be less than 0.2 in most cases.

Table E4-7.4. Summary of internal stability computations for Strength I (max) load combination

Level	Z ft	σ_H ksf	T_{max} k/5 ft wide panel	F^* dim	L_e ft	$\phi_p(P_r)$ k/ft	$\phi_s(T_n)$ k/wire	N_p -	N_t -	N_g -	Bar Mat -
1	1.87	0.40	6.25	1.188	10.31	5.16	3.38	3.4	1.9	4	4W11+W11x0.5'
2	4.37	0.67	8.36	1.110	10.31	11.25	3.38	2.5	2.5	3	3W11+W11x0.5'
3	6.87	0.86	10.80	1.033	10.31	16.47	3.38	2.3	3.2	4	4W11+W11x0.5'
4	9.37	1.02	12.77	0.955	10.31	20.75	3.38	2.2	3.8	4	4W11+W11x0.5'
5	11.87	1.14	14.26	0.438	10.31	12.06	4.84	3.4	2.9	4	4W15+W11x1'
6	14.37	1.22	15.23	0.399	11.24	14.50	4.84	3.1	3.1	4	4W15+W11x1'
7	16.87	1.26	15.71	0.360	12.74	17.41	4.84	2.8	3.2	4	4W15+W11x1'
8	19.37	1.28	16.03	0.214	14.24	13.27	4.84	3.4	3.3	4	4W15+W11x1.5'
9	21.87	1.37	17.10	0.208	15.74	16.12	4.84	3.1	3.5	4	4W15+W11x1.5'
10	24.37	1.51	19.05	0.208	17.24	19.66	4.84	2.9	3.9	4	4W15+W11x1.5'

STEP 8: DESIGN OF FACING ELEMENTS

The precast facing elements must be designed as structural elements with appropriate connection strength as discussed in Chapter 4.

STEP 9: CHECK OVERALL AND COMPOUND STABILITY AT SERVICE LIMIT STATE

From Step 2, it is given that the foundation soil is dense clayey sand that has $\phi_{fd} = 30^\circ$, $\gamma_{fd} = 125$ pcf. Furthermore, the ground in front of the wall is horizontal and the foundation soil has no water table. Therefore, based on observation, overall stability is adequate. For actual projects, overall stability should be investigated at the Service I load combination and a resistance factor of 0.65.

STEP 10: DESIGN WALL DRAINAGE SYSTEM

See Chapter 5 for wall drainage considerations.

E4-2 PRACTICAL CONSIDERATIONS

Following is a general list of practical considerations from a geotechnical and structural viewpoint:

- Attempt should be made to not vary the bar mat configuration too much because that increases the possibility of inadvertent mixing of bar mats and use of wrong bar mat at a given level.

EXAMPLE PROBLEM E5
BRIDGE ABUTMENT SUPPORTED ON SPREAD FOOTING ON TOP OF AN MSE
WALL WITH SEGMENTAL PRECAST PANEL FACING

E5-1 INTRODUCTION

This example problem demonstrates the analysis of a bridge abutment supported on a spread footing on top of an MSE wall with segmental precast panel facing. A typical configuration of such a bridge abutment is shown in Figure E5-1. The analysis of a true bridge abutment is based on various principles that were discussed in Chapters 4 and 5. Table E5-1 presents a summary of steps involved in the analysis of a bridge abutment supported on spread footing. Each of the steps and sub-steps is sequential and if the design is revised at any step or sub-step then all the previous computations need to be re-visited. Each of the steps and the sub-steps in Table E5-1 is explained in detail herein. Practical considerations for implementation of a true abutment system are presented in Section E5-2 after the illustration of the step-by-step procedures.

NOTE: A bridge abutment is a complex structure and should be analyzed very carefully since the performance of the bridge and its approach system will be affected. This example problem presents a typical case that may not be representative of all possible configurations, e.g., skewed bridge abutment, integral abutments, extensible reinforcements or other features may require additional considerations. The example problem is presented herein to demonstrate the formulation of various equations in the LRFD context for complex geometries. The formulations may have to be modified for project-specific bridge abutment configurations.

Table E5-1. Summary of steps in analysis of a true bridge abutment

Step	Item
1	Establish project requirements
2	Establish project parameters
3	Estimate wall embedment depth and length of reinforcement
4	Estimate unfactored loads
5	Summarize applicable load and resistance factors
6	Evaluate external stability of spread footing 6.1 Evaluation of limiting eccentricity 6.2 Evaluation of sliding resistance 6.3 Evaluation of bearing resistance
7	Evaluate external stability of MSE wall 7.1 Evaluation of limiting eccentricity 7.2 Evaluation of sliding resistance 7.3 Evaluation of bearing resistance 7.4 Settlement analysis
8	Evaluate internal stability of MSE wall 8.1 Estimate critical failure surface, variation of K_r and F^* for internal stability 8.2 Establish vertical layout of soil reinforcements 8.3 Calculate horizontal stress and maximum tension at each reinforcement level 8.4 Establish nominal and factored long-term tensile resistance of soil reinforcement 8.5 Establish nominal and factored pullout resistance of soil reinforcement 8.6 Establish number of soil reinforcing strips at each level of reinforcement
9	Design of facing elements
10	Check overall and compound stability at the service limit state.
11	Design wall drainage system

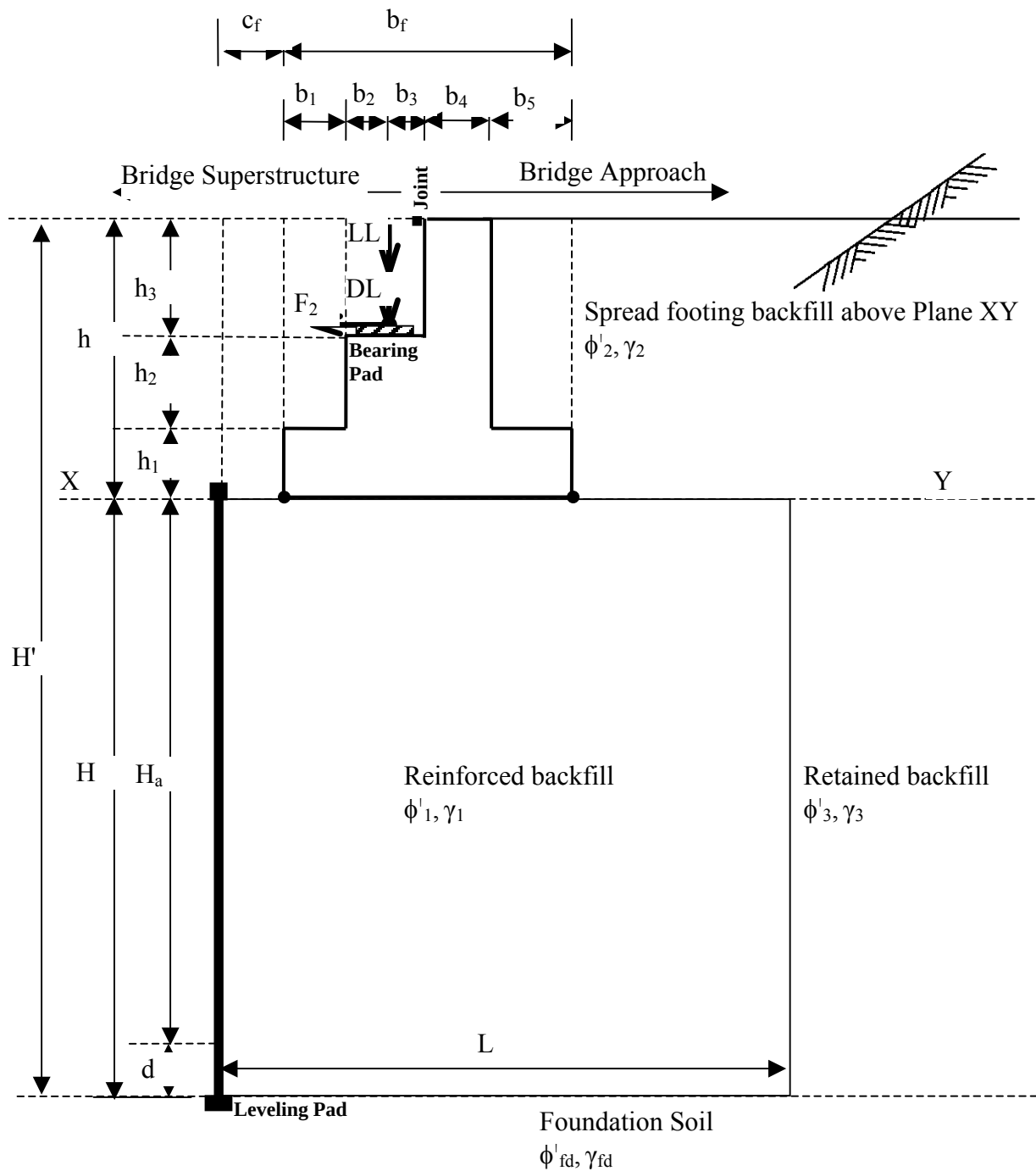


Figure E5-1. Configuration showing various parameters for analysis of a bridge abutment supported on spread footing on top of an MSE wall (not-to-scale).

STEP 1. ESTABLISH PROJECT REQUIREMENTS

- Abutment wall height, $H_a = 23$ ft (measured from finished ground to bottom of spread footing as shown in Figure E5-1)
- Length of wall at abutment = 120 ft
- Design life = 100 years (since the abutment application is considered critical)
- Precast panel units: 10 ft wide x 5 ft tall x 0.5 ft thick
- Type of reinforcement: Grade 65 ($F_y = 65$ ksi), 1.969 in. x 0.157 in. (50 mm wide x 4 mm) thick galvanized steel ribbed strips with zinc coating of 3.386 mils (86 μ m).
- Cast-in-place spread footing
- No approach slab (therefore, consider live load)

STEP 2. EVALUATE PROJECT PARAMETERS

Figure E5-1 shows the typical configuration of a true bridge abutment.

Bridge loading parameters (see Figure E5-1):

- Nominal (unfactored) Dead Load reaction, $DL = 10.60$ k/ft
- Nominal (unfactored) Live Load reaction, $LL = 5.70$ k/ft
- Nominal (unfactored) lateral friction force, F_2 , at the bearing level = 0.82 k/ft

Spread footing configuration (see Figure E5-1):

- Footing base width, $b_f = 10.75$ ft with the following values
 - $b_1 = 1.50$ ft, $b_2 = 1.50$ ft, $b_3 = 1.50$ ft, $b_4 = 1.00$ ft, $b_5 = 5.25$ ft
- Distance between toe of footing and backface of MSE panels, $c_f = 0.5$ ft
- Height of footing, $h = 10.35$ ft with the following values
 - $h_1 = 1.50$ ft, $h_2 = 3.85$ ft, $h_3 = 5.00$ ft
- Bearing pad height, $h_b = 0.08$ ft included in h_3 dimension

Soil Properties (see Figure E5-1):

- Reinforced backfill, $\phi'_1 = 34^\circ$, $\gamma_1 = 125$ pcf, coefficient of uniformity, $C_u = 7.0$ and meeting the AASHTO (2007) requirements for electrochemical properties
- Random backfill behind spread footing, $\phi'_2 = 34^\circ$, $\gamma_2 = 125$ pcf
- Retained backfill, $\phi'_3 = 30^\circ$, $\gamma_3 = 125$ pcf

Other relevant parameters:

- Spread footing concrete, $\gamma_c = 150$ pcf
- Foundation soil, $\phi'_{fd} = 30^\circ$, $\gamma_{fd} = 120$ pcf, clayey sand with no water table

- Factored Bearing resistance of foundation soil
 - For service limit consideration, $q_{nf-ser} = 7.0$ ksf for 1-inch of total settlement
 - For strength limit consideration, $q_{nf-str} = 15.0$ ksf
- Factored Bearing resistance of MSE wall (for use in analysis of spread footing on top of MSE wall)
 - For service limit considerations, $q_{nm-ser} = 4.0$ ksf for $< \frac{1}{2}$ -inch settlement
 - For strength limit considerations, $q_{nm-str} = 7.0$ ksf
- Live load on bridge approach, $h_{eq} = 2$ -ft of soil per Table 3.11.6.4-1 of AASHTO (2007)
- No seismic considerations.

STEP 3. ESTIMATE DEPTH OF EMBEDMENT AND LENGTH OF REINFORCEMENT

Based on Table C.11.10.2.2.-1 of AASHTO (2007), the minimum embedment depth = $H/10$ for abutment walls with horizontal ground in front of wall, i.e., 2.3 ft for abutment height of 23 ft. For this design, assume embedment, $d = 2.5$ ft. Thus, design height of the wall, $H = H_a + d = 23 \text{ ft} + 2.5 \text{ ft} = 25.5 \text{ ft}$.

Due to the large surcharges the length of the reinforcements for bridge abutment applications will be longer than the minimum value of $0.7H$ for walls with level backfill without surcharge(s). A good starting point for length of reinforcements for bridge abutment applications is assuming them to be approximately $1.0H$ rounded to the nearest higher number. Since in this case $H=25.5$ ft, the length of reinforcement for this example problem is assumed to be 26 ft. This length will be verified as part of the design process. The length of the reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone because differential settlements could overstress the reinforcements.

STEP 4. ESTIMATE UNFACTORED LOADS

Tables E5-4.1 and E5-4.2 present the equations for unfactored loads and moment arms about Points A and B shown in Figure E5-2. The moments are a product of the respective forces and moment arms. Each force is assigned a designation representing the applicable load type as per Tables 3.4.1-1 and 3.4.1-2 of AASHTO (2007).

To compute the numerical values of various forces and moments, the parameters provided in Step 2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure are as follows:

$$K_{a1} = (1 - \sin 34^\circ) / (1 + \sin 34^\circ) = 0.283$$

$$K_{a2} = (1 - \sin 34^\circ) / (1 + \sin 34^\circ) = 0.283$$

$$K_{a3} = (1 - \sin 30^\circ) / (1 + \sin 30^\circ) = 0.333$$

For the example problem, Tables E5-4.3 and E5-4.4 summarize the numerical values of unfactored forces and moments, respectively, based on the equations, various dimensions and values of lateral earth pressure coefficients presented above. Refer to Figure E5-2 for notations of various forces. The height of equivalent soil surcharge to represent live load, h_{eq} , will be different for analysis of footing and MSE wall block since the height of the footing is less than 20 ft. In subsequent computations, the notation h_{eqF} is used for h_{eq} for analysis of footing versus h_{eqM} for analysis of MSE block. In this example problem, the height of the footing, h , is 10.35 ft. Based on Table 3.11.6.4-1 of AASHTO (2007), $h_{eqF} = 2.96$ ft. For analysis of MSE wall, $h_{eqM} = 2$ ft since the height of MSE wall, H , is greater than 20 ft.

The unfactored forces and moments in Tables E5-4.3 and E5-4.4 form the basis of all computations in this example problem. The unfactored forces and moments should be multiplied by the appropriate load factors based on the load types identified in the second column of the Tables E5-4.1 and E5-4.2 to perform the analysis for various load combinations such as Strength I, Service I, etc.

The load factors for various load types relevant to this example problem are discussed in Step 5.

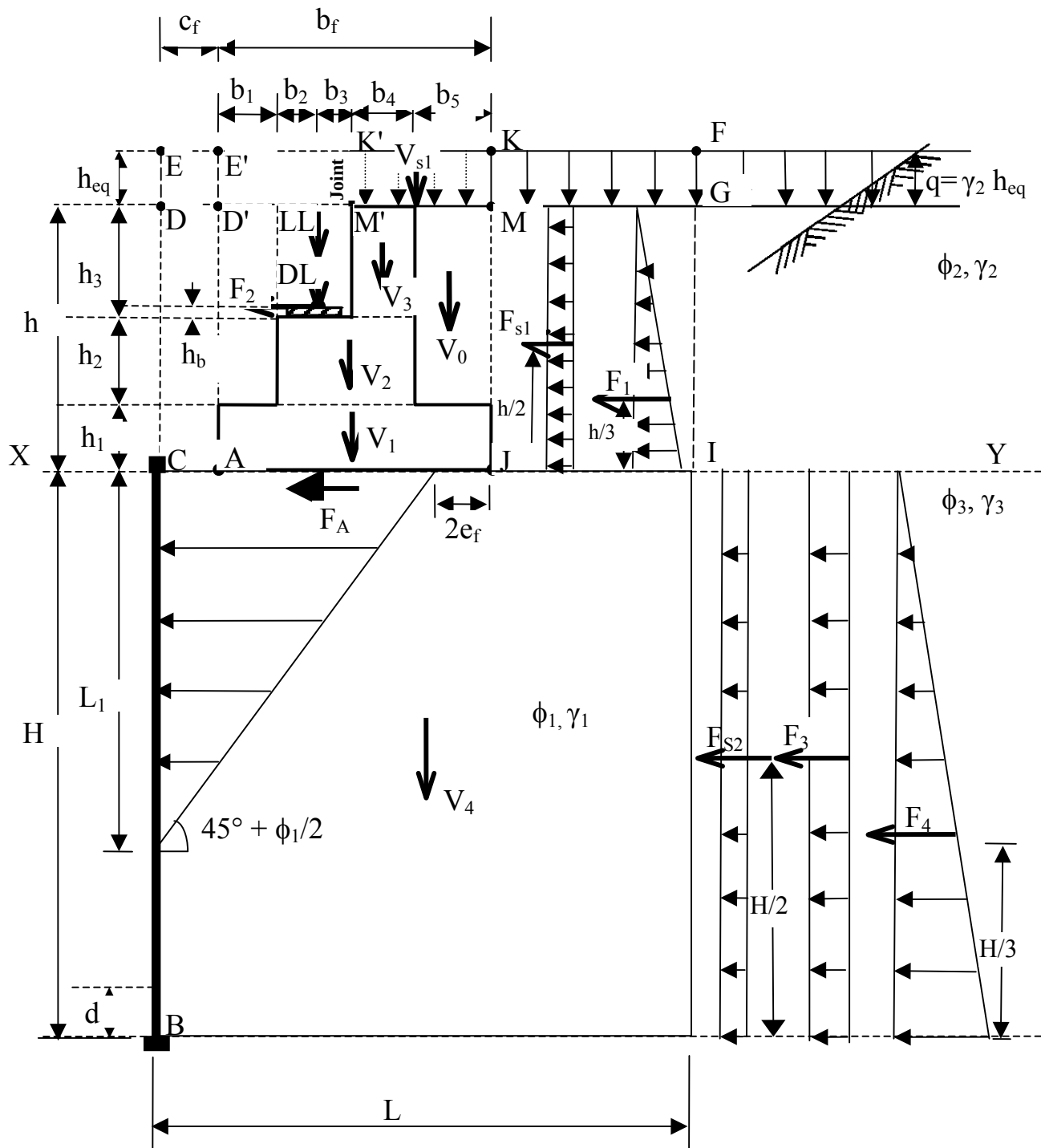


Figure E5-2. Legend for computation of forces and moments (not-to-scale).

Table E5-4.1. Equations of computing unfactored vertical forces and moments

Vertical Force (Force/length units)	LRFD Load Type	Moment arm (Length units)	
		@ Point A	@ Point B
$V_0=(\gamma_2)(h_2+h_3)(b_5)$	EV	$\frac{b_5}{2} + (b_f - b_5)$	$\frac{b_5}{2} + (b_f + c_f - b_5)$
$V_1=(\gamma_c)(b_f)(h_1)$	DC	$\frac{b_f}{2}$	$c_f + \frac{b_f}{2}$
$V_2=(\gamma_c)(b_2+b_3+b_4)(h_2)$	DC	$b_1 + \frac{b_2 + b_3 + b_4}{2}$	$c_f + b_1 + \frac{b_2 + b_3 + b_4}{2}$
$V_3=(\gamma_c)(b_4)(h_3)$	DC	$\frac{b_4}{2} + b_1 + b_2 + b_3$	$c_f + \frac{b_4}{2} + b_1 + b_2 + b_3$
$V_4=(\gamma_1)(H)(L)$	EV	-	L/2
$V_5=(\gamma_2)(h)(L)$	EV	-	L/2
$V_S=(\gamma_2)(h_{eqF})(L) = qL$	LS	-	L/2
$V_{S1}=(\gamma_2)(h_{eqF})(b_4+b_5)$	LS	$\frac{b_4 + b_5}{2} + b_1 + b_2 + b_3$	$c_f + \frac{b_4 + b_5}{2} + b_1 + b_2 + b_3$
DL	DC	$b_1 + b_2$	$c_f + b_1 + b_2$
LL	LL	$b_1 + b_2$	$c_f + b_1 + b_2$
Notes:			
1. Forces V_5 and V_S are needed later (see Figures E5-3 and E5-4).			
2. The load DL can include both “DC” and “DW” type of loads. As a simplification, herein DL is assumed to include both and a “DC” load factor is used.			

Table E5-4.2. Equations of computing unfactored horizontal forces and moments

Horizontal Force (Force/length)	LRFD Load Type	Moment arm (Length units)	
		@ Point A	@ Point B
$F_1 = 1/2 (K_{a2})(\gamma_2)h^2$	EH	h/3	(h/3) + H
F_2	FR	$h_1 + h_2 + h_b$	$h_1 + h_2 + h_b + H$
$F_3 = (K_{a3})[(\gamma_2)(h)](H)$	EH	-	H/2
$F_4 = 1/2(K_{a3})(\gamma_3)H^2$	EH	-	H/3
$F_{S1}=(K_{a2})[(\gamma_2)(h_{eqF})](h)$	LS	h/2	(h/2) + H
$F_{S2}=(K_{a3})[(\gamma_2)(h_{eqM})](H)$	LS	-	H/2
$F_A = F_1 + F_2 + F_{S1}$	Based on each component	-	H (for external stability)
			H – (L ₁ /3) (for internal stability)

Table E5-4.3. Unfactored vertical forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment Arm @ Point B, ft	Moment	Moment at Point A, k-ft/ft	Moment at Point B, k-ft/ft
$V_0 =$	5.81	8.13	8.63	$MV_0 =$	47.19	50.09
$V_1 =$	2.42	5.38	5.88	$MV_1 =$	13.00	14.21
$V_2 =$	2.31	3.50	4.00	$MV_2 =$	8.09	9.24
$V_3 =$	0.75	5.00	5.50	$MV_3 =$	3.75	4.13
$V_4 =$	82.88		13.00	$MV_4 =$		1077.38
$V_5 =$	33.64		13.00	$MV_5 =$		437.29
$V_s =$	9.62		13.00	$MV_s =$		125.06
$V_{s1} =$	2.31	7.63	8.13	$MV_{s1} =$	17.63	18.79
$DL =$	10.60	3.00	3.50	$MDL =$	31.80	37.10
$LL =$	5.70	3.00	3.50	$MLL =$	17.10	19.95
Notes:						
1. V_s and V_{s1} is computed based on $h_{eqF} = 2.96$ ft since $h = 10.35$ ft						

Table E5-4.4. Unfactored horizontal forces and moments

Force	Value k/ft	Moment Arm @ Point A, ft	Moment Arm @ Point B, ft	Moment	Moment at Point A, k-ft/ft	Moment at Point B, k-ft/ft
$F_1 =$	1.89	3.45	28.95	$MF_1 =$	6.53	54.80
$F_2 =$	0.82	5.43	30.93	$MF_2 =$	4.43	25.21
$F_3 =$	11.00		12.75	$MF_3 =$		140.21
$F_4 =$	13.55		8.50	$MF_4 =$		115.15
$F_{S1} =$	1.08	5.18	30.68	$MF_{S1} =$	5.60	33.21
$F_{S2} =$	2.13		12.75	$MF_{S2} =$		27.09
$F_A =$	*		25.50	$MF_A =$		$F_A (25.50)$
Notes:						
1. $F_A = F_1 + F_2 + F_{S1}$ and each of the components of F_A is a different load type and hence has a different load factor.						
2. F_{S1} is computed based on $h_{eqF} = 2.96$ ft since $h = 10.35$ ft						
3. F_{S2} is computed based on $h_{eqM} = 2$ ft since $H > 20$ ft						

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table E5-5.1 summarizes the load factors for the various LRFD load type shown in the second column of Tables E5-4.1 and E5-4.2. **Throughout the computations in this example problem, the forces and moments in Tables E5-4.1 and E5-4.2 should be multiplied by appropriate load factors.** For example, if computations are being done for Strength I (maximum) load combination, the forces and moments corresponding to load V_2 should be multiplied by 1.25 which is associated with load type DC assigned to load V_2 .

