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