Table E5-5.1. Summary of applicable load factors

Load Combination	Load Factors (after AASHTO, 2007 Tables 3.4.1-1 and 3.4.1-2)					
	EV	DC	LL/LS	ES	EH	FR
Strength I (maximum)	1.35	1.25	1.75	1.50	1.50	1.00
Strength I (minimum)	1.00	0.90	1.75	0.75	0.90	1.00
Service I	1.00	1.00	1.00	1.00	1.00	1.00

For computation of factored resistances during evaluation of strength limits states, appropriate resistance factors have to be used. Table E5-5.2 summarizes the applicable resistance factors. For service limit state, all resistance factors are equal to 1.0.

Table E5-5.2. Summary of applicable resistance factors for evaluation of resistances

Item	Resistance Factors	AASHTO (2007) Reference
Sliding of cast-in-place spread footing on MSE wall	$\phi_s = 0.80$	Table 10.5.5.2.2-1
Sliding of MSE wall on foundation soil	$\phi_s = 1.00$	Table 11.5.6-1
Bearing resistance of MSE wall	$\phi_b = 0.65$	Table 11.5.6-1
Tensile resistance (for steel strips)	$\phi_t = 0.75$	Table 11.5.6-1
Pullout resistance	$\phi_p = 0.90$	Table 11.5.6-1

STEP 6. EVALUATE EXTERNAL STABILITY OF SPREAD FOOTING

6.1 Limiting Eccentricity at base of Spread Footing

Since the computations are related to limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. Limiting eccentricity is a strength limit state check and therefore only the strength limits state calculations are necessary. However, service limit state calculations are also included since some of the results will be needed later in internal stability analysis. The critical values from strength limit state calculations based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table E5-6.1. Computations for evaluation of limiting eccentricity for spread footing

Item	Unit	Str I (max)	Str I (min)	Ser I
Sum of overturning moments about Point A $M_{OA} = MF_1 + MF_{s1} + MF_2$	k-ft/ft	24.03	20.11	16.56
Sum of resisting moments about Point A $M_{RA} = MV_0 + MV_1 + MV_2 + MV_3 + MDL$	k-ft/ft	134.50	98.16	103.82
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft/ft	110.47	78.05	87.27
Sum of vertical forces from the footing $V_A = V_0 + V_1 + V_2 + V_3 + DL$	k/ft	27.94	20.28	21.89
Location of resultant from Point A, $a_{nl} = M_A / V_A$	ft	3.95	3.85	3.99
Eccentricity from center of footing, $e_f = 0.5 * b_f - a_{nl}$	ft	1.42	1.53	1.39
Limiting eccentricity, $e = b_f / 4$ for Strength limit state and $e = b_f / 6$ for service limit state	ft	2.69	2.69	1.79
Is the limiting eccentricity criteria satisfied?	-	Yes	Yes	Yes
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point A, M_{OA-C}	k-ft/ft	24.03		
Resisting moments about Point A, M_{RA-C}	k-ft/ft	98.16		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	74.13		
Vertical force, V_{A-C}	k/ft	20.28		
Location of resultant from Point A, $a_{nl} = M_{A-C} / V_{A-C}$	ft	3.66		
Eccentricity from center of footing, $e_f = 0.5 * b_f - a_{nl}$	ft	1.72		
Limiting eccentricity, $e = b_f / 4$	ft	2.69		
Is the limiting eccentricity criteria satisfied?	-	Yes		

6.2 Sliding Resistance at base of Spread Footing

The purpose of these computations is to evaluate the sliding resistance at the base of the spread footing. Since the computations are related to sliding resistance, the beneficial contribution of live load to resisting horizontal forces is neglected. Note that sliding resistance is a strength limit state check and therefore only the strength limits state calculations are necessary. However, service limit state calculations are also included since some of the results will be needed later in internal stability analysis. Since the friction angle of reinforced soil, ϕ'_1 , is same as the friction angle for random fill above base of footing, ϕ'_2 , the sliding check will be performed using $\phi'_1 = \phi'_2 = 34^\circ$. The critical values based on max/min result in the extreme force effect and govern the sliding mode of failure.

Table E5-6.2. Computations for evaluation of sliding resistance for spread footing

Item	Unit	Str I (max)	Str I (min)	Ser I
Sum of horizontal forces on footing that contribute to sliding = $F_A = F_1 + F_{S1} + F_2$	k/ft	5.55	4.41	3.79
Sum of vertical forces from the footing $V_A = V_0 + V_1 + V_2 + V_3 + DL =$	k/ft	27.94	20.28	21.89
Nominal sliding resistance, $V_N = V_A \cdot \tan(\phi'_1)$	k/ft	18.85	13.68	14.76
Factored sliding resistance, $V_F = \phi_s \cdot V_N$	k/ft	15.08	10.94	11.81
Is $V_F > F_A$?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) = $V_F : F_A$	dim	2.72	2.48	3.12
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_F (V_{Fmin})	k/ft	10.94		
Maximum F_A (F_{Amax})	k/ft	5.55		
Is $V_{Fmin} > F_{Amax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = $V_{Fmin} : F_{Amax}$	dim	1.97		

6.3 Bearing Resistance at base of Spread Footing

The purpose of these computations is to evaluate the bearing resistance at the base of the spread footing. Since the computations are related to bearing resistance, the contribution of live load is included to create the extreme force effect and maximize the bearing stress. The bearing stress is compared with bearing resistance to ensure that the footing is adequately sized. Later on, the bearing stress is also used in internal stability computations. Similarly, later the value of dimension L_1 that defines the incremental lateral pressures due to lateral load F_A (see Figure E5-2) will be needed and hence has been computed in Table E5-6.3.

Table E5-6.3. Computations for evaluation of bearing resistance for spread footing

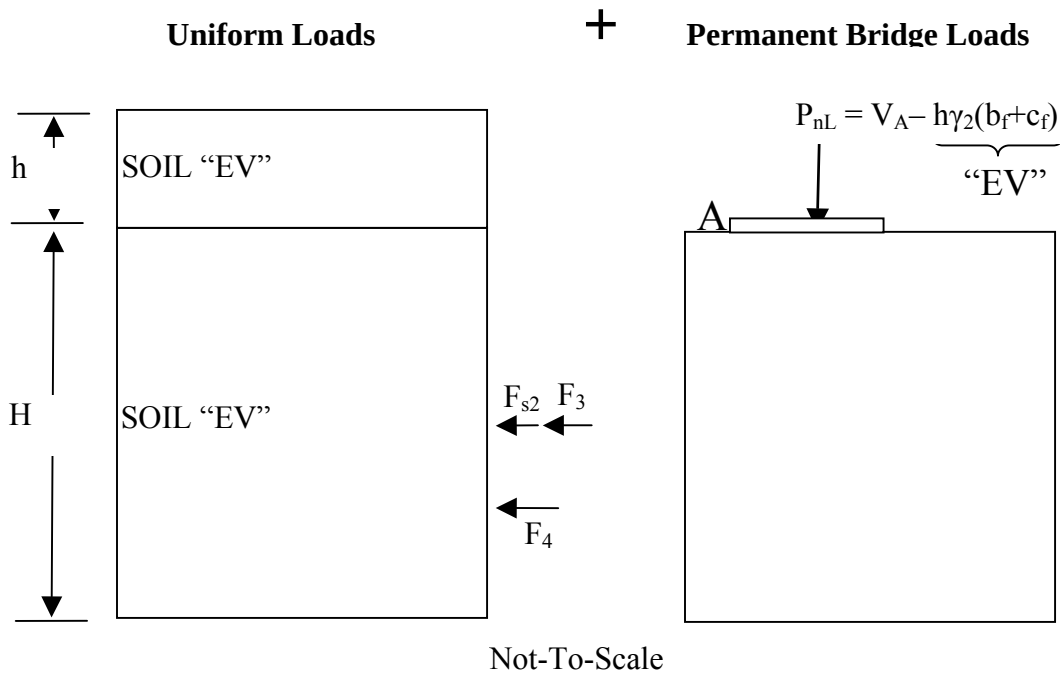
Item	Unit	Str I (max)	Str I (min)	Ser I
Sum of overturning moments about Point A = M_{OA} $=MF_1+MF_{s1}+MF_2$	k-ft/ft	24.03	20.11	16.56
Sum of resisting moments about Point A = M_{RA} = $MV_0+MV_1+MV_2+MV_3+MDL+MLL+MV_{S1}$	k-ft/ft	195.28	158.94	138.56
Factored net moment at Point A, $M_A = M_{RA} - M_{OA} =$	k-ft/ft	171.26	138.84	122.00
Sum of vertical forces from the footing for bearing stress analysis= $V_{Ab}=V_0+V_1+V_2+V_3+DL+LL+V_{S1}$	k/ft	41.96	34.30	29.90
Location of resultant from Point A for bearing stress analysis, $a_{wl} = M_A/V_{Ab}$	ft	4.08	4.05	4.08
Eccentricity from center of footing, $e_f = 0.5*b_f - a_{wl}$	ft	1.29	1.33	1.29
Limiting eccentricity, $e = b_f/4$ for strength limit states and $e = b_f/6$ for service limit state	ft	2.69	2.69	1.79
Is the resultant within limiting value of e_f ?	-	Yes	Yes	Yes
Effective width of spread footing, $b_f' = b_f - 2e_f =$	ft	8.16	8.10	8.16
Bearing stress, $\sigma_v = V_{Ab} / (b_f - 2e_f)$	ksf	5.14	4.24	3.66
Factored bearing resistance, q_R (given)	ksf	7.00	7.00	4.00
Is bearing stress < factored bearing resistance?	dim	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) = $q_R:\sigma_v$	dim	1.36	1.65	1.09
Depth of influence for the lateral force at base of footing, $L_1 = \{c_f + (b_f - 2e_f)\} \tan(45^\circ + 34^\circ/2)$	ft	16.29	16.17	16.29
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point A, M_{OA-C}	k-ft/ft	24.03		
Resisting moments about Point A, M_{RA-C}	k-ft/ft	158.94		
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	134.91		
Vertical force, V_{Ab-C}	k/ft	34.30		
Location of resultant from Point A, $a_{wl} = M_{A-C}/V_{A-C}$	ft	3.93		
Eccentricity from center of footing, $e_f = 0.5*b_f - a_{wl}$	ft	1.44		
Limiting eccentricity, $e = b_f/4$	ft	2.69		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of spread footing, $b_f' = b_f - 2e_f$	ft	7.87		
Bearing stress, $\sigma_{v-c} = V_{Ab-C} / (b_f - 2e_f)$	ksf	4.36		
Factored bearing resistance, q_R (given)	ksf	7.00		
Is bearing stress < factored bearing resistance?	dim	Yes		
Capacity:Demand Ratio (CDR) = $q_R:\sigma_{v-c}$	dim	1.61		

STEP 7. EVALUATE EXTERNAL STABILITY OF MSE WALL

The external stability of MSE wall is a function of the various forces and moments above plane XY in Figure E5-2. In the LRFD context the forces and moments need to be categorized into various load types. The primary load types are soil loads (EV, EH), live load (LL), and permanent loads (DC, DW). The principle of superposition shown in Figure 11.10.10.1-1 of AASHTO (2007) is used in achieving the separation of load types as well as performing external stability computations of the MSE wall. This separation of various load types will also permit a proper evaluation of the internal stability of the MSE wall.

The separation of the loads by principle of superposition is primarily achieved by use of uniform loads and concentrated loads. The uniform loads are used to represent soil and live loads while the concentrated loads are used to represent permanent bridge loads due to the bridge superstructure and the concrete spread footing. Since LL is treated differently in the computations of limiting eccentricity, sliding resistance and bearing resistance, Figures E5-3 and E5-4 show separation of various load types without and with live load components, respectively. The equations shown in these figures are for computing unfactored forces and moments. The relevant LRFD load type (e.g., “EV”, “LL”, etc.) are shown in the figures to permit computations of factored forces and moments using the appropriate load factors as identified in Table E5-5.1 in Step 5.

It should be noted that use of principle of superposition results in an approximate representation of a very complex system. A case can conceivably be made for further separation of forces. However, a more refined system of forces is not justified given that the behavior of most elements of the system is designed to be within elastic range. One case where the approximations shown in Figure E5-3 and E5-4 may not be applicable is that of integral bridge abutments. In such specialized cases, additional forces may need to be considered depending on the actual abutment configuration.



Summary of relevant equations

$$F_A = F_1 + F_2 + F_{S1}$$

$$V_A = V_0 + V_1 + V_2 + V_3 + DL$$

$$M_{RA} = MV_0 + MV_1 + MV_2 + MV_3 + MDL$$

$$M_{OA} = MF_1 + MF_2 + MF_{S1}$$

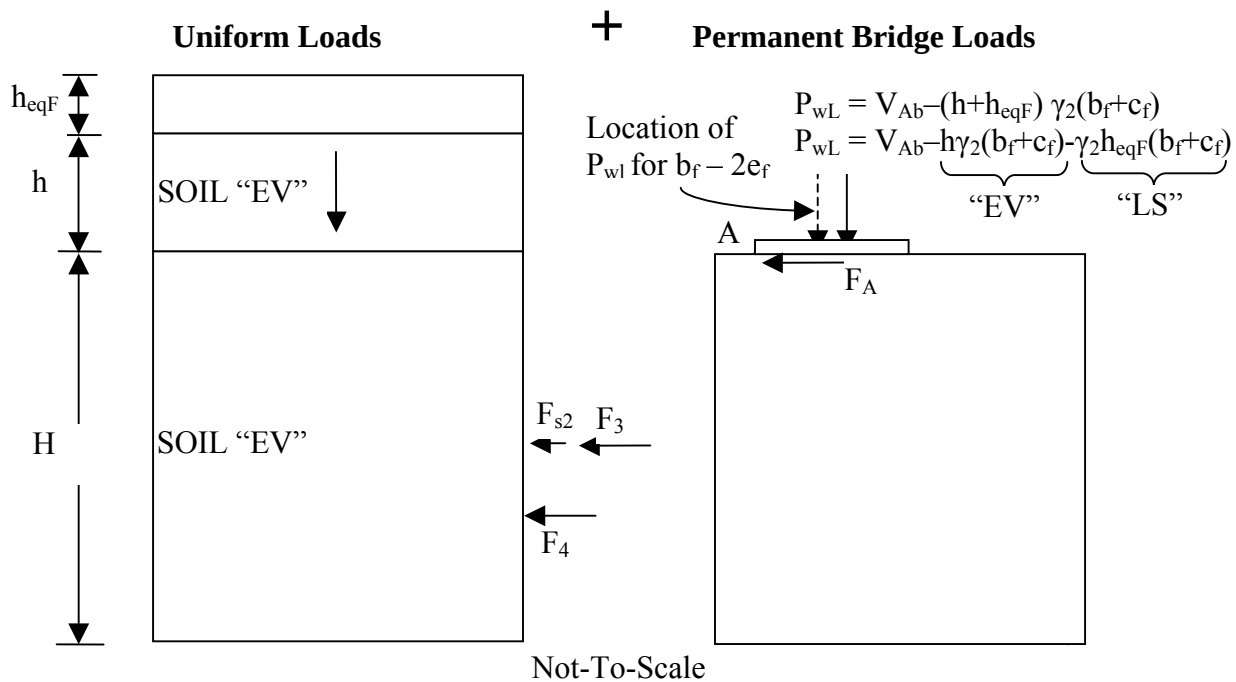
$$M_A = M_{RA} - M_{OA}$$

$$\text{Location of } P_{nL} \text{ from Point A, } a_{nL} = M_A / V_A$$

$$\text{Eccentricity of } P_{nL} \text{ from center of footing, } e_f = 0.5 \cdot b_f - a_{nL}$$

Note: See Figure E5-2 and Table E5-4.1 and E5-4.2 for additional information related to notations for various forces and moments and associated LRFD load types.

Figure E5-3. Superposition of load effects without Live Loads on MSE Wall.



Summary of relevant equations

$$F_A = F_1 + F_2 + F_{S1}$$

$$V_{Ab} = V_0 + V_1 + V_2 + V_3 + DL + LL + V_{S1}$$

$$M_{RAb} = MV_0 + MV_1 + MV_2 + MV_3 + MDL + MLL + MV_{S1}$$

$$M_{OAb} = MF_1 + MF_2 + MF_{S1}$$

$$M_{Ab} = M_{RAb} - M_{OAb}$$

$$\text{Location of } P_{wL} \text{ from Point A, } a_{wL} = M_{Ab} / V_{Ab}$$

$$\text{Eccentricity of } P_{wL} \text{ from center of footing, } e_f = 0.5 * b_f - a_{wL}$$

Note: See Figure E5-2 and Table E5-4.1 and E5-4.2 for additional information related to notations for various forces and moments and associated LRFD load types.

Figure E5-4. Superposition of load effects with Live Loads on MSE Wall.

7.1 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table E5-7.1. Limiting eccentricity is a strength limit state check. However, in Table E5-7.1, the calculations are also performed for service limit state to obtain the effective footing width which will be used to determine the equivalent uniform (Meyerhof) bearing stress in Step 7.3 that will be compared to limiting bearing resistance from serviceability considerations (see Step 2).

Table E5-7.1. Computations for evaluation of limiting eccentricity for MSE wall

Item (Refer to Figure E5-2 for block CDMJ). The top 13 rows in this table are related to computation of P_w , P_{nL} , a_{wL} and a_{nL} values that will also be used in internal stability calculations.	Unit	Str I (max)	Str I (min)	Ser I
Unfactored soil weight in block CDMJ in the abutment footing area = $(h)[(b_f+c_f)*\gamma_2]$	k/ft	14.55	14.55	14.55
Load factor for soil weight in block CDMJ [Load Type "EV"]	dim	1.35	1.00	1.00
Factored soil weight in block CDMJ in the abutment footing area	k/ft	19.65	14.55	14.55
Unfactored LL weight on block CDMJ in the abutment footing area = $(b_f+c_f)*q = (b_f+c_f)*(\gamma_2)(h_{eqF})$	k/ft	4.16	4.16	4.16
Load factor for LL on block CDMJ [Load Type "LS"]	dim	1.75	1.75	1.00
Factored LL weight on block CDMJ in the abutment footing area	k/ft	7.28	7.28	4.16
Vertical weight due to soil weight and LL in block CDMJ	k/ft	26.93	21.84	18.72
Vertical weight from abutment footing including soil on heel and LL = V_{Ab}	k/ft	41.96	34.30	29.90
Vertical weight from abutment footing including soil on heel and no LL = V_A	k/ft	27.94	20.28	21.89
Net load, P, on base of spread footing from the bridge (with consideration of LL), P_{wL}	k/ft	15.03	12.46	11.18
Net load, P, on base of spread footing from the bridge (no consideration of LL), P_{nL}	k/ft	8.29	5.72	7.33
Moment arm of net load P_{nL} from Point B, $L_p = a_{nL} + c_f$	ft	4.45	4.35	4.49
Resisting moment at Point B due to net load P, $M_{pNL} = P_{nL}(L_p)$	k-ft/ft	36.93	24.89	32.90
Vertical load at base of MSE wall without LL, $V_B = V_4 + V_5 + P_{nL}$	k/ft	165.58	122.24	123.84
Resisting moments about Point B without LL surcharge = $M_{RB} = MV_4 + MV_5 + MP_{nL}$	k-ft/ft	2081.72	1539.56	1547.56
Overturning moments about Point B = $M_{OB} = MF_{S3} + MF_3 + MF_4 + MF_A$ (For MF_A See Note 1)	k-ft/ft	666.03	454.90	448.67
Location of the resultant force on base of MSE wall from Point B, $b = (M_{RB} - M_{OB})/V_B$	ft	8.55	8.87	8.87
Eccentricity at base of MSE wall, $e_L = L/2 - b$	ft	4.45	4.13	4.13
Limiting eccentricity, $e = L/4$ for strength limit states and $e = L/6$ for service limit state	ft	6.50	6.50	4.33
Is the resultant within limiting value of e_L ?	-	Yes	Yes	Yes
CRITICAL VALUES BASED ON MAX/MIN				
Overturning moments about Point B, M_{OB-C}	k-ft/ft	666.03		

Resisting moments about Point B, M_{RB-C}	k-ft/ft	1539.56	
Net moment about Point A, $M_{B-C} = M_{RB-C} - M_{OB-C}$	k-ft/ft	873.53	
Vertical force, V_{B-C}	k/ft	122.24	
Location of resultant from Point B, $b = M_{B-C}/V_{B-C}$	ft	7.15	
Eccentricity from center of footing, $e_L = L/2 - b$	ft	5.85	
Limiting eccentricity, $e = L/4$	ft	6.50	
Is the limiting eccentricity criteria satisfied?	-	Yes	

Note 1: $MF_A = (F_A)(H) = (F_1 + F_2 + F_{S1})(H)$ and each of the components of F_A is a different load type and hence has a different load factor.

7.2 Sliding Resistance at Base of MSE Wall

The purpose of these computations is to evaluate the sliding resistance at the base of the MSE wall. Since the computations are related to sliding resistance, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for sliding resistance at the base of the MSE wall are illustrated in Table E5-7.2. Note that sliding resistance is a strength limit state check and therefore service limit state calculations are not performed. Since the friction angle of foundation soil, ϕ'_{fd} , is less than the friction angle for reinforced soil, ϕ_1 , the sliding check will be performed using ϕ'_{fd} .

Table E5-7.2. Computations for evaluation of sliding resistance of MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Lateral load on the MSE wall, $H_m = F_1 + F_2 + F_3 + F_4 + F_{S1} + F_{S2}$	k/ft	46.08	30.22	NA
Vertical load at base of MSE wall without LL surcharge = $V_4 + V_5 + P_{nL}$	k/ft	165.58	122.24	NA
Nominal sliding resistance at base of MSE wall, $V_{Nm} = \tan(\phi'_{fd})(V_4 + V_5 + P_{nL})$	k/ft	95.60	70.57	NA
Factored sliding resistance at base of MSE wall, $V_{Fm} = \phi_s * V_{Nm}$	k/ft	95.60	70.57	NA
Is $V_{Fm} > H_m$?	-	Yes	Yes	NA
Capacity:Demand Ratio (CDR) = V_{Fm}/H_m	dim	2.07	2.34	NA
CRITICAL VALUES BASED ON MAX/MIN				
Minimum V_{Fm} (V_{Fmmin})	k/ft	70.57		
Maximum H_m (H_{mmax})	k/ft	46.08		
Is $V_{Fmmin} > H_{mmax}$?	-	Yes		
Capacity:Demand Ratio (CDR) = V_{Fmmin}/H_{mmax}	dim	1.53		

7.3 Bearing Resistance at base of MSE Wall

For bearing resistance computations, the effect of live load is included since it creates larger bearing stresses. Figure E5-4 is used for bearing resistance computations. The bearing stress

at the base of the MSE wall is due to the effect of (1) net bridge load, and (2) the MSE wall. Each of these two components is briefly discussed below and their computation is illustrated in Table E5-7.3 in conjunction with Figure E5-4.

Component 1: The net bridge load on the footing is assumed to include live load and is denoted as P_{wL} which is assumed to be centered on $b_f - 2e_f$. The stress due to net bridge load P_{wL} is diffused at 1H:2V distribution through the height of the MSE wall. Thus, the vertical stress at the base of the MSE wall due to the net bridge load, P_{wL} is as follows:

$$\Delta\sigma_{v-\text{footing}} = \frac{P_{wL}}{(b_f - 2e_f) + \left(c_f + \frac{H}{2}\right)}$$

Component 2: The MSE wall by itself will create a certain bearing stress at its base due to the effect of other loads, i.e. the effect of V_4 , V_5 , V_S , F_{S2} , F_3 , F_4 , and F_A . The bearing stress due to these loads is as follows:

$$\sigma_v = \frac{\sum V}{L - 2e_L}$$

where $\sum V = V_4 + V_5 + V_S$ and the load eccentricity e_L is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

Total equivalent uniform (Meyerhof) bearing stress: The total equivalent uniform (Meyerhof) bearing stress at the base of the MSE wall is obtained as follows:

$$\sigma_{v\max} = \sigma_v + \Delta\sigma_{v-\text{footing}}$$

In LRFD, $\sigma_{v\max}$ is then compared with the factored bearing resistance when computed for strength limit state and used for settlement analysis when computed for service limit state. The various computations for evaluation of bearing resistance are presented in Table E5-7.3.

6.4 Settlement Analysis

It is critical that the settlement under $\sigma_{v\max}$ be evaluated because the performance of the bridge will be directly affected by the settlement at the back of the MSE wall. Settlement is evaluated at Service I Limit State. Note that due to the reinforced MSE wall the settlement of the spread footing on top of the MSE wall is assumed to be very small, i.e. negligible if the footing is sized such that the bearing stress is less than 4 ksf under Service I load combination. Conservatively, all the settlement at the base of the MSE wall is assumed to occur at the spread footing level, i.e. the MSE wall is assumed to be a rigid block.

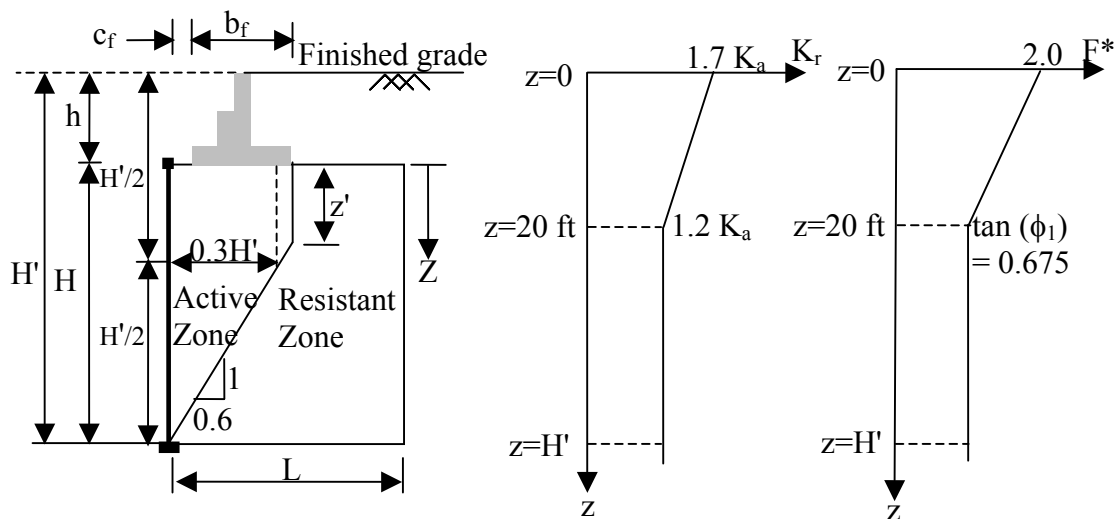
Table E5-7.3. Computations for evaluation of bearing resistance for MSE wall

Item	Unit	Str I (max)	Str I (min)	Ser I
Component 1: Bearing Stress due to P_{wL} acting over b_f-2e_f and distributed 1H:2V through the MSE wall height				
Base width of stress distribution based on 1H:2V distribution and P_{wL} acting on $b_f' = b_f - 2e_f$	ft	21.41	21.35	21.41
Bearing stress due to P_{wL}	ksf	0.70	0.58	0.52
Component 2: Bearing stress due to MSE wall				
Vertical load at base of MSE wall including LL on top, $V = V_4 + V_5 + V_S$	k/ft	174.13	133.35	126.13
Resisting moments @ Point B on the MSE wall, $M_{RB} = MV_5 + MV_S + MV_4$	k-ft/ft	2263.65	1733.52	1639.72
Overtuning moments @ Point B on the MSE wall, $M_{OB} = MF_{S2} + MF_3 + MF_4$	k-ft/ft	571.95	389.77	379.11
Net moment at Point B, $M_B = M_{RB} - M_{OB}$	k-ft/ft	1691.70	1343.74	1260.61
Location of Resultant from Point B, $b = M_B/V$	ft	9.72	10.08	9.99
Eccentricity from center of wall, $e_L = 0.5 \cdot L - b$	ft	3.28	2.92	3.01
Limiting eccentricity, $e = L/4$ for strength limit states and $e = L/6$ for service limit state	ft	6.50	6.50	4.33
Is the resultant within limiting value of e_L ?	-	Yes	Yes	Yes
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	19.43	20.15	19.99
Factored bearing stress due to MSE wall = $V/(L - 2e_L)$	ksf	8.96	6.62	6.31
Total bearing stress due to Component 1 + 2 = σ_{vmax}	ksf	9.66	7.20	6.83
Factored bearing resistance, q_R (given)	ksf	15.00	15.00	7.00
Is $\sigma_{vmax} < q_R$?	-	Yes	Yes	Yes
Capacity:Demand Ratio (CDR) = $q_R : \sigma_{vmax}$	dim	1.55	2.08	1.02
CRITICAL VALUES BASED ON MAX/MIN FOR COMPONENT 2				
Overtuning moments about Point B, M_{OB-C}	k-ft/ft	571.95		
Resisting moments about Point B, M_{RB-C}	k-ft/ft	1733.52		
Net moment about Point B, $M_{B-C} = M_{RB-C} - M_{OB-C}$	k-ft/ft	1161.57		
Vertical force, V_{Bb-C}	k/ft	133.35		
Location of resultant from Point B, $b = M_{B-C}/V_{Bb-C}$	ft	8.71		
Eccentricity from center of wall, $e_L = 0.5 \cdot L - b$	ft	4.29		
Limiting eccentricity, $e = L/4$	ft	6.50		
Is the limiting eccentricity criteria satisfied?	-	Yes		
Effective width of base of MSE wall, $B' = L - 2e_L$	ft	17.42		
Bearing stress, $\sigma_{v-c} = V_{Bb-C} / (L - 2e_L)$	ksf	7.65		
Compute critical total bearing stress				
Total bearing stress due to Component 1+2 = σ_{vmax-C}	ksf	8.35		
Factored bearing resistance, q_R (given)	ksf	15.00		
Is bearing stress < factored bearing resistance?	dim	Yes		
Capacity:Demand Ratio (CDR) = $q_R : \sigma_{vmax-C}$	dim	1.80		

STEP 8: EVALUATE INTERNAL STABILITY ANALYSIS OF MSE WALL

8.1 Estimate critical failure surface, variation of K_r and F^* for internal stability

The quantity $c_f + b_f (=11.25 \text{ ft})$, is greater than $H/3$ ($25.5/3=8.5 \text{ ft}$). Therefore, the modified shape of the maximum tensile force line (i.e., critical failure surface) shown in Figure 5-1 of Chapter 5 has to be used. For the case of inextensible steel ribbed strips, the profile of the critical failure surface, the variation of internal lateral horizontal stress coefficient, K_r , and the variation of the pullout resistance factor, F^* , are as shown in Figure E5-5 wherein other definitions such as measurement of depths Z and z are also shown. It should be noted that the variation of K_r and F^* are with respect to depth z that is measured from top of the spread footing while the critical failure surface is with respect to depth z that is measured from top of coping. The value of K_a is based on the angle of internal friction of the reinforced backfill, ϕ_1 , which is equal to $K_{a1} = 0.283$ calculated in Step 4. Thus, the value of K_r varies from $1.7(0.283) = 0.481$ at $z = 0$ to $1.2(0.283) = 0.340$ at $z = 20 \text{ ft}$. For steel strips, $F^* = 1.2 + \log_{10} C_u$. Using $C_u = 7.0$ as given in Step 2, $F^* = 1.2 + \log_{10}(7.0) = 2.045 > 2.000$. Therefore, use $F^* = 2.000$.



Notes:

- Z is measured below bottom of footing; z is measured from top of finished grade
- H is measured from top of leveling pad to bottom of spread footing
- $z = Z + h$; $z' = H - (c_f + b_f)/0.6$
- Within height z' the length of the reinforcement in the active zone is $L_a = c_f + b_f$

Figure E5-5. Geometry definition, location of critical failure surface and variation of K_r and F^* parameters for steel ribbed strips.

8.2 Establish vertical layout of soil reinforcements and tributary areas

Using the definition of depth Z as shown in Figure E5-5 the following vertical layout of the soil reinforcements is chosen.

$Z =$ 1.12 ft, 2.35 ft, 4.81 ft, 7.27 ft, 9.73 ft, 12.19 ft, 14.65 ft, 17.11 ft, 19.57 ft, 22.03 ft, 24.49 ft.

The above layout leads to 11 levels of reinforcements. The vertical spacing was chosen based on a typical vertical spacing, S_v , of 2.46 ft that is commonly used in the industry for steel ribbed strip reinforcement. The vertical spacing near the top and bottom of the walls are locally adjusted as necessary to fit the height of the wall.

For internal stability computations, each layer of reinforcement is assigned a tributary area, A_{trib} as follows

$$A_{trib} = (w_p)(S_{vt})$$

where w_p is the panel width of the precast facing element and S_{vt} is the vertical tributary spacing of the reinforcements based on the location of the reinforcements above and below the level of the reinforcement under consideration. The computation of S_{vt} is summarized in Table E5-8.1 wherein $S_{vt} = Z^+ - Z^-$. Note that $w_p = 10.00$ ft per Step 2.

Table E5-8.1. Summary of computations for S_{vt}

Level	Z (ft)	Z^- (ft)	Z^+ (ft)	S_{vt} (ft)
1	1.12	0	$1.12 + 0.5(2.35 - 1.12) = 1.735$	1.735
2	2.35	$2.35 - 0.5(2.35 - 1.12) = 1.735$	$2.35 + 0.5(4.81 - 2.35) = 3.58$	1.845
3	4.81	$4.81 - 0.5(4.81 - 2.35) = 3.58$	$4.81 + 0.5(7.27 - 4.81) = 6.04$	2.460
4	7.27	$7.27 - 0.5(7.27 - 4.81) = 6.04$	$7.27 + 0.5(9.73 - 7.27) = 8.50$	2.460
5	9.73	$9.73 - 0.5(9.73 - 7.27) = 8.50$	$9.73 + 0.5(12.19 - 9.73) = 10.96$	2.460
6	12.19	$12.19 - 0.5(12.19 - 9.73) = 10.96$	$12.19 + 0.5(14.65 - 12.19) = 13.42$	2.460
7	14.65	$14.65 - 0.5(14.65 - 12.19) = 13.42$	$14.65 + 0.5(17.11 - 14.65) = 15.88$	2.460
8	17.11	$17.11 - 0.5(17.11 - 14.65) = 15.88$	$17.11 + 0.5(19.57 - 17.11) = 18.34$	2.460
9	19.57	$19.57 - 0.5(19.57 - 17.11) = 18.34$	$19.57 + 0.5(22.03 - 19.57) = 20.80$	2.460
10	22.03	$22.03 - 0.5(22.03 - 19.57) = 20.80$	$22.03 + 0.5(24.49 - 22.03) = 23.26$	2.460
11	24.49	$24.49 - 0.5(24.49 - 22.03) = 23.26$	25.50	2.240

8.3 Calculate horizontal stress and maximum tension at each reinforcement level

The horizontal spacing of the reinforcements is based on the maximum tension ($T_{\max}$) at each level of reinforcements which requires computation of the horizontal stress, σ_H , at each reinforcement level. The reinforcement tensile and pullout resistances are then compared with $T_{\max}$ and an appropriate reinforcement pattern is adopted. This section demonstrates the calculation of horizontal stress, σ_H , and maximum tension, $T_{\max}$.

The horizontal stress, σ_H , at any depth within the MSE wall is based on the following components each of which is summarized in Table E5-8.2.

$$\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}} + \sigma_{H\text{-footing}} + \Delta\sigma_H$$

Table E5-8.2. Summary of load components leading to horizontal stress

Load Component	Load Type	Horizontal Stress
Soil load, $\sigma_{v\text{-soil}}$	EV	$\sigma_{H\text{-soil}} = [K_r \sigma_{v\text{-soil}}] \gamma_{P\text{-EV}}$
Surcharge traffic live load, q	EV	$\sigma_{H\text{-surcharge}} = [K_r q] \gamma_{P\text{-EV}} = [K_r (\gamma_r h_{eqM})] \gamma_{P\text{-EV}}$
Vertical footing load, $\Delta\sigma_{v\text{-footing}}$	ES*	$\sigma_{H\text{-footing}} = [K_r \Delta\sigma_{v\text{-footing}}] \gamma_{P\text{-ES}}$
Horizontal surcharge, F_A	ES*	$\Delta\sigma_H = (2F_A/L_1) \gamma_{P\text{-ES}}$ at $z_0=0$; $\Delta\sigma_H=0$ at $z_0=L_1$
*As per Article 3.116 of AASHTO (2007 with 2009 Interims), the value of ES may be 1.50 or 1.00 based on how the horizontal stresses are computed. First, compute horizontal stresses by using factored loads and use ES=1.0 since the horizontal stresses are based on factored loads. Second, compute horizontal stresses by using nominal loads and then apply ES=1.50. Choose horizontal stresses that are larger from the two approaches as per Article 3.116. of AASHTO (2007 with 2009 Interims).		

Using the unit weight of the reinforced soil mass and heights Z , h , and h_{eqM} , the equation for horizontal stress at any depth Z within the MSE wall can be written as follows (also see Chapter 4):

$$\sigma_H = K_r (\gamma_r Z) \gamma_{P\text{-EV}} + K_r (\gamma_r h) \gamma_{P\text{-EV}} + K_r (\gamma_r h_{eqM}) \gamma_{P\text{-EV}} + K_r (\Delta\sigma_{v\text{-footing}}) \gamma_{P\text{-ES}} + (\Delta\sigma_H) \gamma_{P\text{-ES}}$$

$$\sigma_H = K_r [\gamma_r (Z + h + h_{eqM}) \gamma_{P\text{-EV}} + (\Delta\sigma_{v\text{-footing}}) \gamma_{P\text{-ES}}] + (\Delta\sigma_H) \gamma_{P\text{-ES}}$$

Once the horizontal stress is computed at any given level of reinforcement, the maximum tension, $T_{\max}$, is computed as follows:

$$T_{\max} = (\sigma_H)(A_{\text{trib}})$$

where A_{trib} is the tributary area for the soil reinforcement at a given level as discussed earlier.

The computations for $T_{\max}$ are illustrated at $Z = 7.27$ ft which is Level 4 in the assumed vertical layout of reinforcement. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E5-5.1.

- At $Z=7.27$ ft, the following depths are computed
 $z = Z + h = 7.27 \text{ ft} + 10.35 \text{ ft} = 17.62 \text{ ft}$
 $Z^- = 6.04 \text{ ft}$ (from Table E5-8.1)
 $Z^+ = 8.50 \text{ ft}$ (from Table E5-8.1)
 $z^- = Z^- + h = 6.04 \text{ ft} + 10.35 \text{ ft} = 16.39 \text{ ft}$
 $z^+ = Z^+ + h = 8.50 \text{ ft} + 10.35 \text{ ft} = 18.85 \text{ ft}$
- Obtain K_r by linear interpolation between $1.7K_a = 0.481$ at $z = 0.00$ ft and $1.2K_a = 0.339$ at $z = 20.00$ ft as follows:
At $z^- = 16.39$ ft, $K_{r(z^-)} = 0.339 + (20.00 \text{ ft} - 16.39 \text{ ft})(0.481 - 0.340)/20.00 \text{ ft} = 0.365$
At $z^+ = 18.85$ ft, $K_{r(z^+)} = 0.339 + (20.00 \text{ ft} - 18.85 \text{ ft})(0.481 - 0.340)/20.00 \text{ ft} = 0.348$
- Compute $\sigma_{H\text{-soil}} = [k_r \sigma_{v\text{-soil}}] \gamma_{P\text{-EV}}$ due to soil surcharge as follows:
 $\gamma_{P\text{-EV}} = 1.35$ from Table E5-5.1
At $z^- = 16.39$ ft,
 $\sigma_{v\text{-soil}(z^-)} = (0.125 \text{ kcf})(16.39 \text{ ft}) = 2.05 \text{ ksf}$
 $\sigma_{H\text{-soil}(z^-)} = [K_{r(z^-)} \sigma_{v\text{-soil}(z^-)}] \gamma_{P\text{-EV}} = (0.365)(2.05 \text{ ksf})(1.35) = 1.01 \text{ ksf}$
At $z^+ = 18.85$ ft,
 $\sigma_{v\text{-soil}(z^+)} = (0.125 \text{ kcf})(18.85 \text{ ft}) = 2.36 \text{ ksf}$
 $\sigma_{H\text{-soil}(z^+)} = [K_{r(z^+)} \sigma_{v\text{-soil}(z^+)}] \gamma_{P\text{-EV}} = (0.348)(2.36 \text{ ksf})(1.35) = 1.11 \text{ ksf}$
 $\sigma_{H\text{-soil}} = 0.5(1.01 \text{ ksf} + 1.11 \text{ ksf}) = 1.06 \text{ ksf}$
- Compute $\sigma_{H\text{-surcharge}} = [K_r q] \gamma_{P\text{-EV}}$ due to traffic (live load) surcharge as follows:
 $\gamma_{P\text{-EV}} = 1.35$ from Table E5-5.1
 $q = (\gamma_f)(h_{eqM}) = (0.125 \text{ kcf})(2.00 \text{ ft}) = 0.25 \text{ ksf}$
At $z^- = 16.39$ ft, $\sigma_{H\text{-surcharge}(z^-)} = [K_{r(z^-)} q] \gamma_{P\text{-EV}} = (0.365)(0.25 \text{ ksf})(1.35) = 0.12 \text{ ksf}$
At $z^+ = 18.85$ ft, $\sigma_{H\text{-surcharge}(z^+)} = [K_{r(z^+)} q] \gamma_{P\text{-EV}} = (0.348)(0.25 \text{ ksf})(1.35) = 0.12 \text{ ksf}$
 $\sigma_{H\text{-surcharge}} = 0.5(0.12 \text{ ksf} + 0.12 \text{ ksf}) = 0.12 \text{ ksf}$

- Compute $\sigma_{H\text{-footing}} = [K_r \Delta\sigma_{v\text{-footing}}] \gamma_{P\text{-ES}}$ as follows:

$$\Delta\sigma_{v\text{-footing}} = \frac{P_{wL}}{(b_f - 2e_f) + \left(c_f + \frac{Z}{2}\right)}$$

Method A: Use factored loads and $\gamma_{P\text{-ES}} = 1.00$

From Table E5-6.3, $b_f - 2e_f = 8.16$ ft

From Step 2, $c_f = 0.5$ ft

From Table E5-7.1, $P_{wL} = 15.03$ k/ft

From Table E5-8.1, $Z^- = 6.04$ ft and $Z^+ = 8.50$ ft

Using above values

$\Delta\sigma_{v\text{-footing}}(Z^-) = 1.29$ ksf and $\Delta\sigma_{v\text{-footing}}(Z^+) = 1.16$ ksf

$\gamma_{P\text{-ES}} = 1.00$ since the factored loads were used.

$\sigma_{H\text{-footing}}(Z^-) = [K_r(Z^-) \Delta\sigma_{v\text{-footing}}(Z^-)] \gamma_{P\text{-ES}} = (0.365)(1.29 \text{ ksf})(1.00) = 0.47$ ksf

$\sigma_{H\text{-footing}}(Z^+) = [K_r(Z^+) \Delta\sigma_{v\text{-footing}}(Z^+)] \gamma_{P\text{-ES}} = (0.348)(1.16 \text{ ksf})(1.00) = 0.40$ ksf

$\sigma_{H\text{-footing}} = 0.5(0.47 \text{ ksf} + 0.40 \text{ ksf}) = 0.44$ ksf

Method B: Use nominal loads and $\gamma_{P\text{-ES}} = 1.50$

From Table E5-6.3, $b_f - 2e_f = 8.16$ ft

From Step 2, $c_f = 0.5$ ft

From Table E5-7.1, $P_{wL} = 11.18$ k/ft

From Table E5-8.1, $Z^- = 6.04$ ft and $Z^+ = 8.50$ ft

Using above values

$\Delta\sigma_{v\text{-footing}}(Z^-) = 0.96$ ksf and $\Delta\sigma_{v\text{-footing}}(Z^+) = 0.87$ ksf

$\gamma_{P\text{-ES}} = 1.50$ since the nominal loads were used.

$\sigma_{H\text{-footing}}(Z^-) = [K_r(Z^-) \Delta\sigma_{v\text{-footing}}(Z^-)] \gamma_{P\text{-ES}} = (0.365)(0.96 \text{ ksf})(1.50) = 0.53$ ksf

$\sigma_{H\text{-footing}}(Z^+) = [K_r(Z^+) \Delta\sigma_{v\text{-footing}}(Z^+)] \gamma_{P\text{-ES}} = (0.348)(0.87 \text{ ksf})(1.50) = 0.45$ ksf

$\sigma_{H\text{-footing}} = 0.5(0.53 \text{ ksf} + 0.45 \text{ ksf}) = 0.49$ ksf

Use $\sigma_{H\text{-footing}} = 0.49$ ksf

- Compute $\Delta\sigma_H = (2F_A/L_1)\gamma_{P\text{-ES}}$ at $Z = 0$; $\Delta\sigma_H = 0$ at $Z = L_1$ as follows:

Method A: Use factored loads and $\gamma_{P\text{-ES}} = 1.00$

From Table E5-6.3, $L_1 = 16.29$ ft

From Table E5-6.2, $F_A = 5.55$ k/ft

From Table E5-8.1, $Z^- = 6.04$ ft and $Z^+ = 8.50$ ft

$\gamma_{P\text{-ES}} = 1.00$ since the factored loads were used.

$$\text{At } Z^- = 6.04 \text{ ft}$$

$$\Delta\sigma_{H(Z^-)} = [(2)(5.55 \text{ k/ft}/16.29 \text{ ft})][16.29 \text{ ft} - 6.04 \text{ ft}/16.29 \text{ ft}] [1.00] = 0.43 \text{ ksf}$$

$$\text{At } Z^+ = 8.50 \text{ ft}$$

$$\Delta\sigma_{H(Z^+)} = [(2)(5.55 \text{ k/ft}/16.29 \text{ ft})][16.29 \text{ ft} - 8.50 \text{ ft}/16.29 \text{ ft}] [1.00] = 0.33 \text{ ksf}$$

$$\Delta\sigma_H = 0.5(0.43 \text{ ksf} + 0.33 \text{ ksf}) = 0.38 \text{ ksf}$$

Method B: Use nominal loads and $\gamma_{p-ES} = 1.50$

From Table E5-6.3, $L_1 = 16.29 \text{ ft}$

From Table E5-6.2, $F_A = 3.79 \text{ k/ft}$

From Table E5-8.1, $Z^- = 6.04 \text{ ft}$ and $Z^+ = 8.50 \text{ ft}$

$\gamma_{p-ES} = 1.50$ since the nominal loads were used.

$$\text{At } Z^- = 6.04 \text{ ft}$$

$$\Delta\sigma_{H(Z^-)} = [(2)(3.79 \text{ k/ft}/16.29 \text{ ft})][16.29 \text{ ft} - 6.04 \text{ ft}/16.29 \text{ ft}] [1.50] = 0.44 \text{ ksf}$$

$$\text{At } Z^+ = 8.50 \text{ ft}$$

$$\Delta\sigma_{H(Z^+)} = [(2)(3.79 \text{ k/ft}/16.29 \text{ ft})][16.29 \text{ ft} - 8.50 \text{ ft}/16.29 \text{ ft}] [1.50] = 0.33 \text{ ksf}$$

$$\Delta\sigma_H = 0.5(0.44 \text{ ksf} + 0.33 \text{ ksf}) = 0.39 \text{ ksf}$$

Use $\Delta\sigma_H = 0.39 \text{ ksf}$

- Using values calculated above, compute $\sigma_H = \sigma_{H\text{-soil}} + \sigma_{H\text{-surcharge}} + \sigma_{H\text{-footing}} + \Delta\sigma_H$ as follows:

$$\sigma_H = 1.06 \text{ ksf} + 0.12 \text{ ksf} + 0.49 \text{ ksf} + 0.39 \text{ ksf} = 2.06 \text{ ksf}$$

- Based on Table E5-8.1, the vertical tributary spacing at Level 4 is $S_{vt} = 2.46 \text{ ft}$
- The panel width, w_p , is 10.00 ft (given in Step 1)
- The tributary area, A_{trib} , is computed as follows:

$$A_{trib} = (2.46 \text{ ft})(10.00 \text{ ft}) = 24.60 \text{ ft}^2$$
- The maximum tension at Level 4 is computed as follows:

$$T_{max} = (\sigma_H)(A_{trib}) = (2.06 \text{ ksf})(24.60 \text{ ft}^2) = 50.7 \text{ k/panel of 10 ft width}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

8.4 Establish nominal and factored long-term tensile resistance of soil reinforcement

The nominal tensile resistance of galvanized steel ribbed strip soil reinforcement is based on the design life and estimated loss of steel over the design life during corrosion.

As per Step 1, the soil reinforcement for this example is assumed to be Grade 65 ($F_y = 65$ ksi), 1.969 in. x 0.157 in. (50 mm wide x 4 mm) thick galvanized steel ribbed strips with zinc coating of 3.386 mils (86 μm). As per Step 2, the reinforced backfill meets the AASHTO (2007) requirements for electrochemical properties. For this reinforced backfill, the basis for calculating the thickness losses due to corrosion is as follows per Article 11.10.6.4.2a of AASHTO (2007):

Zinc loss = 0.58 mil for first 2 years and 0.16 mil per year thereafter

Steel loss = 0.47 mil/year/side

Based on the above corrosion rates, the following can be calculated:

$$\text{Life of zinc coating (galvanization)} = 2 \text{ years} + (3.386 - 2 \times 0.58) / 0.16 \approx 16 \text{ years}$$

As per Step 1, the design life is 100 years. The base carbon steel will lose thickness for 100 years – 16 years = 84 years at a rate of 0.47 mil/year/side. Therefore, the anticipated thickness loss is calculated as follows:

$$E_R = (0.47 \text{ mil/year/side}) (84 \text{ years}) (2 \text{ sides}) = 78.96 \text{ mils} = 0.079 \text{ in.}, \text{ and}$$

$$E_C = 0.157 \text{ in.} - 0.079 \text{ in.} = 0.078 \text{ in.}$$

Based on a 1.969 wide strip, the cross-sectional area at the end of 100 years will be equal to (1.969 in.) (0.078 in.) = 0.154 in².

For Grade 65 steel with $F_y = 65$ ksi, the nominal tensile resistance at end of 100 year design life will be $T_n = 65 \text{ ksi} (0.154 \text{ in}^2) = 10.00 \text{ k/strip}$. Using the resistance factor, $\phi_t = 0.75$ as listed in Table E5-5.2, the factored tensile resistance, $T_r = 10.00 \text{ k/strip} (0.75) = 7.50 \text{ k/strip}$.

8.5 Establish nominal and factored pullout resistance of soil reinforcement

The nominal pullout resistance, P_r , of galvanized steel ribbed strip soil reinforcement is based on various parameters in the following equation:

$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}})]$$

For this example problem, the following parameters are constant at levels of reinforcements:

$$b = 50 \text{ mm} = 1.97 \text{ in.} = 0.164 \text{ ft}$$

$\alpha = 1.0$ for inextensible reinforcement per Table 11.10.6.3.2-1 of AASHTO (2007)

The computations for P_r are illustrated at $Z = 7.27 \text{ ft}$ which is Level 4 as measured from top of the wall. Assume Strength I (max) load combination for illustration purposes and use appropriate load factors from Table E5-5.1.

- At $Z = 7.27 \text{ ft}$, $z = Z + h = 7.27 \text{ ft} + 10.35 \text{ ft} = 17.62 \text{ ft}$
- Obtain F^* at $z = 17.62 \text{ ft}$ by linear interpolation between 2.000 at $z = 0$ and 0.675 at $z = 20 \text{ ft}$ as follows:
$$F^* = 0.675 + (20.00 \text{ ft} - 17.62 \text{ ft})(2.000 - 0.675)/20 \text{ ft} = 0.832$$
- Compute effective length L_e as follows:
$$z' = H - (c_f + b_f)/0.6 = 25.5 \text{ ft} - (0.5 \text{ ft} + 10.75 \text{ ft})/0.6 = 6.75 \text{ ft}$$

Since $Z > z'$, $L_e = L - [0.6(H - Z)]$
$$L_e = 26 \text{ ft} - 0.6(25.5 \text{ ft} - 7.27 \text{ ft}) = 15.06 \text{ ft}$$
- Per Article 11.10.6.3.2 use unfactored vertical stress for pullout resistance. Thus, $\gamma_{P\text{-EV}} = 1.00$. Thus, compute $(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}}) = (0.125 \text{ kcf})(17.62 \text{ ft})(1.00) = 2.20 \text{ ksf}$
- Compute nominal pullout resistance as follows:
$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v\text{-soil}})(\gamma_{P\text{-EV}})]$$

$$P_r = (1.0)(0.832)(2)(0.164 \text{ ft})(15.06 \text{ ft})(2.20 \text{ ksf}) = 9.04 \text{ k/strip}$$
- Compute factored pullout resistance as follows:
$$P_{rr} = \phi P_r = (0.90)(9.04 \text{ k/strip}) = 8.14 \text{ k/strip}$$

Using similar computations, the various quantities can be developed at other levels of reinforcements and load combinations.

8.6 Establish number of soil reinforcing strips at each level of reinforcement

Based on $T_{\max}$, T_r and P_{rr} , the number of strip reinforcements at any given level of reinforcements can be computed as follows:

- Based on tensile resistance considerations, the number of strip reinforcements, N_t , is computed as follows:

$$N_t = T_{\max}/T_r$$

- Based on pullout resistance considerations, the number of strip reinforcements, N_p , is computed as follows:

$$N_p = T_{\max}/P_{rr}$$

Using the Level 4 reinforcement at $Z = 7.27$ ft, the number of strip reinforcements can be computed as follows:

- $T_{\max} = 50.7$ k/panel of 10 ft width, $T_r = 7.50$ k/strip, $P_{rr} = 8.14$ k/strip
- $N_t = T_{\max}/T_r = (50.7 \text{ k/panel of 10 ft width})/(7.50 \text{ k/strip}) = 6.8$ strips/panel of 10 ft width
- $N_p = T_{\max}/P_{rr} = (50.7 \text{ k/panel of 10 ft width})/(8.14 \text{ k/strip}) = 6.2$ strips/panel of 10 ft width
- Since $N_t > N_p$, tension breakage is the governing criteria and therefore the governing value, N_g , is 6.8. Round up to select 7 strips at Level 4 for each panel of 10 ft width.

The computations in Sections 8.4 to 8.6 are repeated at each level of reinforcement. Table E5-8.3 presents a summary of the computations at all levels of reinforcement for Strength I (max) load combination. The last column of Table E5-8.3 provides horizontal spacing of the reinforcing strips which is obtained by dividing the panel width, w_p , by the governing number of strips, N_g . Similar computations can be performed for Strength I (min) and Service I load combination but they will not govern the design because the load factors for these two load combinations are less than those for Strength I (max) load combination.

Note to users: All the long-form step-by-step calculations illustrated in Step 8 were based on hand-calculations in which numbers were rounded to the third or fourth significant digit as appropriate in each step. Table E5-8.3 was generated using a spreadsheet in which numbers at all calculation steps were not rounded. Thus, the end result in Table E5-8.3 may be somewhat different when compared to long-form hand calculations. However, the difference should be less than 0.2 in most cases.

Table E5-8.3. Summary of internal stability computations for Strength I (max) load combination

Level	Z ft	K _r dim	σ _H ksf	T _{max} k/10 ft wide panel	F* dim	L _e ft	φ _p (P _r) k/strip	φ _s (T _n) k/strip	N _p	N _t	N _g	S _h ft
1	1.12	0.399	2.30	39.86	1.240	14.75	7.74	7.50	5.1	5.3	6	1.7
2	2.35	0.391	2.22	41.00	1.158	14.75	8.01	7.50	5.1	5.5	6	1.7
3	4.81	0.373	2.14	52.65	0.995	14.75	8.21	7.50	6.4	7.0	8	1.3
4	7.27	0.356	2.05	50.46	0.832	15.06	8.15	7.50	6.2	6.7	7	1.4
5	9.73	0.339	1.99	48.87	0.675	16.54	8.27	7.50	5.9	6.5	7	1.4
6	12.19	0.339	1.97	48.39	0.675	18.01	10.12	7.50	4.8	6.4	7	1.4
7	14.65	0.339	1.97	48.52	0.675	19.49	12.14	7.50	4.0	6.5	7	1.4
8	17.11	0.339	2.02	49.63	0.675	20.97	14.34	7.50	3.5	6.6	7	1.4
9	19.57	0.339	2.14	52.55	0.675	22.44	16.73	7.50	3.1	7.0	8	1.3
10	22.03	0.339	2.26	55.54	0.675	23.92	19.29	7.50	2.9	7.4	8	1.3
11	24.49	0.339	2.38	53.22	0.675	25.39	22.04	7.50	2.4	7.1	8	1.3

STEP 9: DESIGN OF FACING ELEMENTS

Facing panels for true bridge abutment applications require special attention and project specific design. As per Article 11.10.11 of AASHTO (2007), due to the relatively high bearing pressures near the panel connections, the adequacy and nominal capacity of panel connections should be determined by conducting pullout and flexural tests on full-sized panels.

STEP 10: CHECK OVERALL AND COMPOUND STABILITY AT SERVICE LIMIT STATE

From Step 2, it is given that the foundation soil is dense clayey sand that has $\phi_{fd} = 30^\circ$, $\gamma_{fd} = 120$ pcf. Furthermore, the ground in front of the wall is horizontal and the foundation soil has no water table. Therefore, based on observation, overall stability is adequate. For actual projects, overall stability should be investigated at the Service I load combination and a resistance factor of 0.65.

STEP 11: DESIGN WALL DRAINAGE SYSTEMS

See Chapters 5 and 6 for wall drainage considerations. For a true bridge abutment, the drainage system for the MSE wall must be carefully integrated with the other bridge drain systems, such as the deck drainage. Often storm drain pipes are placed through the MSE wall backfill in true bridge abutments. Every attempt must be made to relocate these drain features behind the reinforced backfill.

E5-2 PRACTICAL CONSIDERATIONS

The design of a true bridge abutment is a complex process. The actual detailing of the abutment is particularly important given that a number of disciplines ranging from geotechnical, structural, hydraulics, roadway, utilities and aesthetics have specific requirements at abutment locations. All relevant input must be sought and incorporated into the project plans and specifications. Following is a general list of practical considerations from a geotechnical and structural viewpoint:

1. As noted in Article 11.10.11 of AASHTO (2007), the governing density, length, and cross-section of the soil reinforcements in Table E-4-8.3 shall be carried on the wingwalls of a minimum horizontal distance equal to 50 percent of the height of the abutment. Since the height of the abutment is 25.5 ft, the minimum horizontal distance along the wing wall is therefore $25.5 \text{ ft} / 2 = 12.5 \text{ ft}$. This dimension is greater than $c_f + b_f = 0.5 \text{ ft} + 10.75 \text{ ft} = 11.25 \text{ ft}$. Thus, the 2-way reinforcement is equal in both directions under the full width of the spread footing which provides a consistent bearing resistance across the footing.
2. Use of an approach slab is redundant for a true bridge abutment since the MSE wall and the spread footing above it settle as a unit. However, some agencies require the use of approach slab in which case special details may be necessary. Depending on the design of the approach slab system, live load may be omitted on the bridge approaches.
3. Commonly bridge abutments on spread footings on top of MSE walls are stand alone abutments with wing walls. Assuming that the wing walls are part of the MSE wall system, there will be 2-way reinforcement within the length of reinforcement perpendicular to the abutment face. It is preferable that reinforcement is not placed on top of each other in the zone of 2-way reinforcement. The overlapping reinforcement should be separated by 3 in. to 6 in. of soil or some multiple of

compacted fill height. This may be achieved by appropriately stepping of the leveling pad between the abutment face wall and the wing walls.

4. To prevent adverse stress concentrations at the reinforcement connections, the minimum vertical clearance between the bottom of the footing and the top level of reinforcement should be 1 ft.
5. In the height h_1 and h_2 shown in Figure E5-1, a false panel can be placed to cover the step in the footing. Often the coping is extended up in this area. Styrofoam or similar lightweight material which is fairly impermeable is placed in this area to minimize lateral loads on the coping or MSE panel and prevent migration of moisture into the backfill at the corrosion critical panel-reinforcement connection location.

EXAMPLE E6

TRAFFIC BARRIER IMPACT LOADING ON SEGMENTAL PRECAST PANEL MSE WALL WITH STEEL GRID REINFORCEMENT

E6-1 INTRODUCTION

This example problem is an extension of Example Problem E4, and demonstrates the analysis of a MSE wall with a traffic barrier impact load. The MSE wall is assumed to include a segmental precast panel face with steel grid (bar mat) reinforcements. The MSE wall configuration to be analyzed is shown in Figure E4-1.

The analysis is based on various principles that were discussed in Section 7.3. Table E6-1 presents a summary of steps involved in this traffic barrier analysis. The analysis uses the reinforcement spacing and sizing developed in Example E4. Practical considerations are presented in Section E6-2 after the illustration of the step-by-step procedures.

**Table E6-1. Summary of additional steps for traffic barrier impact load on
a MSE wall with level backfill and live load surcharge**

Step	Item
4	Estimate unfactored loads
5	Summarize applicable load and resistance factors
7	Evaluate internal stability of MSE wall 7.3 Calculate horizontal stress and maximum tension at each reinforcement level 7.4 Establish nominal and factored long-term tensile resistance of soil reinforcement 7.5 Check/establish nominal and factored pullout resistance of soil reinforcement 7.6 Check/establish number of soil reinforcing elements at each level of reinforcement
8	Design of facing elements

STEP 4. ESTIMATE UNFACTORED LOADS

Traffic barrier impact affects the internal stability of the reinforced soil wall. Therefore, only internal loads are discussed below. See Example E4 for external loads.

The coefficients of active lateral earth pressure for internal stability is:

$$K_{ar} = (1 - \sin 34^\circ) / (1 + \sin 34^\circ) = 0.283$$

STEP 5. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table E4-5.1 summarizes the load factors for the various LRFD load type.

Table E6-5.1. Applicable load factors

Load Combination	Load Factors (after Tables 3.4.1-1 and 3.4.1-2 (AASHTO, 2007))		
	EV	LL	CT
Strength I (maximum)	$\gamma_P = 1.35$	1.75	—
Extreme II	$\gamma_P = 1.35$	0.50	1.00

For computation of factored resistances during evaluation of extreme limits states, appropriate resistance factors have to be used. Article 11.5.7 Extreme Event Limit (AASHTO, 2007) states: *The applicable load combinations and load factors specified in Table 3.4.1-1 shall be investigated. Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme limit state.* Table E6-5.2 summarizes the applicable resistance factors.

Table E6-5.2. Summary of applicable resistance factors for evaluation of extreme limit states

Item		Resistance Factors	AASHTO (2007) Reference
Tensile resistance (for steel bar mats)	Strength Limit State	$\phi_t = 0.65$	Table 11.5.6-1
	Extreme Limit State	$\phi_t = 1.0$	Article 11.5.7
Pullout resistance	Strength Limit State	$\phi_p = 0.90$	Table 11.5.6-1
	Extreme Limit State	$\phi_t = 1.0$	Article 11.5.7

STEP 7: EVALUATE INTERNAL STABILITY ANALYSIS OF MSE WALL

Only the upper two layers of soil reinforcement are examined for the traffic barrier impact. Tensile and pullout resistance are checked.

7.2 Establish vertical layout of soil reinforcements

The following vertical layout of the top three soil reinforcements was chosen in Problem E4.

$$Z = 1.87 \text{ ft}, 4.37 \text{ ft}, \text{ and } 6.87 \text{ ft},$$

For internal stability computations, each layer of reinforcement is assigned a tributary area, A_{trib} as follows:

$$A_{trib} = (w_p)(S_{vt})$$

where w_p is the panel width of the precast facing element and S_{vt} is the vertical tributary spacing of the reinforcements based on the location of the reinforcements above and below the level of the reinforcement under consideration. The computation of S_{vt} is summarized in Table E6-7.1 wherein $S_{vt} = z^+ - z^-$. Note that $w_p = 5.00$ ft per Step 2.

Table E6-7.1. Summary of computations for S_{vt}

Level	Z (ft)	Z^- (ft)	Z^+ (ft)	S_{vt} (ft)
1	1.87	0	$1.87+0.5(4.37-1.87)=3.12$	3.12
2	4.37	$4.37-0.5(4.37-1.87)=3.12$	$4.37+0.5(6.87-4.37)=5.62$	2.50
3	6.87	$6.87-0.5(6.87-4.37)=5.62$		

7.3 Calculate horizontal stress and maximum tension at each reinforcement level

The computations for T_{max} are computed for Level 1 and 2 Reinforcements, at $z_o = 1.87$ ft and 4.37 ft, respectively. Extreme Event II load combination is used, with appropriate load factors from Table E6-5.1. The maximum load at a given level is:

$$T_{max} = \sigma_H A_{trib} + \gamma_{CT} (T_{CT})$$

where A_{trib} = tributary area for the soil reinforcement at a given level

T_{CT} = tensile load for the impact loading

The impact loads vary between reinforcement tensile rupture design and pullout design. Therefore, two maximum loads must be computed for traffic barrier impact loading – T_{max-R} and T_{max-PO} . For tensile rupture check, the upper layer of soil reinforcement is designed for a rupture impact load of 2,300 lb/ft of wall; and the second layer is designed with a rupture impact load of 600 lb/ft. Pullout is resisted over a greater length of wall than the reinforcement rupture loads. Therefore, for pullout, the upper layer of soil reinforcement is designed for a pullout impact load of 1,300 lb/ft of wall; and the second layer is designed with a pullout impact load of 600 lb/ft.

Top Layer (Level 1):

- At $Z = 1.87$ ft, the following depths are computed
 $Z^- = 0$ ft (from Table E6-7.1)
 $Z^+ = 3.12$ ft (from Table E6-7.1)
- Obtain K_r by linear interpolation between $2.5K_a = 0.707$ at $Z = 0$ and $1.2K_a = 0.340$ at $Z = 20.00$ ft as follows:
At $Z^- = 0$ ft, $K_{r(z^-)} = 0.707$
At $Z^+ = 3.12$ ft, $K_{r(z^+)} = 0.340 + (20.00 \text{ ft} - 3.12 \text{ ft})(0.707 - 0.340)/20.00 \text{ ft} = 0.650$
- Compute $\sigma_H = k_r [\gamma_r (Z + h_{eq})] \gamma_{P-EV}$ per EQ 4-34, as follows:
 $\gamma_{P-EV} = 1.35$ from Table E6-5.1
At $Z^- = 0$
 $\sigma_{H(Z^-)} = K_{r(z^-)} [\gamma_r (Z_{(z^-)} + h_{eq})] \gamma_{P-EV} = (0.707)[125 \text{ pcf} (0 + 2)](1.35) = 239 \text{ psf}$
At $Z^+ = 3.12$ ft,
 $\sigma_{H-soil(Z^+)} = K_{r(z^+)} [\gamma_r (Z_{(z^+)} + h_{eq})] \gamma_{P-EV} = (0.650)[125 \text{ pcf} (3.12 + 2)](1.35) = 562 \text{ psf}$
 $\sigma_H = 0.5(239 \text{ psf} + 562 \text{ psf}) = 400 \text{ psf}$
- Based on Table E6-7.1, the vertical tributary spacing at Level 1 is $S_{vt} = 3.12$ ft
- The panel width, w_p , is 5.00 ft (given in Step 1, Example E4)
- The tributary area, A_{trib} , is computed as follows:
 $A_{trib} = (3.12 \text{ ft})(5.00 \text{ ft}) = 15.60 \text{ ft}^2$
- The maximum tension at Level 1 with the 2,300 lb/ft impact load is computed as follows:
 $T_{\max-R} = (\sigma_H)(A_{trib}) + \gamma_{CT} (2,300 w_p) = (400 \text{ psf})(15.60 \text{ ft}^2) + 1.0 [2,300 \text{ lb/ft} (5 \text{ ft})]$
 $= 6,240 + 11,500 = 17.74 \text{ k/panel of 5-ft width}$

Second Layer (Level 2):

- At $Z = 4.37$ ft, the following depths are computed
 $Z^- = 3.12$ ft (from Table E6-7.1)
 $Z^+ = 3.12$ ft (from Table E6-7.1)

- Obtain K_r by linear interpolation between $2.5K_a = 0.707$ at $Z = 0$ and $1.2K_a = 0.340$ at $Z = 20.00$ ft as follows:

$$\text{At } Z^- = 3.12 \text{ ft, } K_{r(z^-)} = 0.340 + (20.00 \text{ ft} - 3.12 \text{ ft})(0.707 - 0.340)/20.00 \text{ ft} = 0.650$$

$$\text{At } Z^+ = 5.62 \text{ ft, } K_{r(z^+)} = 0.340 + (20.00 \text{ ft} - 5.62 \text{ ft})(0.707 - 0.340)/20.00 \text{ ft} = 0.604$$

- Compute $\sigma_H = k_r [\gamma_r (Z + h_{eq})] \gamma_{P-EV}$ per EQ 4-34, as follows:

$$\gamma_{P-EV} = 1.35 \text{ from Table E6-5.1}$$

$$\text{At } Z^- = 3.12 \text{ ft,}$$

$$\sigma_{H(Z^-)} = K_{r(z^-)} [\gamma_r (Z_{(z^-)} + h_{eq})] \gamma_{P-EV} = (0.650)[125 \text{ pcf} (3.12 + 2)](1.35) = 562 \text{ psf}$$

$$\text{At } Z^+ = 5.62 \text{ ft,}$$

$$\sigma_{H\text{-soil}(Z^+)} = K_{r(z^+)} [\gamma_r (Z_{(z^+)} + h_{eq})] \gamma_{P-EV} = (0.604)[125 \text{ pcf} (5.62 + 2)](1.35) = 777 \text{ psf}$$

$$\sigma_H = 0.5(562 \text{ psf} + 777 \text{ psf}) = 670 \text{ psf}$$

- Based on Table E6-7.1, the vertical tributary spacing at Level 2 is $S_{vt} = 2.50$ ft
- The panel width, w_p , is 5.00 ft (given in Step 1, Example E4)
- The tributary area, A_{trib} , is computed as follows:

$$A_{trib} = (2.50 \text{ ft})(5.00 \text{ ft}) = 12.50 \text{ ft}^2$$
- The maximum tension at Level 2 with the 600 lb/ft impact load is computed as follows:

$$T_{\max-R} = (\sigma_H)(A_{trib}) + \gamma_{CT} (600 w_p) = (670 \text{ psf})(12.50 \text{ ft}^2) + 1.0 [600 \text{ lb/ft} (5 \text{ ft})]$$

$$= 8,375 + 3,000 = 11.38 \text{ k/panel of 5-ft width}$$

7.4 Establish factored long-term tensile resistance of soil reinforcement for extreme event

The nominal tensile resistance of galvanized steel bar mat soil reinforcement is based on the design life and estimated loss of steel over the design life during corrosion – see Example E4 for corrosion loss calculations.

For W11 wires

$$T_n = 65 \text{ ksi} (0.0795 \text{ in}^2) = 5.17 \text{ k/wire.}$$

Using the extreme event resistance factor, $\phi_t = 1.0$ as listed in Table E6-5.2, the factored tensile resistance, $T_r = 5.17 \text{ k/wire} (1.0) = 5.17 \text{ k/wire.}$

7.5 Establish nominal and factored pullout resistance of soil reinforcement

The nominal pullout resistance, P_r , of galvanized steel bar mat (grid) reinforcement is based on various parameters in the following equation:

$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_v)(\gamma_{P-EV})]$$

$$\text{Where } \sigma_v = \gamma (Z + h_{eq})$$

Since the steel bar mat has welded connections, it can be considered inextensible with $\alpha = 1$.

Assume a W11 transverse wire which has a nominal diameter of 0.374 in. The transverse spacing of transverse wires, S_t , is equal to 6 in. for the top two layers of reinforcement, as determined in Example Problem E4.

$$\text{For, } S_t = 6 \text{ in., } t/S_t = 0.3748 \text{ in./6 in.} = 0.0623$$

Based on the value of t/S_t , the F^* parameter varies from $20(t/S_t)$ at $z = 0$ ft to $10(t/S_t)$ at $z \geq 20$ ft and greater.

$$\text{For } t/S_t = 0.0623, F^* = 1.247 \text{ at } Z = 0 \text{ ft and } F^* = 0.623 \text{ at } Z \geq 20 \text{ ft}$$

Assume bar mat width, $b = 1$ ft for computing pullout resistance on a per foot width basis. The actual bar mat width will be computed based on comparison of the pullout resistance with T_{max} . The number of longitudinal wires and thus the width of the bar mats will be determined in Example E4..

The computations for P_r are for the top two layers at $z = 1.87$ and 4.37 , respectively. For pullout, the upper layer of soil reinforcement be designed for a pullout impact load of 1,300 lb/1ft (19.0 kN/m) of wall; and the second layer be designed with a pullout impact load of 600 lb/1ft (8.8 kN/m).

Top Layer (Level 1)

- Obtain F^* at $z = 1.87$ ft by linear interpolation between 1.247 at $Z = 0$ and 0.623 at $Z = 20$ ft as follows:
$$F^* = 0.623 + (20.00 \text{ ft} - 1.87 \text{ ft})(1.247 - 0.623)/20 \text{ ft} = 1.189$$
- Compute effective length L_e as follows:
The pullout resistance of the soil reinforcement to the impact load is resisted over the full-length of the reinforcements (i.e., L).
Therefore, $L_e = L = 18 \text{ ft}$
- Compute $(\sigma_v)(\gamma_{P-EV})$
Per Article 11.10.6.3.2 of AASHTO (2007), use unfactored vertical stress for pullout resistance. Thus,
$$\gamma_{P-EV} = 1.00$$
$$(\sigma_{v\text{-soil}} + h_{eq})(\gamma_{P-EV}) = (125 \text{ pcf})(1.87 \text{ ft} + 2 \text{ ft})(1.00) = 484 \text{ psf}$$
- Compute nominal pullout resistance as follows:
$$P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v\text{-soil}})(\gamma_{P-EV})]$$
$$P_r = (1.0)(1.189)(2)(1.00 \text{ ft})(18 \text{ ft})(484 \text{ psf}) = 20,717 \text{ lb/ft}$$
- Compute factored pullout resistance as follows:
$$P_{rr} = \phi P_r = (1.0)(20.72 \text{ k/ft}) = 20.72 \text{ k/ft}$$
- The maximum pullout tension at Level 1 with the 600 lb/ft impact load and σ_H from Step 7.3 is computed as follows:
$$T_{\text{max-PO}} = (\sigma_H)(A_{\text{trib}}) + \gamma_{CT} (1,300 \text{ w}_p) = (400 \text{ psf})(15.60 \text{ ft}^2) + 1.0 [1,300 \text{ lb/ft} (5 \text{ ft})]$$
$$= 6,240 + 6,500 = 12.74 \text{ k/panel of 5-ft width}$$

Second Layer (Level 2)

- Obtain F^* at $z = 4.37$ ft by linear interpolation between 1.247 at $Z = 0$ and 0.623 at $Z = 20$ ft as follows:
$$F^* = 0.623 + (20.00 \text{ ft} - 4.37 \text{ ft})(1.247 - 0.623)/20 \text{ ft} = 1.111$$
- Compute effective length L_e as follows:
The pullout resistance of the soil reinforcement to the impact load is resisted over the full-length of the reinforcements (i.e., L).
Therefore, $L_e = L = 18 \text{ ft}$

- Compute $(\sigma_v)(\gamma_{P-EV})$
Per Article 11.10.6.3.2 of AASHTO (2007), use unfactored vertical stress for pullout resistance. Thus,
 $\gamma_{P-EV} = 1.00$
 $(\sigma_{v-soil} + h_{eq})(\gamma_{P-EV}) = (125 \text{ pcf})(4.37 \text{ ft} + 2 \text{ ft})(1.00) = 796 \text{ psf}$
- Compute nominal pullout resistance as follows:
 $P_r = \alpha(F^*)(2b)(L_e)[(\sigma_{v-soil})(\gamma_{P-EV})]$
 $P_r = (1.0)(1.111)(2)(1.00 \text{ ft})(18 \text{ ft})(796 \text{ psf}) = 31,836 \text{ lb/ft}$
- Compute factored pullout resistance as follows:
 $P_{rr} = \phi P_r = (1.0)(31.84 \text{ k/ft}) = 31.84 \text{ k/ft}$
- The maximum pullout tension at Level 2 with the 600 lb/ft impact load and σ_H from Step 7.3 is computed as follows:
 $T_{\max-PO} = (\sigma_H)(A_{trib}) + \gamma_{CT} (600 \text{ w}_p) = (670 \text{ psf})(12.50 \text{ ft}^2) + 1.0 [600 \text{ lb/ft} (5 \text{ ft})]$
 $= 8,375 + 3,000 = 11.38 \text{ k/panel of 5-ft width}$

7.6 Establish number of longitudinal wires at each level of reinforcement

Based on $T_{\max-R}$, T_r , $T_{\max-PO}$, and P_{rr} , the number of longitudinal wires at any given level of reinforcements can be computed as follows:

- Based on tensile resistance considerations, the number of longitudinal, N_t , is computed as follows:

$$N_t = T_{\max-R}/T_r$$

- Based on pullout resistance considerations, the number of longitudinal wires, N_p , is computed as follows:

$$N_p = 1 + (T_{\max-PO}/P_{rr})/(S_l)$$

Top Layer (Level 1) Reinforcement at $Z = 1.87 \text{ ft}$, the number of W11 longitudinal wires for 5 ft wide panel can be computed as follows:

- $T_{\max-R} = 17.74 \text{ k/panel of 5-ft width}$, $T_r = 5.17 \text{ k/wire}$

- $N_t = T_{\max-R}/T_r = (17.74 \text{ k/panel of 5-ft width})/(5.17 \text{ k/wire}) = 3.4$ longitudinal wires/panel of 5-ft width
- $N_p = T_{\max-PO}/P_{rr} = 1 + [(12.74 \text{ k/panel of 5-ft width})/(20.72 \text{ k/ft})]/(0.5 \text{ ft})$
 $= 1 + 1.4 = 2.4$ longitudinal wires/panel
- Since $N_t > N_p$, tension breakage is the governing criteria and therefore the governing value, N_g , is 4.0. Select 4 longitudinal wires at Level 1 for each panel of 5-ft width. Thus, the Strength I steel bar mat configuration at Level 1 of 4W11 + W11x0.5' is sufficient for the Extreme Event II traffic barrier loading.

Second Layer (Level 2) Reinforcement at $Z = 4.37$ ft, the number of W11 longitudinal wires for 5 ft wide panel can be computed as follows:

- $T_{\max-R} = 11.38 \text{ k/panel of 5-ft width}$, $T_r = 5.17 \text{ k/wire}$,
- $N_t = T_{\max-R}/T_r = (11.38 \text{ k/panel of 5-ft width})/(5.17 \text{ k/wire}) = 2.2$ longitudinal wires/panel of 5-ft width
- $N_p = T_{\max-PO}/P_{rr} = 1 + [(11.38 \text{ k/panel of 5-ft width})/(31.84 \text{ k/ft})]/(0.5 \text{ ft})$
 $= 1 + 0.87 = 1.7$ longitudinal wires/panel
- Since $N_t > N_p$, tension breakage is the governing criteria and therefore the governing value, N_g , is 4.0. Select 3 longitudinal wires at Level 2 for each panel of 5-ft width. Thus, the Strength I steel bar mat configuration at Level 2 of 3W11 + W11x0.5' is sufficient for the Extreme Event II traffic barrier loading.

STEP 8: DESIGN OF FACING ELEMENTS

The precast facing elements must be designed as structural elements with appropriate connection strength as discussed in Chapter 4. The upper facing panel should be separated from the barrier slab with 1 to 2 in. of expanded polystyrene (see Figure 5-2(b)). The distance should be adequate to allow the barrier and slab to resist the impact load in sliding and overturning without loading the facing panel.

EXAMPLE E7

SEGMENTAL PRECAST PANEL MSE WALL WITH SEISMIC LOADING

E7-1 INTRODUCTION

This example problem demonstrates the analysis of the Example #4 MSE wall for earthquake loading. The MSE wall has a level backfill and live load surcharge, and is assumed to include a segmental precast panel face with steel grid (bar mat) reinforcements. The MSE wall configuration to be analyzed is shown in Figure E4-1. The analysis is based on various principles that were discussed in Chapter 7. Table E7-1 presents a summary of steps involved in the analysis. Each of the steps and sub-steps is sequential and if the design is revised at any step or sub-step then all the previous computations need to be re-visited. Each of the steps and the sub-steps in Table E7-1 is explained in detail herein.

Table E7-1. Summary of steps in analysis of MSE wall with seismic loading

Step	Item
GENERAL	
1	Complete static analysis/design
2	Summarize applicable load and resistance factors
EXTERNAL STABILITY	
1	Establish initial wall design based on static loading
2	Establish seismic hazard, and estimate peak ground acceleration (PGA) and spectral acceleration at 1-second, S_1 .
3	Establish site effects
4	Determine maximum accelerations, k_{max} , and peak ground velocity (PVG)
5	Obtain an average peak ground acceleration, k_{av} , within the reinforced soil zone
6	Determine the total (static + dynamic) thrust P_{AE} , using one of the following two methods 1 Mononobe-Okabe (M-O) formulation 2 Generalized Limit Equilibrium (GLE) slope stability
7	Determine the horizontal inertial force, P_{IR} , of the total reinforced wall mass
8	Check sliding stability ⇒ if sliding stability is met, go to Step 11 ⇒ if sliding stability is not met, go to Step 9
9	Determine the wall yield seismic coefficient, k_v , where wall sliding is initiated.
10	Determine the wall sliding displacement based on the following relationships between d , k_v/k_{max} , k_{max} , PGV, and site location
11	Evaluate the limiting eccentricity and bearing resistance
12	If Step 11 criteria are not met, adjust the wall geometry and repeat Steps 6 to 11, as needed
13	If Step 11 criteria are met, assess acceptability of amount of sliding displacement

INTERNAL STABILITY	
1	Compute the internal dynamic force, P_i , of the active wedge
2	Compute maximum combined factored loads in the soil reinforcements
3	Check tensile resistance of soil reinforcements
4	Check pullout resistance of the soil reinforcements
5	Check connection resistance

GENERAL

STEP 1. COMPLETE STATIC ANALYSIS/DESIGN

Initial wall design based upon static loading was established in Example E4.

STEP 2. SUMMARIZE APPLICABLE LOAD AND RESISTANCE FACTORS

Table E7-2 summarizes the load factors for the various LRFD load type, including seismic (extreme event I), shown in second column of Tables E4-4.1 and E4-4.2. **Throughout the computations in this example problem, the forces and moments in Tables E4-4.1 and E4-4.2 should be multiplied by appropriate load factors.**

Table E7-2. Summary of applicable load factors

Load Combination	Load Factors (after AASHTO, 2007 Tables 3.4.1-1 and 3.4.1-2)			
	EV	EH	LS	EQ
Extreme Event I	γ_p	γ_p	$\gamma_{EQ} = 1.00$	1.00
Strength I (maximum)	$\gamma_p = 1.35$	$\gamma_p = 1.50$	1.75	—
Strength I (minimum)	$\gamma_p = 1.00$	$\gamma_p = 0.90$	1.75	—
Service I	1.00	1.00	1.00	—

For computation of factored resistances during evaluation of extreme event I limits states, appropriate resistance factors have to be used. Table E7-3 summarizes the applicable resistance factors.

Table E7-3. Summary of applicable combined static/earthquake resistance factors

Item	Resistance Factors	AASHTO (2007) Reference
Sliding of MSE wall on foundation soil	$\phi_s = 1.00$	Table 11.5.6-1
Bearing resistance	$\phi_b = 1.00$	Article 10.5.5.3.3
Tensile resistance (for steel bar mats)	$\phi_t = 0.85$	Table 11.5.6-1
Pullout resistance	$\phi_p = 1.20$	Table 11.5.6-1

EVALUATE XTERNAL STABILITY ANALYSIS OF MSE WALL

STEP 1. ESTABLISH INITIAL WALL DESIGN

Initial wall design based upon static loading was established in Example E4.

STEP 2. ESTABLISH SEISMIC HAZARD, AND ESTIMATE PEAK GROUND ACCELERATION (PGA) AND SPECTRAL ACCELERATION AT 1-SECOND, S_1 .

The USGS Seismic Design Parameters CD (Version 2.10) {provided with AASHTO LRFD Bridge Specifications) was used to determine these design parameters. An assumed location of Latitude 40.66 and Longitude -111.51 was used for this design example. The following parameters were established:

- $PGA = 0.206 \text{ g}$
- $S_1 \text{ at } 1.0 \text{ sec Period} = 0.177 \text{ g}$

STEP 3. ESTABLISH SITE EFFECTS

From the assumed location (Latitude and Longitude) and the USGS Seismic Design Parameters CD (Version 2.10). The following Site Class was found.

- Site Class B

From AASHTO Table 3.10.3.2-1, and using Site Class B and $PGA = 0.206\text{g}$, the F_{pga} value at zero-period on acceleration spectrum is established.

- $F_{pga} = 1.00$

From AASHTO Table 3.10.3.2-3, and using Site Class B and $S_1 = 0.177\text{g}$, F_v value is established.

- $F_v = 1.0$

STEP 4. DETERMINE MAXIMUM ACCELERATIONS, k_{max} , AND PEAK GROUND VELOCITY (PVG)

With Equation 7-1:

$$k_{max} = F_{pga}(PGA) = 1.00(0.206 \text{ g}) = 0.206 \text{ g}$$

With Equation 7-2:

$$\text{PGV (in/sec)} = 38 F_v S_1 = 38 (1.0) (0.177 \text{ g}) = 6.726$$

**STEP 5. OBTAIN AN AVERAGE PEAK GROUND ACCELERATION, k_{av} ,
WITHIN THE REINFORCED SOIL ZONE**

The average peak ground acceleration, using a wall height dependent reduction factor, α , within the reinforced soil zone, using Equation 7-3, is equal to:

$$k_{av} = \alpha k_{max}$$

For Site Class B and using Equation 7-4 the wall height factor, α , is equal to:

$$\alpha = 120\% \left\{ 1 + 0.01H \left[0.5 \left(\frac{F_v S_1}{k_{max}} \right) - 1 \right] \right\} = 120\% \left\{ 1 + 0.01 (25.64 \text{ ft}) \left[0.5 \left(\frac{1.0 \times 0.177 \text{ g}}{0.206 \text{ g}} \right) - 1 \right] \right\}$$

$$\alpha = 1.0245$$

Therefore,

$$k_{av} = \alpha k_{max} = 1.024 (0.206 \text{ g}) = 0.211 \text{ g}$$

STEP 6. DETERMINE THE TOTAL (STATIC + DYNAMIC) THRUST P_{AE}

The total (static + dynamic) thrust P_{AE} , may be determined using one of the following two methods

- 1 Mononobe-Okabe (M-O) formulation
- 2 Generalized Limit Equilibrium (GLE) slope stability

The M-O formulation is used for this design example.

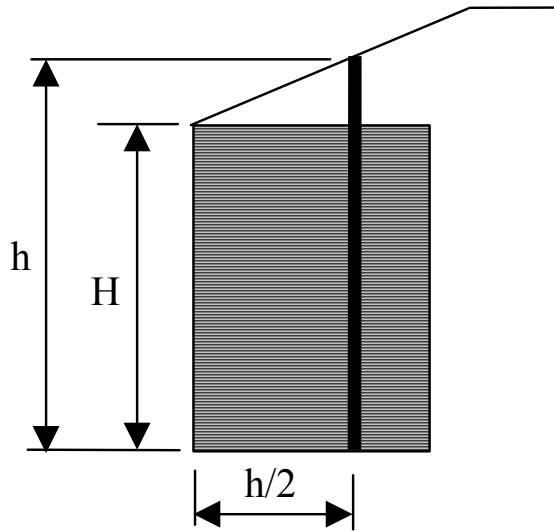
Assumptions:

- $k_v = 0$
- $k_h = k_{max} = 0.206 \text{ g}$

With the Mononobe-Okabe (M-O) formulation (see Equation 7-5):

$$P_{AE} = 0.5(K_{AE})\gamma_f h^2$$

where h is the wall height along the vertical plane within the reinforced soil mass as shown below (and in Figure 7-1), γ_b is the unit weight of the retained backfill and K_{AE} is obtained as per Equation 7-6, as follows.



$$h = H + \frac{\tan I (0.5H)}{(1 - 0.5 \tan I)} ; \text{ where } I \text{ is the backfill slope angle}$$

$$h = 25.64 + \frac{\tan 0 (0.5) 25.64}{[1 - (0.5) \tan 0]} = 25.64 \text{ ft}$$

Per Equation 7-6, K_{AE} is equal to:

$$K_{AE} = \frac{\cos^2(\phi'_b - \xi - 90 + \theta)}{\cos \xi \cos^2(90 - \theta) \cos(\delta + 90 - \theta + \xi) \left[1 + \sqrt{\frac{\sin(\phi'_b + \delta) \sin(\phi'_b - \xi - I)}{\cos(\delta + 90 - \theta + \xi) \cos(I - 90 + \theta)}} \right]^2}$$

with

$$\xi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) = \tan^{-1}\left(\frac{0.206 g}{1-0}\right) = 11.64$$

δ = angle of wall friction = lesser of the angle of friction for the reinforced soil mass (ϕ'_r) and the retained backfill (ϕ'_b) = 30°

I = the backfill slope angle = $\beta = 0^\circ$

ϕ'_b = angle of internal friction for retained backfill = 30°

θ = the slope angle of the face = 90°

$$K_{AE} = \frac{\cos^2(30-11.64-90+90)}{\cos 11.64 \cos^2(90-90) \cos(30+90-90+11.64) \left[1 + \sqrt{\frac{\sin(30+30) \sin(30-11.64-0)}{\cos(30+90-90+11.64) \cos(0-90+90)}} \right]^2}$$

$$K_{AE} = \frac{\cos^2(18.36)}{\cos 11.64 \cos(41.64) \left[1 + \sqrt{\frac{\sin(60) \sin(18.36)}{\cos(41.64)}} \right]^2} = \frac{0.9008}{0.7320 [1 + 0.6042]^2} = 0.4782$$

Therefore

$$P_{AE} = 0.5(K_{AE})\gamma_b h^2 = 0.5(0.4782)(125 \text{ lb/ft}^3)(25.64 \text{ ft})^2 = 19.65 \text{ k/ft}$$

STEP 7 DETERMINE THE HORIZONTAL INERTIA FORCE, P_{IR}

Determine the horizontal inertial force, P_{IR} , of the total reinforced wall mass with Equation 7-7, as follows:

$$P_{IR} = 0.5(k_{av})(W)$$

where W is the weight of the full reinforced soil mass and any overlying permanent slopes and/or permanent surcharges within the limits of the reinforced soil mass. The inertial force is assumed to act at the centroid of the mass used to determine the weight W .

$$W = 25.64 \text{ ft} (18 \text{ ft}) (125 \text{ pcf}) = 57.68 \text{ k/ft}$$

$$P_{IR} = 0.5(k_{av})(W) = 0.5 (0.211 \text{ g}) (57.68 \text{ k/ft}) = 6.09 \text{ k/ft}$$

STEP 8 CHECK SLIDING STABILITY

From page 7-6, check the sliding stability using a resistance factor, ϕ_τ , equal to 1.0 and the full, nominal weight of the reinforced zone and any overlying permanent surcharges. If the sliding stability is met, the design is satisfactory and go to Step 11. If not, go to Step 9.

Compute the total horizontal force, T_{HF} , for M-O Method as follows:

$$T_{HF} = \text{Horizontal component of } P_{AE} + P_{IR} + \gamma_{EQ}(q_{LS})(H)(K_{AE}) + \text{other horizontal nominal forces due to surcharges (with load factor}=1.0)$$

where, γ_{EQ} is the load factor for live load in Extreme Event I limit state and q_{LS} is the intensity of the live load surcharge.

$$\begin{aligned} T_{HF} &= P_{AE} \cos\delta + P_{IR} + \gamma_{EQ}(q_{LS}) \\ &= 19.65 \text{ k/ft} (\cos 30^\circ) + 6.09 \text{ k/ft} + 0.5(0.25 \text{ ksf})(25.64 \text{ ft})(0.4785) = \\ &= 17.02 \text{ k/ft} + 6.09 \text{ k/ft} + 1.53 \text{ k/ft} \\ &= 24.64 \text{ k/ft} \end{aligned}$$

Compute the sliding resistance, R_τ , as follows:

$$R_\tau = \Sigma V (\mu)$$

where μ is the minimum of $\tan\phi'_r$, $\tan\phi'_f$ or (for continuous reinforcement) $\tan\phi$ (as discussed in Section 4.5.6.a) and ΣV is the summation of the vertical forces as follows:

$$\Sigma V = W + P_{AE}\sin\delta + \text{permanent nominal surcharge loads within the limits of the reinforced soil mass}$$

$$\Sigma V = W + P_{AE}\sin\delta = 57.68 \text{ k/ft} + 19.65 \text{ k/ft} (\sin 30^\circ) = 57.68 + 9.84 = 67.52 \text{ k/ft}$$

$$R_\tau = \Sigma V (\mu) = 67.52 \text{ k/ft} (\tan 30^\circ) = 38.98 \text{ k/ft}$$

The sliding stability capacity to demand (CDR) ratio is calculated as:

$$CDR_{\text{sliding}} = R_\tau / T_{HF} = 38.98 \text{ k/ft} / 24.64 \text{ k/ft} = 1.58 > 1.0 \therefore \text{O.K., and go to Step 11}$$

STEP 11. EVALUATE ECCENTRICITY AND BEARING RESISTANCE

Evaluate the limiting eccentricity and bearing resistance. Include all applicable loads for Extreme Event I. For the M-O method, add other applicable forces to P_{AE} . Check the limit states using the following criteria:

1. For limiting eccentricity, for foundations on soil and rock, the location of the resultant of the applicable forces should be within the middle two-thirds of the wall base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths of the wall base for $\gamma_{EQ} = 1.0$. Interpolate linearly between these values as appropriate.
2. For bearing resistance compare the bearing pressure to the nominal bearing resistance (i.e., use a resistance factor of 1.0) based on full width of the reinforced zone.

11.1 Limiting Eccentricity at Base of MSE Wall

The purpose of these computations is to evaluate the limiting eccentricity at the base of the MSE wall. Since the computations are related to limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. The computations for limiting eccentricity at the base of the MSE wall are illustrated in Table E4-6.2. Limiting eccentricity is a strength limit state check and therefore service limit state calculations are not performed. The critical values based on max/min result in the extreme force effect and govern the limiting eccentricity mode of failure.

Table E7-4. Computations for evaluation of limiting eccentricity for MSE wall

Item	Unit	Extreme Event I	
		($\gamma_p = \text{max}$)	($\gamma_p = \text{min}$)
Total vertical load at base of MSE wall without LL, $V_A = V_1$	k/ft	77.88	57.69
$P_{AE} \sin \delta$	k/ft	9.84	9.84
Resisting moments about Point A without LL surcharge = $M_{RA} = M_1 V_1 + L (P_{AE} \sin \delta)$	k-ft/ft	878.05	696.33
Total thrust seismic overturning moment about Pt A = $P_{AE} \cos \delta (h/2) = 19.65 \text{ k/ft } (\cos 30^\circ) (25.64/2 \text{ ft})$	k-ft/ft	218.16	218.16
LL component overturning moment about Point A = $1.53 \text{ k/ft } (25.64/2 \text{ ft})$	k-ft/ft	19.65	19.65
Inertial force, P_{IR} overturning moment about Pt A = max: $6.09 \text{ k/lft } (1.35)(25.64/2 \text{ ft})$ min: $6.09 \text{ k/lft } (1.00)(25.64/2 \text{ ft})$	k-ft/ft	105.36	78.04
Overturning moment, M_{OA} (static + seismic)	k-ft/ft	343.17	315.85
Net moment about Point A = $M_A = M_{RA} - M_{OA}$	k-ft/ft	534.88	380.48
Location of the resultant force on base of MSE wall from Point A, $a = M_A / (V_A + P_{AE} \sin \delta)$	ft	6.10	5.63
Eccentricity at base of MSE wall, $e_L = L/2 - a$	ft	2.9	3.37
Limiting eccentricity, $e = 0.40L$ for extreme event I limit state	ft	6.60	6.60
Is the resultant within limiting value of e ?	-	Yes	Yes
Calculated e_L/L	-	0.16	0.19
CRITICAL VALUES BASED ON MAX/MIN			
Overturning moments about Point A, M_{OA-C}	k-ft/ft	343.17	
Resisting moments about Point A, M_{RA-C}	k-ft/ft	696.33	
Net moment about Point A, $M_{A-C} = M_{RA-C} - M_{OA-C}$	k-ft/ft	353.16	
Vertical force, $V_{A-C} = V_1 + P_{AE} \sin \delta$	k/ft	67.52	
Location of resultant from Point A, $a_{nl} = M_{A-C} / V_{A-C}$	ft	5.23	
Eccentricity from center of wall base, $e_L = 0.5 * L - a_{nl}$	ft	3.76	
Limiting seismic eccentricity, $e = 0.4L$	ft	7.20	
Is the limiting eccentricity criteria satisfied?	-	Yes	

11.2 Bearing Resistance at Base of MSE Wall

For bearing resistance computations, the effect of live load is included since it creates larger bearing stresses. For seismic bearing resistance compare the bearing pressure to the nominal bearing resistance (i.e., use a resistance factor of 1.0) based on full width of the reinforced zone. Therefore, seismic bearing stress at the base of the MSE wall can be computed as follows:

$$\sigma_v = \frac{\Sigma V}{L - 2e}$$

where $\Sigma V = R = V_1 + V_S + P_{AE} \sin \delta$ is the resultant of vertical.

Table E7-5. Computations for evaluation of seismic bearing resistance
(see Table E4-6.3. for static values)

Item	Unit	Extreme Event I ($\gamma_p = \max$)
Static Vertical load at base of MSE wall including LL on top, $\Sigma V = R = V_1 + V_S$	k/ft	80.13
Seismic Vertical load at base of MSE wall, $P_{AE} \sin \delta$	k/ft	9.84
Total vertical load at base of MSE wall without LL, $V_A = R = V_1 + V_S + P_{AE} \sin \delta$	k/ft	89.97
Resisting moments about Point A = M_{RA} $= MV_1 + MV_S + MP_V$	k-ft/ft	898.03
Overturning moments about Point A = M_{OA} $= MF_1 + MF_2 + MF_{IR}$	k-ft/ft	343.17
Net moment at Point A, $M_A = M_{RA} - M_{OA}$	k-ft/ft	554.86
Location of the resultant force on MSE block from Point A, $a = (M_{RA} - M_{OA})/V_A$	ft	6.17
Eccentricity at base of MSE block, $e_L = L/2 - a$	ft	2.83
Limiting e_L	ft	4.50
Is the resultant within limiting value of e_L ?	ft	YES
Effective width of base of MSE wall, $B' = L - 2e$	ft	12.34
Bearing stress due to MSE wall $= \Sigma V/(B') = \sigma_v$	ksf	7.29
Bearing resistance (10.50 ksf given in E4 for static resistance, $\phi_{\text{seismic}} / \phi_{\text{static}} = 1.0/0.65$, therefore, seismic resistance = 16.15 ksf)	ksf	16.15
Is bearing stress less than the bearing resistance?	—	YES

EVALUATE INTERNAL STABILITY ANALYSIS OF MSE WALL

STEP 1. COMPUTE INTERNAL DYNAMIC FORCE, P_i

For internal stability, the active wedge is assumed to develop an internal dynamic force, P_i , that is equal to the product of the mass in the active zone and the wall height dependent average seismic coefficient, k_{av} . P_i is computed as:

$$P_i = k_{av} W_a$$

W_a is the soil weight of the active zone as shown by shaded area in Figure E7-1.

$$W_a = [0.3H(H/2) + 0.5(0.3H)(H/2)] \gamma_r$$

$$W_a = [0.3 (25.64 \text{ ft})^2(1/2) + 0.5(0.3)(25.64 \text{ ft})^2(1/2)] 125 \text{ pcf}$$

$$W_a = [98.61 \text{ ft}^2 + 49.31 \text{ ft}^2] 125 \text{ pcf} = 18.49 \text{ k/ft}$$

$$P_i = 0.211g (18.49 \text{ k}) = 3.90 \text{ k/ft}$$

The inertial force is distributed to the n number of reinforcement layers equally as follows. From Example E4, $n = 10$.

$$T_{md} = \frac{P_i}{n} = \frac{3.90 \text{ k / lft}}{10} = 0.39 \text{ k / lft} = 1.95 \text{ k / 5 ft panel width}$$

STEP 2. COMPUTE MAXIMUM FACTORED LOADS IN THE REINFORCEMENTS

The load factor for seismic forces is equal to 1.0. The total factored load applied to the reinforcement on a load per unit of wall width basis is determined as follows, with T_{max} the factored static load applied to the reinforcements determined using the appropriate equations in Chapters 4 and 6.

$$T_{total} = T_{max} + T_{md}$$

T_{total} for each layer of reinforcement is listed in Table E7-6, with T_{max} from Table E4-7.4

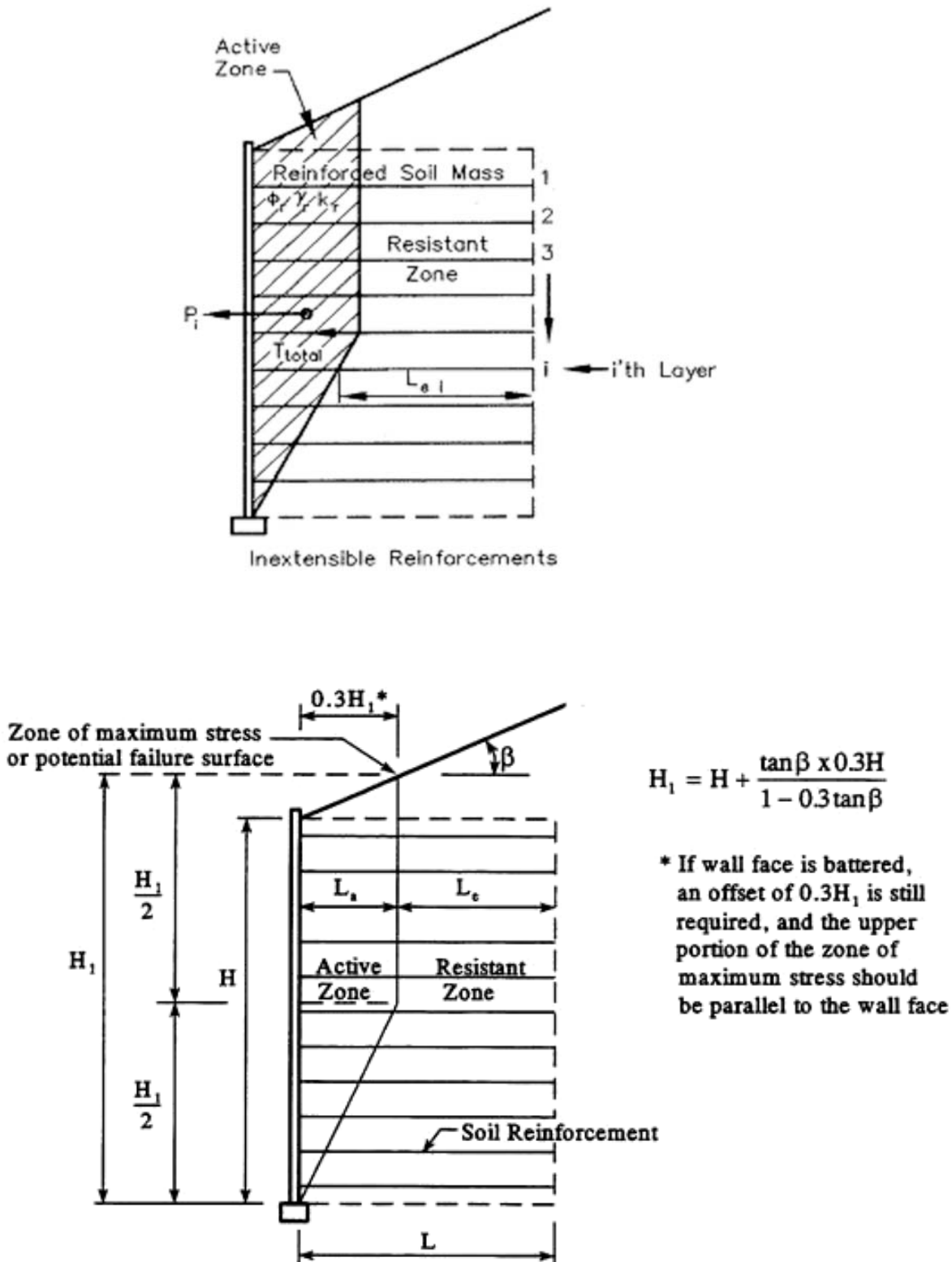


Figure E7-1. Seismic internal stability for inextensible reinforcements.
(from Figure 4-9 and Figure 7-5)

STEP 3. CHECK TENSILE RESISTANCE OF THE SOIL REINFORCEMENTS

As listed in Table E7-3, the resistance factor for metallic bar mats while evaluating tensile failure under combined static and earthquake loading is equal to 0.85.

T_{total} for each layer of reinforcement is listed in Table E7-6, with T_{max} and N_g from Table E4-7.4.

**Table E7-6. Summary of tensile resistance computations for
Extreme Event I load combination**

Level	Z ft	σ_H ksf	T_{max} kips/ 5 ft panel	T_{md}	T_{total}	T_n k/wire	ϕ	Bar Mat	N_g	$(\phi T_n) N_g$ k/5' panel	CDR
1	1.87	0.40	6.25	1.95	8.20	5.17	0.85	4W11	4	17.58	2.14
2	4.37	0.67	8.36	1.95	10.31	5.17	0.85	3W11	3	13.18	1.28
3	6.87	0.87	10.80	1.95	12.75	5.17	0.85	4W11	4	17.58	1.38
4	9.37	1.03	12.77	1.95	14.72	5.17	0.85	4W11	4	17.58	1.19
5	11.87	1.15	14.26	1.95	16.21	7.42	0.85	4W15	4	25.23	1.56
6	14.37	1.23	15.23	1.95	17.18	7.42	0.85	4W15	4	25.23	1.47
7	16.87	1.26	15.71	1.95	17.66	7.42	0.85	4W15	4	25.23	1.43
8	19.37	1.27	16.03	1.95	17.98	7.42	0.85	4W15	4	25.23	1.40
9	21.87	1.37	17.10	1.95	19.05	7.42	0.85	4W15	4	25.23	1.32
10	24.37	1.51	19.05	1.95	21.00	7.42	0.85	4W15	4	25.23	1.20

Capacity to demand ratios (CDR) are greater than 1.00, therefore, tensile resistance under seismic loading is adequate for all layers of soil reinforcement.

STEP 4. CHECK PULLOUT RESISTANCE OF THE SOIL REINFORCEMENTS

The seismic tensile and total tensile loads are summarized below in Table E7-7. The static factor pullout resistance values listed below, in terms of k/ft, are from Table E4-7.4.

For seismic loading conditions, the value of F^* , the pullout resistance factor, is reduced to 80 percent of the value used for static design, unless dynamic pullout tests are performed to directly determine the F^* value. Therefore, the static values listed must be reduced by 80% to determine seismic pullout resistance.

The pullout resistance factor is equal to 0.9 and to 1.20 for static and for seismic conditions, respectively. Therefore, the factored static values listed must be multiplied by a ratio of 1.20/0.9 to determine factored seismic pullout resistance.

T_{total} is presented in terms of kips per 5-foot panel width. Therefore, the static pullout values listed must be multiplied by 5 to go from a per foot to a per panel basis.

**Table E7-7. Summary of pullout computations for
Extreme Event I load combination**

Level	Z ft	σ_H ksf	T_{max} kips/ 5 ft panel	T_{md} kips/ 5 ft panel	T_{total} kips/ 5 ft panel	Static ϕP_r k/ft	$\times$ 80%	$\times$ 1.20 /0.9	$\times$ 5 ft	Seismic ϕP_r k/5' panel	CDR
1	1.87	0.44	6.25	1.95	8.20	5.16	0.8	1.33	5	27.52	3.36
2	4.37	0.67	8.36	1.95	10.31	11.25	0.0	1.33	5	60.00	5.82
3	6.87	0.87	10.80	1.95	12.75	16.47	0.8	1.33	5	87.84	6.89
4	9.37	1.03	12.77	1.95	14.72	20.75	0.8	1.33	5	110.7	7.52
5	11.87	1.15	14.26	1.95	16.21	12.06	0.8	1.33	5	64.32	3.97
6	14.37	1.23	15.23	1.95	17.18	14.50	0.8	1.33	5	77.33	4.50
7	16.87	1.26	15.71	1.95	17.66	17.41	0.8	1.33	5	92.85	5.26
8	19.37	1.27	16.03	1.95	17.98	13.27	0.8	1.33	5	70.77	3.94
9	21.87	1.37	17.10	1.95	19.05	16.12	0.8	1.33	5	85.97	4.51
10	24.37	1.51	19.05	1.95	21.00	19.66	0.8	1.33	5	104.8	4.99

Capacity to demand ratios (CDR) are greater than 1.00, therefore, pullout resistance under seismic loading is adequate for all layers of soil reinforcement.

STEP 5: CHECK CONNECTION RESISTANCE

The precast facing elements must be designed as structural elements with appropriate connection strength as discussed in Chapter 4. For the Extreme Event I limit state the connection, at each level, must be designed to resist the total (static + seismic) factored load, T_{total} . The factored long-term connection strength, ϕT_{ac} , must be greater than T_{total} . The resistance factor for combined static and seismic loads for steel grid reinforcement is 0.85 (the static resistance factor is 0.65).

EXAMPLE E8

REINFORCED SOIL SLOPE DESIGN – ROAD WIDENING

E8-1 INTRODUCTION

A 0.6 mile (1 km) long, 16.5 ft (5 m) high, 2.5H:1V side slope road embankment in a suburban area is to be widened by one lane. At least a 20 ft (6.1 m) width extension is required to allow for the additional lane plus shoulder improvements. A 1H:1V reinforced soil slope up from the toe of the existing slope will provide 25 ft (7.6 m) width to the alignment. The following provides the steps necessary to perform a preliminary design for determining the quantity of reinforcement to evaluate the feasibility and cost of this option. The reader is referred to the design steps in section 9.2 to more clearly follow the meaning of the design sequence.

STEP 1. SLOPE DESCRIPTION

a. Geometric and load requirements

- $H = 16.5 \text{ ft (5 m)}$
- $\beta = 45^\circ$
- $q = 200 \text{ psf (10 kPa)}$ (for dead weight of pavement section) + 2% road grade

b. Performance requirements

- External Stability:
Sliding Stability: $FS_{\min} = 1.3$
Overall slope stability and deep seated: $FS_{\min} = 1.3$
Dynamic loading: no requirement
Settlement: analysis required
- Compound Failure: $FS_{\min} = 1.3$
- Internal Stability: $FS_{\min} = 1.3$

STEP 2. ENGINEERING PROPERTIES OF FOUNDATION SOILS

- Review of soil borings from the original embankment construction indicates foundation soils consisting of stiff to very stiff, low-plasticity, silty clay with interbedded seams of sand and gravel. The soils tend to increase in density and strength with depth.
- $\gamma_d = 121 \text{ lb/ft}^3$ (19 kN/m³), $\omega_{\text{opt}} = 15\%$, $c_u = 2000 \text{ psf}$ (96 kPa), $\phi' = 28^\circ$, and $c' = 0$
- At the time of the borings, $d_w = 6.6 \text{ ft}$ (2 m) below the original ground surface.

STEP 3. PROPERTIES OF REINFORCED AND EMBANKMENT FILL

The existing embankment fill is a clayey sand and gravel. For preliminary evaluation, the properties of the embankment fill are assumed for the reinforced section as follows:

U.S. Sieve Size	Percent passing
4 in. (100 mm)	100
¾ in. (20 mm)	99
No. 4 (4.75 mm)	63
No. 20 (0.425 mm)	45
No. 200 (0.075 mm)	25

PI (of fines) = 10
Gravel is durable
pH = 7.5
- $\gamma_r = 133 \text{ lb/ft}^3$ (21 kN/m³), $\omega_{\text{opt}} = 15 \%$
- $\phi' = 33^\circ$, $c' = 0$
- Soil is relatively inert, based on neutral pH tests for backfill and geology of area.

STEP 4. DESIGN PARAMETERS FOR REINFORCEMENT

For preliminary analysis use default values.

- Long-Term Allowable Strength for Geosynthetic Reinforcement:
$$T_{\text{al}} = T_{\text{ULT}}/\text{RF}$$
- Pullout Factor of Safety: $\text{FS}_{\text{PO}} = 1.5$

STEP 5. CHECK UNREINFORCED STABILITY

Using STABL4M, a search was made to find the minimum unreinforced safety factor and to define the critical zone. Both rotational and wedge stability evaluations were performed with Figure E8-1 showing the rotational search. The minimum unreinforced safety factor was 0.68 with the critical zone defined by the target factor of safety FS_R as shown in Figure E8-1b. Remember that the critical zone from the unreinforced analysis roughly defines the zone needing reinforcement.

STEP 6. CALCULATE T_S FOR THE FS_R

From the computer runs, obtain FS_U , M_D , and R for each failure surface within the critical zone and calculate T_S from equation 9-1 as follows. (Note: with minor code modification, this could easily be done as part of the computer analysis.)

- a. Calculate the total reinforcement tension T_S , required:

$$T_s = (1.3 - FS_U) \frac{M_D}{R}$$

Evaluating all of the surfaces in the critical zone indicates maximum total tension $T_{S-MAX} = 3400 \text{ lb/ft}$ (49.6 kN/m) for $FS_U = 0.89$ as shown in Figure 9-11c.

- a. Checking T_{S-MAX} by using the design charts in Figure 9-5:

$$\phi_r = \tan^{-1} \left(\frac{\tan \phi_r}{FS_R} \right) = \tan^{-1} \left(\frac{\tan 33^\circ}{1.3} \right) = 26.5^\circ$$

From Figure 9-5, $K \approx 0.14$

and,

$$H' = H + q/\gamma_r + 0.3 \text{ ft (for 2\% road grade)}$$

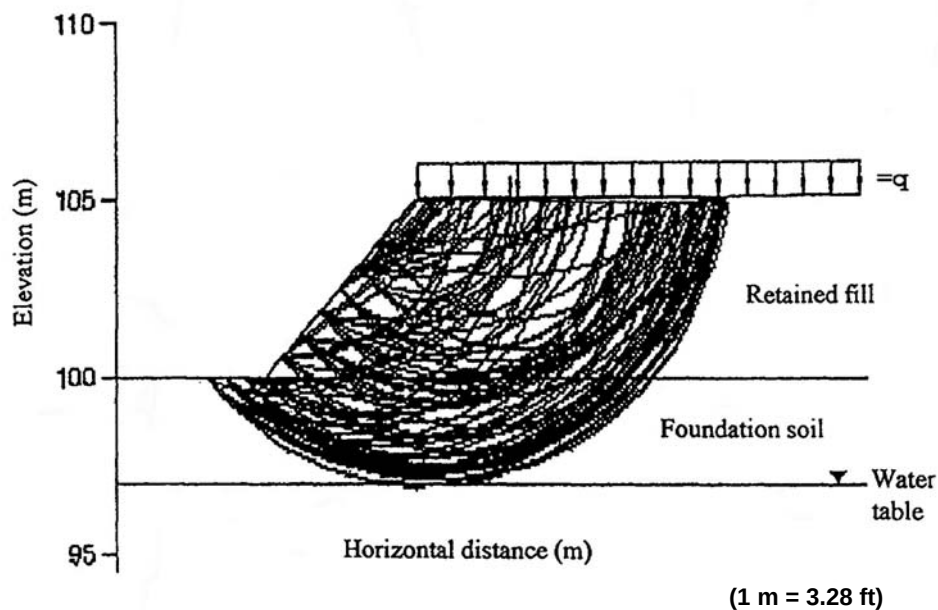
$$= 16.5 \text{ ft} + (200 \text{ psf} \div 133 \text{ lb/ft}^3) + 0.3 \text{ ft} = 18.3 \text{ ft (5.6 m)}$$

then,

$$T_{S-MAX} = 0.5 K \gamma_r H'^2 = 0.5 (0.14) (133 \text{ lb/ft}^3) (18.3 \text{ ft})^2 = 3120 \text{ lb/ft (46.5 kN/m)}$$

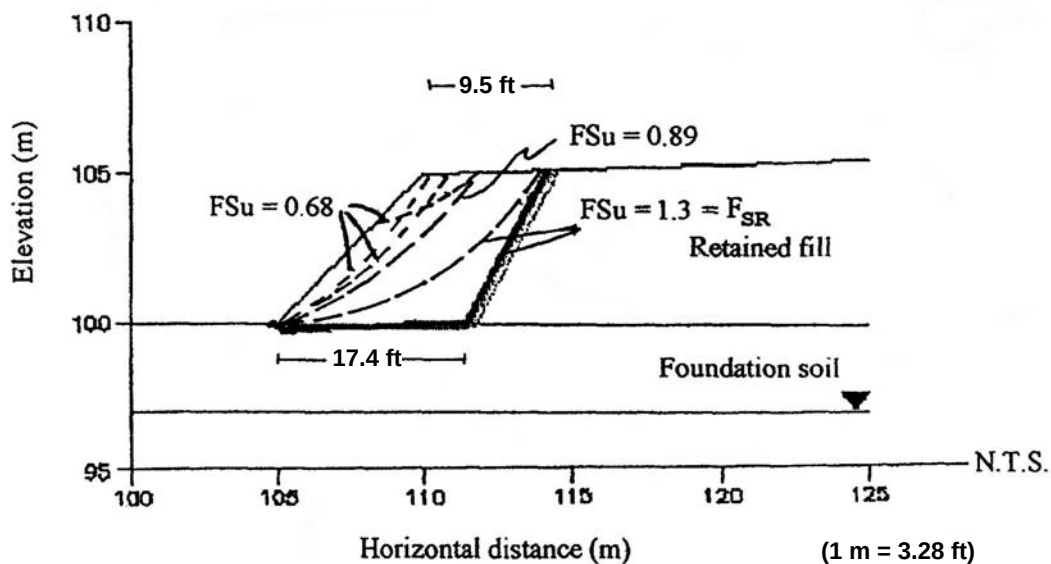
The evaluation using Figure 9-5 appears to be in reasonably good agreement with the computer analysis for this simple problem.

Bishop Circular Surfaces - Search for Critical Surfaces



A) Unreinforced stability analysis.

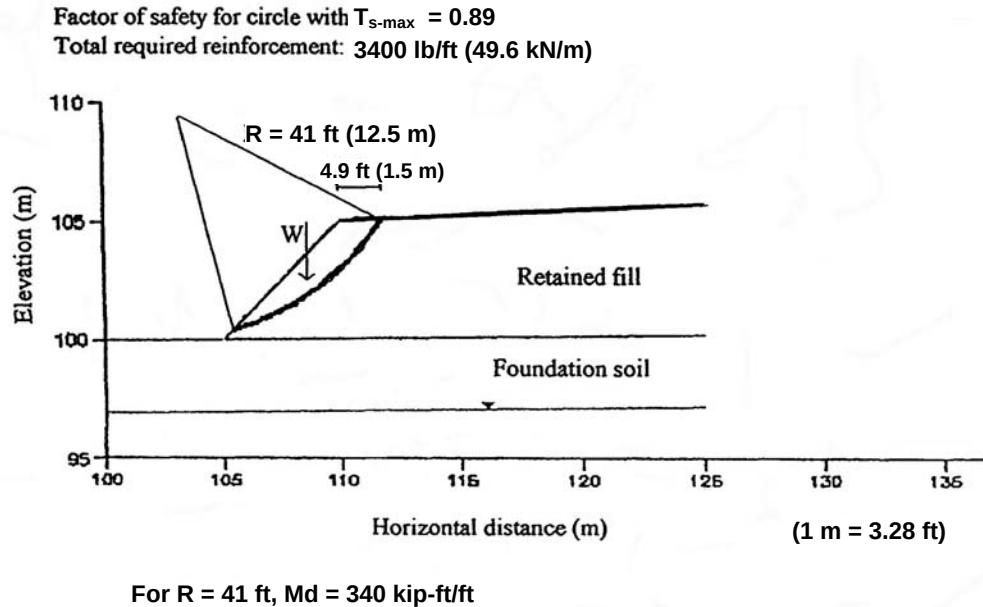
Minimum Factor of Safety



B) Results of unreinforced stability analysis.

Figure E8-1. Unreinforced stability analysis results.

Strength Design - Reinforcement for Critical Surface



C) Surface requiring maximum reinforcement (i.e. most critical reinforced surface)

Figure E8-1. Unreinforced stability analysis results (continued).

c. Determine the distribution of reinforcement.

Since $H < 20$ ft (6 m), use a uniform spacing. Due to the cohesive nature of the reinforced fill, maximum compaction lifts of 8 in. (200 mm) are recommended.

d. As discussed in the design section, to avoid wrapping the face and surficial stability issues, use $S_v = 16$ in. (400 mm) reinforcement spacing; therefore, $N = 16.5$ ft / 1.33 ft = 12.4, use 12 layers with the bottom layer placed after the first lift of embankment fill.

$$T_{\max} = \frac{T_{S-\max}}{N} = \frac{3400 \text{ lb/ft}}{12} = 283 \text{ lb/ft}$$

(Note: Other reinforcement options such as using short secondary reinforcements at every lift with spacing and strength increased for primary reinforcements may be considered and evaluated in order to select the most cost-effective final design.)

- e. Since this is a simple structure, rechecking T_s above each layer or reinforcement is not performed.
- f. For preliminary analysis of the required reinforcement lengths, the critical zone found in the computer analysis (Figure 9-11b) could be used to define the limits of the reinforcement. This is especially true for this problem since the sliding failure surface with $FS \geq 1.3$ encompasses the rotational failure surface with $FS \geq 1.3$.

From direct measurement at the bottom and top of the sliding surface in Figure E8-1b, the required lengths of reinforcement are:

$$\begin{aligned} L_{\text{bottom}} &= 17.4 \text{ ft (5.3 m)} \\ L_{\text{top}} &= 9.5 \text{ ft (2.9 m)} \end{aligned}$$

Check length of embedment beyond the critical surface L_e and factor of safety against pullout.

Since the most critical location for pullout is the reinforcement near the top of the slope (depth $Z = 8 \text{ in. } \{200 \text{ mm}\}$), subtract the distance from the critical surface to the face of the slope in Figure E8-1c (i.e., 4.9 ft) from L_{top} . This gives L_e at top = 4.6 ft (1.4 m).

Assuming the most conservative assumption for pullout factors F^* and α from Chapter 3, Section 3.4 gives $F^* = 0.67 \tan \phi$ and $\alpha = 0.6$

Therefore,

$$FS = \frac{L_e F^* \alpha \sigma_v C}{T_{\text{max}}} = \frac{4.6 \text{ ft} (0.67 \tan 33^\circ) (0.6) (0.67 \text{ ft} \times 133 \text{ lb/ft}^3 + 200 \text{ psf}) (2)}{283 \text{ lb/ft}}$$

$$FS_{\text{po}} = 2.4 > 1.5 \text{ required}$$

Check the length requirement using Figure 9-5.

For L_B , use $\phi_{\min}$ from foundation soil

$$\phi'_f = \tan^{-1} \left(\frac{\tan 28^\circ}{1.3} \right) = 22.2^\circ$$

From Figure 9-5: $L_B/H' = 0.96$

thus, $L_B = 18.3 \text{ ft} (0.96) = 17.6 \text{ ft} (5.4 \text{ m})$

For L_T , use $\phi_{\min}$ from reinforced fill

$$\phi'_f = \tan^{-1} \left(\frac{\tan 33^\circ}{1.3} \right) = 26.5^\circ$$

From Figure 9-5: $L_T/H' = 0.52$

thus, $L_T = 18.3 \text{ ft} (0.52) = 9.5 \text{ ft} (2.9 \text{ m})$

The evaluation again, using Figure 9-5, is in good agreement with the computer analysis.

- g. This is a simple structure and additional evaluation of design lengths is not required. For a preliminary analysis, and a fairly simple problem, Figure 9-5 or any number of proprietary computer programs could be used for a rapid evaluation of T_{S-MAX} and T_{max} .

In summary, 12 layers of reinforcement are required with a long-term allowable material strength T_{al} of 284 lb/ft (4.14 kN/m) and an average length of 13.4 ft (4 m) over the full height of embankment.

E8-2. COMPUTER-AIDED SOLUTION

Users of this manual will likely use a computer program(s) to work through reinforced slope design. Before using any program, users should be very familiar with the method of analysis used in the computer program. One method of checking the results produced by the software is to work through examples of problems with known solutions. Users are encouraged to use the previous two examples in evaluating and gaining familiarity with computer software. For example, Design Example E8 is contained as an input file in the program ReSSA and a step-by-step tutorial of this example is located on the software developer's web page: <http://www.geoprograms.com/>.

EXAMPLE E9

REINFORCED SOIL SLOPE DESIGN – HIGH SLOPE FOR NEW ROAD CONSTRUCTION

E9-1 INTRODUCTION

An embankment will be constructed to elevate an existing roadway that currently exists at the toe of a slope with a stable 1.6H: 1V configuration. The maximum height of the proposed embankment will be 62 ft (19 m) and the desired slope of the elevated embankment is 0.84H:1.0V. A geogrid with an ultimate tensile strength of 6,850 lb/ft (100 kN/m) based on ASTM D6637 wide width method is desired for reinforcing the new slope. A uniform surcharge of 250 lb/ft² (12 kPa) is to be used for the traffic loading condition. Available information indicated that the natural foundation soils have a drained friction angle of 34° and effective cohesion of 250 lb/ft² (12 kPa). The fill to be used in the reinforced section will have a minimum friction angle of 34°.

The reinforced slope design must have a minimum factor of safety of 1.5 for slope stability. The minimum design life of the new embankment is 75 years.

Determine the number of layers, vertical spacing, and total length required for the reinforced section.

STEP 1. GEOMETRIC AND LOADING REQUIREMENTS FOR DESIGN

a. Slope description:

- Slope height, $H = 62 \text{ ft (19 m)}$
- Reinforced slope angle, $\theta = \tan^{-1}(1.0/0.84) = 50^\circ$
- Existing slope angle, $\beta = \tan^{-1}(0.61/1.0) = 31.4^\circ$
- Surcharge load, $q = 250 \text{ psf (12.5 kN/m}^2\text{)}$

b. Performance requirements:

- External stability
 - Sliding: $FS \geq 1.5$
 - Deep seated (overall stability): $FS \geq 1.5$
 - Dynamic loading: no requirement
 - Settlement: analysis required

- Internal stability
Slope stability: $FS \geq 1.5$

STEP 2. ENGINEERING PROPERTIES OF THE NATURAL SOILS IN THE SLOPE

For this project, the foundation and existing embankment soils have the following strength parameters:

$$\phi' = 34^\circ, c' = 250 \text{ psf (12 kPa)}$$

Depth of water table, $d_w = 1.5 \text{ m}$ below base of embankment

STEP 3. PROPERTIES OF AVAILABLE FILL

The fill material to be used in the reinforced section was reported to have the following properties:

$$\gamma = 120 \text{ pcf (18.8 kN/m}^3\text{)}, \phi' = 34^\circ, c' = 0$$

STEP 4. REINFORCEMENT PERFORMANCE REQUIREMENT

Allowable tensile force per unit width of reinforcement, T_{al} , with respect to service life and durability requirements:

$$T_{al} = T_{ULT}/RF \text{ and } RF = RF_{ID} \times RF_{CR} \times RF_D$$

For the proposed geogrid to be used in the design of the project, the following factors are used:

RF_D = durability factor of safety = 1.25.

RF_{ID} = construction damage factor of safety = 1.2.

RF_{CR} = creep reduction factor = 3.0.

Note: A $FS = 1.5$ will be applied to the geogrid reinforcement in stability analysis.

Reduction factors were determined by the owner based on evaluation of project conditions and geogrid tests and field performance data submitted by the manufacturer. Therefore:

$$T_{al} = \frac{(6850 \text{ lb/ft})}{(1.25)(1.2)(3)} = 1520 \text{ lb/ft (22 kN/m)}$$

Pullout Resistance: $FS = 1.5$ for granular soils with a 3 ft (1 m) minimum length in the resisting zone.

STEP 5. CHECK UNREINFORCED STABILITY

The unreinforced slope stability was checked using the rotational slip surface method, as well as the wedge shaped failure surface method, to determine the limits of the reinforced zone and the required total reinforcement tension to obtain a factor of safety of 1.5.

The proposed new slope was first analyzed without reinforcement using a hand solution (e.g., the FHWA *Soils and Foundations Reference Manual*, {Samtani and Nowatzki, 2006}) or computer programs such as STABL4M, ReSSA, XSTABL, or RSS. The computer program calculates factors of safety (FS) using the Modified Bishop Method for circular failure surface. Failure is considered through the toe of the slope and the crest of the new slope as shown in the design example Figure E9-1a. Note that the minimum factor of safety for the unreinforced slope is less than 1.0. The failure surfaces are forced to exit beyond the crest until a factor of safety of 1.5 or more is obtained. Several failure surfaces should be evaluated using the computer program.

Next, the Janbu Method for wedge shaped failure surfaces is used to check sliding of the reinforced section for a factor of safety of 1.5, as shown on the design example Figure E9-1a. Based on the wedge shaped failure surface analysis, the limits of the critical zone to be reinforced are reduced to 46 ft (14 m) at the top and 56 ft (17 m) at the bottom for the required factor of safety.

STEP 6. CALCULATE T_S FOR $FS_R = 1.5$

- a. The total reinforcement tension T_S required to obtain a $FS_R = 1.5$ is then evaluated for each failure surface. The most critical surface is the surface requiring the maximum reinforced tension T_{S-MAX} . An evaluation of all the surfaces in the critical zone indicated $T_{S-MAX} = 66.7$ kips/ft (1000 kN/m) and is determined as:

$$T_S = (FS_R - FS_U) \frac{M_D}{D} = (1.5 - FS_U) \frac{M_D}{R}$$

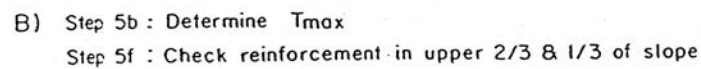
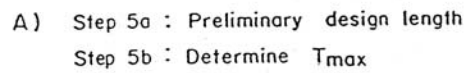
The most critical circle is where the largest $T_S = T_{S-MAX}$. As shown on the design example Figure 9-12a, T_{S-MAX} is obtained for $FS_U = 0.935$.

For this surface, $M_D = 14,827$ kips-ft/ft (67,800 kN-m/m) as determined stability analysis.

$D = R$ for geosynthetics = radius of critical circle

$R = 125.6$ ft (38.3 m)

$$T_{S-MAX} = (1.5 - 0.935) \frac{14827 \text{ k-ft/ft}}{125.6 \text{ ft}} = 66.7 \text{ kips/ft (1000 kN/m)}$$


$$T_{\text{Top}} = 1/6 T_{s-\text{max}} = 11.2 \text{ kips/ft}$$
RSS Design Examples
November 2009

b. Check using chart design procedure:

For $\theta = 50^\circ$, and

$$\phi'_f = \tan^{-1}(\tan \phi_r / FS_R) = \tan^{-1}(\tan 34^\circ / 1.5) = 24.2^\circ$$

Force coefficient, $K = 0.21$ (from Figure 9-5a) and,

$$H' = H + q/\gamma_r = 62 \text{ ft} + (250 \text{ psf})/(120 \text{ pcf}) = 64 \text{ ft}$$

then,

$$T_{S-MAX} = 0.5 K \gamma_r (H')^2 = 0.5(0.21)(120 \text{ pcf}) (64 \text{ ft})^2 \\ = 52 \text{ kips/ft (766 kN/m)}$$

Values obtained from both procedures are comparable within 25 percent. Since the chart procedure does not include the influence of water, use $T_{S-MAX} = 1000 \text{ kN/m}$.

c. Determine the distribution of reinforcement

Based on the overall embankment height divide the slope into three reinforcement zones of equal height as in equations 9-4 through 9-6.

$$T_{\text{bottom}} = \frac{1}{2} T_{S-MAX} = (\frac{1}{2})(67 \text{ k/ft}) = 33.5 \text{ kips/ft (500 kN/m)}$$

$$T_{\text{middle}} = \frac{1}{3} T_{S-MAX} = (\frac{1}{3})(67 \text{ k/ft}) = 22.3 \text{ kips/ft (330 kN/m)}$$

$$T_{\text{top}} = \frac{1}{6} T_{S-MAX} = (\frac{1}{6})(67 \text{ k/ft}) = 11.2 \text{ kips/ft (170 kN/m)}$$

d. Determine reinforcement vertical spacing S_v

Minimum number of layers,
$$N = \frac{T_{S-MAX}}{T_{\text{allowable}}} = \frac{67 \text{ k/ft}}{1.5 \text{ k/ft}} = 44.7$$

Distribute at bottom 1/3 of slope:
$$N_B = \frac{33.5 \text{ k/ft}}{1.5 \text{ k/ft}} = 22.3 \text{ use 23 layers}$$

At middle 1/3 of slope:
$$N_M = \frac{22.3 \text{ k/ft}}{1.5 \text{ k/ft}} = 15 \text{ layers}$$

At upper 1/3 of slope:
$$N_T = \frac{11.2 \text{ k/ft}}{1.5 \text{ k/ft}} = 7.5 \text{ use 8 layers}$$

Total number of layers: $46 > 44.7$ OK

Vertical spacing:

Total height of slope = 62 ft (19 m)

Height for each zone = $62 \text{ ft} / 3 = 21 \text{ ft}$ (6.3 m)

Required spacing:

At bottom 1/3 of slope:

$$S_{\text{required}} = \frac{62 \text{ ft}}{23 \text{ layers}} = 0.91 \text{ ft, use 8 in. spacing}$$

At middle 1/3 of slope:

$$S_{\text{required}} = \frac{62 \text{ ft}}{15 \text{ layers}} = 1.4 \text{ in., use 18 in spacing}$$

At top 1/3 of slope:

$$S_{\text{required}} = \frac{62 \text{ ft}}{8 \text{ layers}} = 2.6 \text{ ft, use 24 in. spacing}$$

Provide 6 ft length of secondary reinforcement layers in the upper 1/3 of the slope, between primary layers (based on primary reinforcement spacing at a 16 in. vertical spacing).

- e. The reinforcement tension required within the middle and upper 1/3 of the unreinforced slope is then calculated using the slope stability program to check that reinforcement provided is adequate as shown in the design example Figure E9-1b.

Top 2/3 of slope: $T_{S-\text{MAX}} = 31.3 \text{ k/ft} < N \cdot T_{\text{al}} = 23 \text{ layers} \times 1.5 \text{ k/ft} = 34.5 \text{ kips/ft}$ OK

Top 1/3 of slope: $T_{S-\text{MAX}} = 10 \text{ k/ft} < N \cdot T_{\text{a}} = 8 \text{ layers} \times 1.5 \text{ k/ft} = 12 \text{ kips /ft}$ OK

- f. Determine the reinforcement length required beyond the critical surface for the entire slope from Figure E9-1a, used to determine T_{max} as,

$$L_e = \frac{T_{\text{max}} FS}{F * \alpha \sigma_v C} = \frac{(1520 \text{ k / ft})(1.5)}{(0.8 \tan 34^\circ)(0.66)(120 \text{ pcf} \times Z)(2)} = \frac{26.4 \text{ ft}}{Z}$$

At depth Z , from the top of the crest, L_e is found and compared to the available length of reinforcement that extends behind the T_{DESIGN} failure surface, as determined by the sliding wedge analysis:

$$Z = 2 \text{ ft}, L_e = 13.2 \text{ ft}, \text{ available length, } L_e = 17 \text{ ft OK}$$

$$Z = 4 \text{ ft}, L_e = 6.6 \text{ ft}, \text{ available length, } L_e = 16 \text{ ft OK}$$

$$Z = 6 \text{ ft}, L_e = 4.4 \text{ ft}, \text{ available length, } L_e = 16 \text{ ft OK}$$

$$Z = 8 \text{ ft}, L_e = 3.3 \text{ ft}, \text{ available length, } L_e = 16 \text{ ft OK}$$

$$Z > 8 \text{ ft}, L_e = 3.0 \text{ ft}, \text{ available length, } L_e = > 16 \text{ ft OK}$$

Further checks of Z are unnecessary.

Checking the length using Figure 9-5b for $\phi_f = 24^\circ$

$$L_T/\dot{H} = 0.65 \rightarrow L_T = 42 \text{ ft}$$

$$L_B/\dot{H} = 0.80 \rightarrow L_B = 51 \text{ ft}$$

Results from both procedures check well against the wedge failure analysis in step 5a. Realizing the chart solution does not account for the water table use top length $L_T = 46$ ft (14 m) and bottom length $L_B = 56$ ft (17 m) as determined by the computer analyses in step 5a.

- g. The available reinforcement strength and length were checked using the slope stability program for failure surfaces extending beyond the $T_{S-\text{MAX}}$ failure surface and found to be greater than required.

STEP 7. CHECK EXTERNAL STABILITY

- a. Sliding Stability.

The external stability was checked using the computer program for wedge shaped failure surfaces. The FS obtained for the failure surface outside the reinforced section, defined with a 46 ft (14 m) length at the top and a 56 ft (17 m) length at the bottom, was 1.5.

b. Deep Seated Global Stability.

The overall deep-seated failure analysis indicated that a factor of safety of 1.3 exists for failure surfaces extending outside the reinforced section (as shown in the design example Figure E9-1b). This is due to the grade at the toe of the slope that slopes down into the lake. The factor of safety for deep-seated failure does not meet requirements. Therefore, the reinforcement would have to be extended to a greater length, the toe of the new slope should be regraded, or the slope would have to be constructed at a flatter angle.

For the option of extending the reinforcement length, local bearing must be checked. Local bearing (lateral squeeze) failure does not appear to be a problem as the foundation soils are granular and will increase in shear strength due to confinement. Also, the foundation soil profile is consistent across the embankment such that global bearing and local bearing will essentially result in the same factor of safety. For these conditions, the lower level reinforcements could simply be extended back to an external stability surface that would provide $FS = 1.5$ as shown in Figure E9-2.

If the foundation soils were cohesive and limited to a depth of less than 2 times the base width of the slope, then local stability should be evaluated. As an example, assume that the foundation soils had an undrained shear strength of 2080 psf (100 kPa) and extended to a depth of 33 ft (10 m), at which point the granular soils were encountered. Then, in accordance with equation 9-15,

$$FS_{\text{squeezing}} = \frac{2c_u}{\gamma D_s \tan\theta} + \frac{4.14c_u}{H\gamma} =$$
$$FS_{\text{squeezing}} = \frac{2(1040 \text{ psf})}{120 \text{ pcf} (33 \text{ ft})(\tan 50^\circ)} + \frac{4.14(1040 \text{ psf})}{62 \text{ ft}(120 \text{ pcf})} = 1.02$$

Since $FS_{\text{squeezing}}$ is lower than the required 1.3, extending the length of the reinforcement would not be an option without improving the stability conditions. This could be accomplished by either reducing the slope angle or by placing a surcharge at the toe, which effectively reduces the slope angle.

c. Foundation Settlement.

Due to the granular nature of the foundation soils, long-term settlement is not of concern.

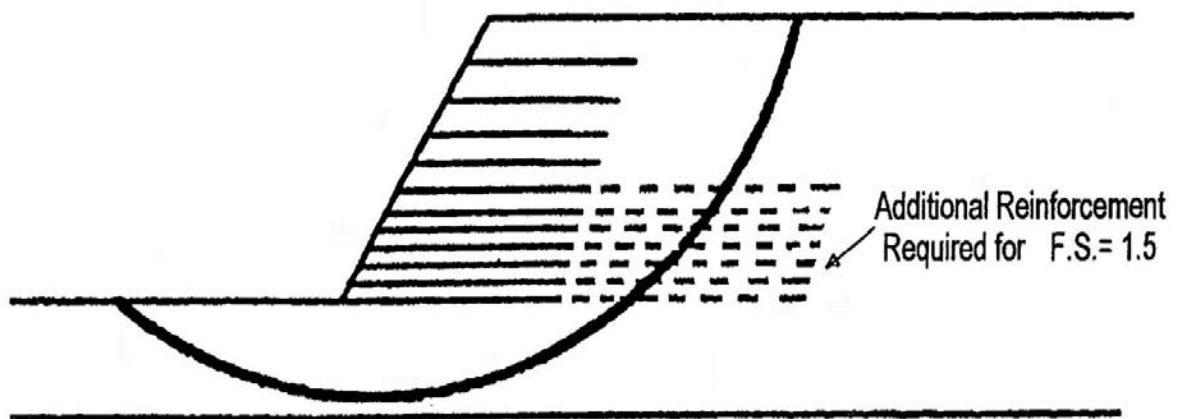


Figure E9-2. Design Example 2: global stability.

EXAMPLE E10

REINFORCED SOIL SLOPE DESIGN – FACING STABILITY CALCULATION

E10-1 INTRODUCTION

Economies can sometimes be achieved by using higher strength primary reinforcement at wider spacing combined with short intermediate reinforcement layers to meet maximum spacing requirements, provide compaction aids and face stability. The calculations for face stability evaluation of slopes using intermediate reinforcement will be demonstrated for the slope in Example E8, with modified primary reinforcement. The guidelines for intermediate reinforcement presented under Step 6 of section 9.2 Reinforced Slope Design Guidelines will be followed.

To evaluate cost alternatives in Example E8, modify primary reinforcement by doubling strength to 570 lb/ft (8.3 kN/m) and doubling vertical spacing. Intermediate reinforcement will be placed at 24 in. (800 mm) vertical spacing, centered between the primary reinforcement (at 24 in. {800 mm} spacing). The length of intermediate reinforcement will be set at 4 ft (1.2 m) and minimum long term tensile strength, T_{ab} , of 380 lb/ft (5.5 kN/m) will be used to meet constructability requirements.

Surficial failure planes may extend to a depth of about 3 to 6% of the slope height. Therefore, the stability safety factor will be checked for depths up to 6% of slope height, for dry conditions. Also, checks will be performed at various depths assuming saturation to that depth, to see if project conditions (e.g., local rainfall) need to be further evaluated.

E10-2. CHECK STABILITY SAFETY FACTOR FOR VARIOUS DEPTHS TO POTENTIAL FAILURE PLANE.

Compute depth equal to 6% of slope height.

$$(0.06) 16.5 \text{ ft} = 1 \text{ ft}$$

Check stability at 6 in., 12 in. and 24 in. depths to potential failure plane. Use Equation 9-8 with the following parameters.

$$F.S. = \frac{c'H + (\gamma_g - \gamma_w)H z \cos^2 \beta \tan \phi' + F_g (\cos \beta \sin \beta + \sin^2 \beta \tan \phi')}{\gamma_g H z \cos \beta \sin \beta}$$

where

c' = effective cohesion — assume equal to zero, which conservatively neglects vegetative reinforcement. See Gray and Sotir (1995), for guidance of estimating strength of vegetation, if desired to include in analysis.

ϕ' = effective friction angle — 33°

γ = unit weight of fill soil — 134 lb/ft^3

z = vertical depth to failure plane — $\frac{1}{2} \text{ ft}$, 1 ft , 2 ft

H = vertical slope height — 16.5 ft

β = slope angle — 45°

F_g = summation of geosynthetic resisting force — varies by z , as strength at shallow embedment will likely be controlled by pullout resistance, therefore, compute by failure plane depth

Geosynthetic available reinforcement strength is based on pullout toward the front face of the slope (i.e., the geosynthetic resistance to the outward movement of the wedge of soil above and below the geosynthetic).

Primary reinforcement –

$$T_a (= T_{al}) = 570 \text{ lb/ft}$$

Strength limited by pullout resistance near the face, with $FS_{PO} = 1.0$, equals

$$T = F^* \alpha \sigma_v C L_e$$

Where, F^* and α are as assumed for the geogrid in Example 1, and

σ_v = the weight of the triangular wedge of soil over the geosynthetic = $\frac{1}{2}\gamma z$

$$L_e = z / \tan 45^\circ = z$$

$$C = 2$$

$$T = (0.8 \tan 33^\circ) (0.66) [\frac{1}{2} (134 \text{ lb/ft}^3) (z)] (2) (z)$$

$$T = 46 z^2 \text{ lb/ft}$$

Therefore,

$$@ 0.5 \text{ ft}, T = 11 \text{ lb/ft}$$

$$@ 1.0 \text{ ft}, T = 46 \text{ lb/ft}$$

$$@ 2.0 \text{ ft}, T = 184 \text{ lb/ft}$$

Intermediate reinforcement –

$$T_a (= T_{al}) = 380 \text{ lb/ft}$$

Strength limited by pullout resistance, with $FS_{PO} = 1.0$, equals

$$T = F^* \alpha \sigma_v C L_e$$

assuming F^* and α parameters equal to those of the primary reinforcement again leads to,

$$@ 0.5 \text{ ft, } T = 11 \text{ lb/ft}$$

$$@ 1.0 \text{ ft, } T = 46 \text{ lb/ft}$$

$$@ 2.0 \text{ ft, } T = 184 \text{ lb/ft}$$

The slope contains 6 layers of primary reinforcement and 6 layers of intermediate reinforcement. Therefore,

$$@ 0.5 \text{ ft} - F_g = 6 (11 \text{ lb/ft}) + 6 (11 \text{ lb/ft}) = 132 \text{ lb/ft}$$

$$@ 1.0 \text{ ft} - F_g = 6 (46 \text{ lb/ft}) + 6 (46 \text{ lb/ft}) = 552 \text{ lb/ft}$$

$$@ 2.0 \text{ ft} - F_g = 6 (184 \text{ lb/ft}) + 6 (184 \text{ lb/ft}) = 2208 \text{ lb/ft}$$

With $c = 0$ and dry conditions, equation (9-8) reduces to

$$F.S. = \frac{\gamma H z \cos^2 \beta \tan \phi' + F_g (\cos \beta \sin \beta + \sin^2 \beta \tan \phi')}{\gamma H z \cos \beta \sin \beta}$$

$$F.S. = \frac{(134 \text{ lb/ft}^3)(16.5 \text{ ft})(\cos^2 45^\circ)(\tan 33^\circ)z + F_g [\cos 45^\circ (\sin 45^\circ) + \sin^2 45^\circ (\tan 33^\circ)]}{(134 \text{ lb/ft}^3)(16.5 \text{ ft})(\cos 45^\circ)(\sin 45^\circ)z}$$

$$F.S. = \frac{717 \text{ lb/ft}^2 z + F_g (0.82)}{1105 \text{ lb/ft}^2 z}$$

Therefore,

$$@ 0.5 \text{ ft, } FS = 0.84$$

$$@ 1.0 \text{ ft, } FS = 1.1$$

$$@ 2.0 \text{ ft, } FS = 1.5$$

Thus, assuming cohesion equal to zero, it is computed that the slope face is unstable at shallow depths (0.5 ft to 1 ft). A small amount of cohesion may be provided by the soil fill and/or vegetation. Assume a nominal amount of cohesion (e.g., 42 psf {2 kPa}), and recompute factors of safety.

$$cH = 42 \text{ psf} (16.4\text{ft}) = 688 \text{ lb/ft}$$

Then, the factor of safety is equal to

$$F.S. = \frac{688 \text{ lb/ft}^2 + 717 \text{ lb/ft}^2 z + F_g (0.82)}{1105 \text{ lb/ft}^2 z}$$

and

$$@ 0.5 \text{ ft, } FS = 2.1$$

$$@ 1.0 \text{ ft, } FS = 1.7$$

$$@ 2.0 \text{ ft, } FS = 1.8$$

Thus, with only a small amount of cohesion the slope face would be stable.

E10-3. CHECK THE SAFETY FACTOR FOR VARIOUS DEPTHS TO POTENTIAL FAILURE PLANE ASSUMING SATURATION TO THAT DEPTH, TO SEE IF REASONABLE FOR PROJECT CONDITIONS.

With parameters of $\gamma_g - \gamma_w = 72 \text{ lb/ft}^3$ and cohesion of 42 psf (implies cohesion is derived from vegetation, and is retained under saturated conditions)

Then, the factor of safety is equal to

$$F.S. = \frac{688 \text{ lb/ft}^2 + 385 \text{ lb/ft}^2 z + F_g (0.82)}{1105 \text{ lb/ft}^2 z}$$

and

$$@ 0.5 \text{ ft, } FS = 1.8$$

$$@ 1.0 \text{ ft, } FS = 1.4$$

$$@ 2.0 \text{ ft, } FS = 1.5$$

Again, the slope is stable provided vegetation is established on the slope face. A geosynthetic erosion mat would also help maintain the face stability.

APPENDIX F

OTHER DESIGN PROCEDURES AND ANALYSIS MODELS

- F.1 Simplified Method and ASD Platform
- F.2 Coherent Gravity Method
- F.3 National Concrete Masonry Association Procedure
- F.4 GRS
- F.5 FHWA Structure Stiffness Method
- F.6 K-Stiffness Method
- F.7 Deep Patch

There are several means, other than the *Simplified Method* analysis model with the LRFD platform, which can be used for the design of MSE walls. Several of these are summarized below. Note that some design methods can be used in either a LRFD or an ASD platform. The use of the Simplified Method analysis model with the LRFD platform is recommended for the design of transportation MSE wall structures.

F.1 Simplified Method and ASD Platform

As previously (Section 4.1.1) noted, Engineers have been designing MSE highway walls using an ASD (allowable stress design) procedure since MSE walls were introduced in the early 1970's. All uncertainty in applied loads and material resistance are combined in a single factor of safety or allowable material stress. The advantages of progressing to a LRFD procedure were summarized in Section 4.4.1.

Future MSE walls will be designed with the LRFD procedure. Therefore, current guidance, i.e., AASHTO Standard Specifications for Highway Bridges, 17th Edition (2002) and FHWA NHI-00-043 (Elias et al., 2001), on MSE wall design using ASD procedures will not be updated. Note that the AASHTO (2002) and FHWA (2001) ASD references will not be updated by AASHTO or FHWA, respectively. Any designers engineering MSE walls with the ASD procedures in the future may want to refer to current LRFD procedures for any updates which may also be applicable to ASD procedure designs (e.g., seismic loading for external stability analysis).

The *Simplified Method* of analysis has been used with ASD procedures since 1996. This method was developed using FHWA research (Christopher et al., 1990) and existing design methods (i.e., coherent gravity method, tie-back wedge method) as a starting point to create a single method for agencies and vendors to use (Elias and Christopher, 1997; AASHTO, 1997; Allen et al., 2001). The simplified method uses a variable state of stress for internal stability analysis. This variable state of stress is defined in terms of a multiple of the active lateral earth pressure coefficient, K_a , and is a function of the type of reinforcement used and depth from the top of wall. This single method of design is applicable to all types of soil reinforcement. Thus, the simplified method offers the following advantages over other methods:

- Straight forward by avoiding iterative processes to determine the reinforcement requirements (i.e., it is simple and easy to use).
- Justified empirically in comparison to other methods available at the time of its development based on instrumented full scale structures, and the simplifications do not appear to compromise the Simplified Method's accuracy (Allen et al., 2001).

- Found to be more accurate for upper reinforcements in sloped surcharges (Allen et al., 2001).
- Eliminates variations in method to determine internal lateral stress.
- Eliminates variations in assumptions of the critical failure surface.
- Accounts for the differences in reinforcement type and is easily adapted to new MSE wall reinforcement types as they become available.

The simplified method has been adapted to LRFD procedures in AASHTO (2007) and in this manual. As noted in the introduction to this section, today, the use of the Simplified Gravity analysis model with the LRFD platform is recommended for the design of transportation MSE wall structures.

F.2 Coherent Gravity Method

The coherent gravity method, or analysis model, has been used for several decades in the ASD procedure. It can also be used with the LRFD procedure. The 2009 AASHTO Interims note that the maximum reinforcement loads shall be calculated using the Simplified Method or the Coherent Gravity Method. For state and agencies using the Coherent Gravity Method, the load in the reinforcements shall be obtained in the same way as the Simplified Method, except: (i) for steel reinforced wall systems, the lateral earth pressure coefficient used shall be equal to k_o at the point of intersection of the theoretical failure surface with the ground surface at or above the wall top, transitioning to a k_a at a depth of 20.0 ft below the intersection point, and constant at k_a at depths greater than 20.0 ft. and (ii) If used for geosynthetic reinforced systems, k_a shall be used throughout the wall height.

AASHTO also states that other widely accepted and published design methods for calculation of reinforcement loads may be used at the discretion of the wall owner or approving agency, provided the designer develops method-specific resistance factors for the method employed. AASHTO recommends that the resistance factors recommended for the Simplified Method should also be used for the Coherent Gravity Method.

The primary differences between the *coherent gravity method* and the *simplified method* are: (i) the coherent gravity method includes the pressure at each reinforcement elevation in the vertical pressure sum; (ii) the effect of the overturning moment caused by the retained backfill lateral earth pressure is included in the vertical pressure at each reinforcement elevation; and (iii) the lateral pressure varies from K_o at the top of the soil to K_a at a depth of 20 ft (6 m) below, and constant at K_a below the 20 ft (6 m) depth for metallic reinforcements. This is illustrated in Figure F-1. As discussed in Chapter 4 (and illustrated in Figure 4-9), the

metallic reinforcement lateral pressure varies from $1.7 K_a$ (strips) or $2.5 K_a$ (mats and grids) at top of soil to K_a at a depth of 20 ft (6 m), and constant at K_a thereafter. For comparison purposes, for a $\phi = 34^\circ$ soil, $K_o = 1 - \sin 34^\circ = 0.50$; for a $\phi = 34^\circ$ soil, $K_a = 0.283$; thus, $K_o = 1.77 K_a$.

For geosynthetic reinforcements, the lateral pressure is constant at K_a for both the coherent gravity method and the simplified method. Therefore, the *coherent gravity method* is typically used only for metallic reinforcements.

Previous research (FHWA RD-89-043) focused on defining the state of stress for internal stability, as a function of K_a , type of reinforcement used (geotextile, geogrid, metal strip or metal grid), and depth from the surface. The results from these efforts were synthesized in a *simplified method*, which can be used for all types of soil reinforcements. The simplified coherent gravity method is a single, logical method that can be used with LRFD or ASD procedure. As previously indicated in the Simplified Method section, there are a number of advantages to agencies. The method has been used for the past 12 years to safely design retaining walls. In comparison studies with field measured data, Allen et al. (2001) found the following:

- Overall, the Simplified Method and the FHWA Structure Stiffness Method produce a prediction that is slightly conservative, whereas the Coherent Gravity Method produces a prediction that is slightly nonconservative.
- The Coherent Gravity Method has been found to consistently provide lower predicted loads in structures with stiff reinforcement systems and in the upper reinforcements of sloped surcharges than measured field loads, while the Simplified Method more accurately predicts these reinforcement loads (Allen et al., 1993).
- The assumption that the reinforcement stress is reduced with increased reinforcement length is questionable and not supported by field measurements.

FHWA supports the use of a single method in order to maintain consistency in design. There are always concerns that designers will be confused and combine aspects of alternate methods that could produce nonconservative results. Agencies should be cognizant of the pending change to the AASHTO LRFD code and evaluate whether or not to allow the use of the Coherent Gravity Method (and/or other methods) in addition to the Simplified Method. Agencies specifications should be updated to reflect use of just the Simplified Method or the acceptance of either method.

Again, the use of the Simplified Gravity analysis model with the LRFD platform is recommended for the design of transportation MSE wall structures.

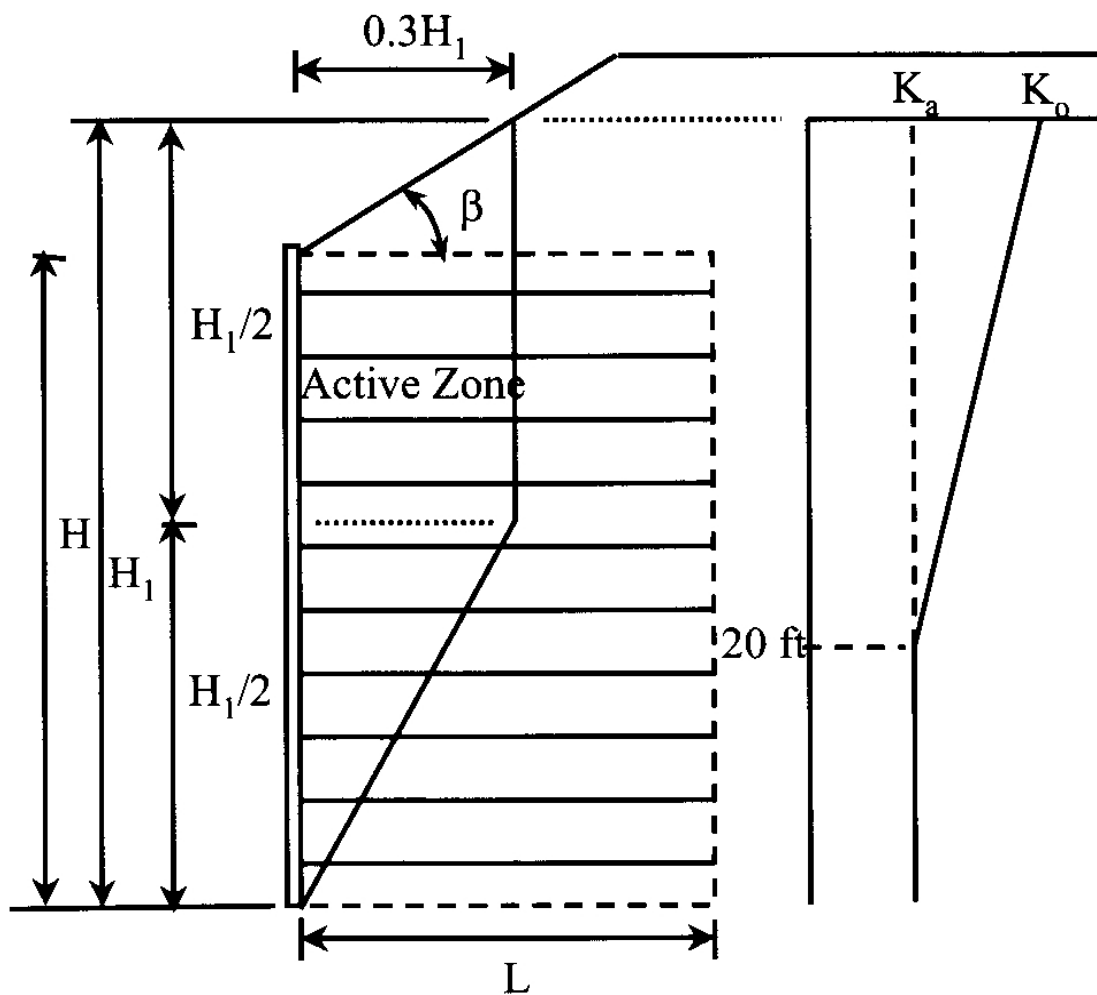


Figure F-1. Coherent gravity method lateral pressure coefficient for internal stability (2009 Interims to AASHTO, 2007).

F.3 National Concrete Masonry Association Procedure

The National Concrete Masonry Association (NCMA) method and analysis model was developed in 1993 (Simac et al.) specifically for, and is widely used with, modular block faced (a.k.a. segmental blocks), geosynthetic reinforced soil walls. It is an ASD procedure. It was updated in 1997 (NCMA), and a third update is reportedly in-progress.

The principal differences between the NCMA method and the ASD Simplified Method are: (i) internal stability lateral pressure is set equal to the Coulomb active earth pressure coefficient, instead of the Rankine coefficient; (ii) assumed failure plane is the Coulomb active pressure wedge, instead of the Rankine active pressure wedge; (iii) the minimum reinforcement length to height ratio is 0.6, versus 0.7; and (iv) the connection strength requirements are based upon short-term testing, instead of being based upon long-term testing, as required by AASHTO.

F.4 GRS

The Geosynthetic Reinforced Soil (GRS) analysis model is used with ASD procedure. This method was developed in Colorado specifically for geosynthetic soil reinforcements and wrap-around or block facings. The GRS design method is documented in NCHRP Report 556 (Wu et al., 2006). The GRS design method is a modification of the FHWA ASD Simplified Method (Elias et al., 2001). The soil reinforcement model is based upon closely-spaced vertically adjacent layers of reinforcement and soil arching, versus the FHWA method that this based upon a tied-back wedge model.

Additional principal differences between the GRS method and the ASD Simplified Method are: (i) the default vertical reinforcement spacing is 8 in. (0.2 m), and maximum spacing (for abutments) is 16 in. (0.4 m); (ii) the reinforcement length may be truncated in the bottom portion of the wall where the foundation is competent; (iii) the soil reinforcement is specified on a basis of minimum ultimate tensile strength and a minimum tensile stiffness; and (iv) connection strength is not a design requirement where the maximum reinforcement vertical spacing is 8 in. (0.2 m) and reinforced fill is a compacted select fill.

F.5 FHWA Structure Stiffness Method (from Allen et al., 2001; Christopher et al., 1990)

The Structure Stiffness Method was developed as the result of a major FHWA research project in which a number of full-scale MSE walls were constructed and monitored. Combined with an extensive review of previous fully instrumented wall case histories (Christopher et al., 1990; Christopher, 1993), small-scale and full-scale model walls were constructed and analytical modeling was conducted (Adib, 1988). This method is similar to the Tieback Wedge Method, but the lateral earth pressure coefficient is determined as a function of depth below the wall top, reinforcement type, and global wall stiffness, rather than using K_a directly. Furthermore, the location of the failure surface is the same as is used for the Coherent Gravity Method (Figure 3) for MSE walls with inextensible soil reinforcement. It is a Rankine failure surface for MSE walls with extensible soil reinforcement. The design methodology is summarized in equations 8, 9, and 10. Note that because the reinforcement stress, and the strength required to handle that stress, varies with the global wall stiffness, some iteration may be necessary to match the reinforcement to the calculated stresses.

$$T_{max} = S_v R_c K_r (\gamma Z + S + q)$$

$$K_r = K_a (\Omega_1 (1 + 0.4 (S_r / 47880)) (1 - Z/6) + \Omega_2 Z/6) \text{ if } Z \leq 6 \text{ m}$$

$$K_r = K_a \Omega_2 \quad \text{if } Z > 6 \text{ m}$$

$$S_r = EA / (H/n)$$

Where, K_r is the lateral earth pressure coefficient,

S_r is the global reinforcement stiffness for the wall (i.e., the average reinforcement stiffness over the wall face area),

Ω_1 is a dimensionless coefficient equal to 1.0 for strip and sheet reinforcements or equal to 1.5 for grids and welded wire mats,

Ω_2 is a dimensionless coefficient equal to 1.0 if S_r is less than or equal to 47880 kPa or equal to Ω_1 if S_r is greater than 47880 kPa, EA is the reinforcement modulus times the reinforcement area in units of force per unit width of wall,

H/n is the average vertical spacing of the reinforcement, and n is the total number of reinforcement layers.

This stiffness approach was based on numerous full-scale observations that indicated that a strong relationship between reinforcement stiffness and reinforcement stress levels existed, and it was theoretically verified through model tests and numerical modeling.

F.6 K-Stiffness Method

The K-Stiffness Method, or analysis model, is relatively new method that may be used with the ASD or LRFD procedure. This method was researched and developed by Allen and Bathurst (2003), Allen et al. (2003), Allen et al. (2004) and was calibrated against measurements of loads and strains from a large database of full-scale geosynthetic and full-scale steel reinforced soil walls. The method is targeted to accurately predict working loads in the soil reinforcement, though wall behavior near failure of some of the walls by excessive deformation or rupture was considered in the development of the design model (see Allen et al., 2003) to insure that such behavior would be precluded if the K-Stiffness Method is properly used and design parameters properly selected. From that research, the K-Stiffness method defined a design limit state that has not been considered in the other design models – a soil failure limit state. This is especially important for geosynthetic walls, since the geosynthetic reinforcement continues to strain and gain tensile load long after the soil has reached its peak strength and begun dropping to a residual value. Therefore, if the strain in the soil is limited to prevent it from going past peak to a residual value, failure by excessive deformation or rupture is prevented and equilibrium is maintained. This is a key design philosophy in the K-Stiffness design model.

An analysis of the K-Stiffness predictions relative to the full scale measurements indicate that the K-Stiffness method is a more accurate method for estimating loads in the soil reinforcements than other currently available design models and thereby has the potential to reduce reinforcement requirements and improve the economy of MSE walls (Allen et al., 2003 and 2004). The improvement (i.e., economy) is significant for both geosynthetic and steel reinforcement, though more pronounced for geosynthetic reinforcements. A couple of geosynthetic reinforced walls have been designed using the K-Stiffness Method, built, and fully instrumented by the Washington State Department of Transportation (WSDOT). Because they were designed with the K-Stiffness Method, the amount of soil reinforcement in the wall was reduced by a third to one-half of the reinforcement required by the AASHTO Simplified Method. Results reported by Allen and Bathurst (2006) for the largest wall (36 ft high, 600 ft long) indicate that the K-Stiffness method accurately predicted the strains in the reinforcement, and the wall has performed well since its construction. The other wall has also performed well, and full results for both walls will be available in a forthcoming WSDOT research report. The K-Stiffness Method's ability to accurately predict reinforcement strains provides promise for having the ability to accurately predict wall deformations for the serviceability limit state. See Allen and Bathurst (2003) for additional details on this issue.

The K-Stiffness method has been adapted to LRFD design procedure by the Washington DOT, and load and resistance factors to use with this method are detailed in the WSDOT Geotechnical Design Manual (2006), as well as step-by-step procedures for use of the K-Stiffness Method for design of MSE walls. The method begins with a prediction of the total lateral load to be resisted by the soil reinforcement which is consistent with the approach used by the Simplified Method. The K-Stiffness Method then takes that total lateral load and adjusts it empirically based on the effects of global reinforcement stiffness, local reinforcement stiffness, facing stiffness/toe restraint, facing batter, soil shear strength, and distribution of the total lateral force to the individual layers based on observations from many full scale wall case histories. The formulation of global reinforcement stiffness is consistent with that used in the FHWA Structure Stiffness Method (Christopher et al., 1990; Christopher, 1993). The soil shear strength (the plane strain shear strength is used for this method) is used as an index to correlate to the stiffness of the soil backfill, which is the real property of interest with regard to prediction of soil reinforcement loads at working stress conditions. Note that the methods used in historical practice (e.g., the Simplified Method) calculate the vertical stress resulting from gravity forces within the reinforced backfill at each level, resulting in a linearly increasing gravity force with depth and a lateral stress distribution that continuously increases with depth below the wall top. The K-Stiffness Method instead calculates the maximum gravity force resulting from the gravity forces within the reinforced soil backfill to determine the maximum reinforcement load within the entire wall reinforced backfill, $T_{\max}$, and then adjusts that maximum reinforcement load with depth for each of the layers using a load distribution factor, $D_{\max}$ to determine $T_{\max}$.

The method is summarized as follows:

$$\sigma_v = \gamma_p \gamma_r H + \gamma_p \gamma_f S + \gamma_{LL} q + \gamma_p \Delta \sigma_v, \text{ and}$$

$$T_{\max} = 0.5 S_v K \sigma_v D_{\max} \Phi_g \Phi_{\text{local}} \Phi_{fb} \Phi_{fs} + \gamma_p \Delta \sigma_H S_v$$

where,

σ_v = the factored pressure due to resultant of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present (KSF)

γ_p = the load factor for vertical earth pressure EV

γ_{LL} = the load factor for live load surcharge per the AASHTO LRFD Specifications

q = live load surcharge (KSF)

H = the total vertical wall height at the wall face (FT)

S = average soil surcharge depth above wall top (FT)

$\Delta\sigma_v$ = vertical stress increase due to concentrated surcharge load above the wall (KSF)

S_v = tributary area (assumed equivalent to the average vertical spacing of the reinforcement at each layer location when analyses are carried out per unit length of wall), in FT

K = is an index lateral earth pressure coefficient for the reinforced backfill, and shall be set equal to K_0 as calculated per Article 3.11.5.2 of the AASHTO LRFD Specifications. K shall be no less than 0.3 for steel reinforced systems.

$D_{\max}$ = distribution factor to estimate $T_{\max}$ for each layer as a function of its depth below the wall top relative to $T_{\max\max}$ (the maximum value of $T_{\max}$ within the wall)

S_{global} = global reinforcement stiffness (KSF)

Φ_g = global stiffness factor

Φ_{local} = local stiffness factor

Φ_{fb} = facing batter factor

Φ_{fs} = facing stiffness factor

$\Delta\sigma_H$ = horizontal stress increase at reinforcement level resulting from a concentrated horizontal surcharge load per Article 11.10.10.1 of the AASHTO LRFD Specifications (KSF)

The WSDOT GDM (2006, or most current update) should be consulted for the details on the calculation of $T_{\max}$ for each layer and how to apply this methodology to MSE wall design.

It should be noted that the K-Stiffness Method has been updated to consider a number of additional wall case histories, and additionally to consider the effect of backfill soil cohesion. See Bathurst et al. (2008) for details. While consideration of soil cohesion does help to improve the K-Stiffness Method prediction accuracy for wall backfill soil that contain a significant cohesive component to its soil shear strength, in general, it is not recommended to consider soil cohesion in the soil backfill for design purposes due to unknown long-term effect of moisture infiltration in the backfill and possibly soil creep.

F.7 Deep Patch

The deep patch is a mitigation technique for sliding roadway sections. It is typically used on roads that suffer from chronic slide movements that are primarily the result of side cast fill construction. One of the main advantages of the deep patch technique is that it is constructed with equipment that works from the roadway and does not require accessing the toe of the failed area. This technique is generally not expected to completely arrest movement seen in the road but rather slow it down to manageable levels.

Deep patch repairs consist of reinforcing the top of a failing embankment with several layers of soil reinforcement. This work is typically done with a small construction crew consisting of a laborer, hydraulic excavator, and a dump truck. The design is based on determining the extent of the roadway failure based on visual observations of cracking and then and then using analytical or empirical methods for determining the reinforcement requirements. An empirical design procedure is presented in Highway Deep Patch Road Embankment Repair Application Guide which was produced by the U.S. Department of Agriculture (USDA) Forest Service in partnership with FHWA Federal Lands Highway Division. An analytical approach is summarized as follows:

1. Characterize the existing soil properties, new fill properties if applicable, and establish the desired slope stability factor of safety after the deep patch mitigation technique is implemented.
2. Generate a cross section of the failed embankment at a location that represents the most severe movement.
3. Locate the cracks furthest from the edge of the embankment slope break (hinge point) on the cross section. Similar to the concept of MSE wall internal active and restive mechanisms the active portion of the embankment movement will be considered to be taking place on the outside of the embankment crack limits and the resisting portion on the inside of the crack limits.
4. Determine the distance from the crack limits to the embankment slope hinge.
5. Determining the total reinforcement tension required per unit width as described in Chapter 9 or using reinforced soil slope software.
6. Based on the site limitations and geometry determine the reinforcement spacing and corresponding number of reinforcement layers (Typically 2-5 layers). Divide T_s by the

number of layers to obtain the required reinforcement tensile strength per layer and per unit width ($T_{req'd}$).

7. Determine the minimum required pullout length (L_e) by using a factor of safety of 1.5 and setting $T_{req'd} = T_{max}$. Determine the minimum reinforcement length by adding L_e to the distance from the crack to the slope face for each layer.
8. Select a reinforcement in which the long-term allowable strength per unit width (Chapter 3) is greater than $T_{req'd}$.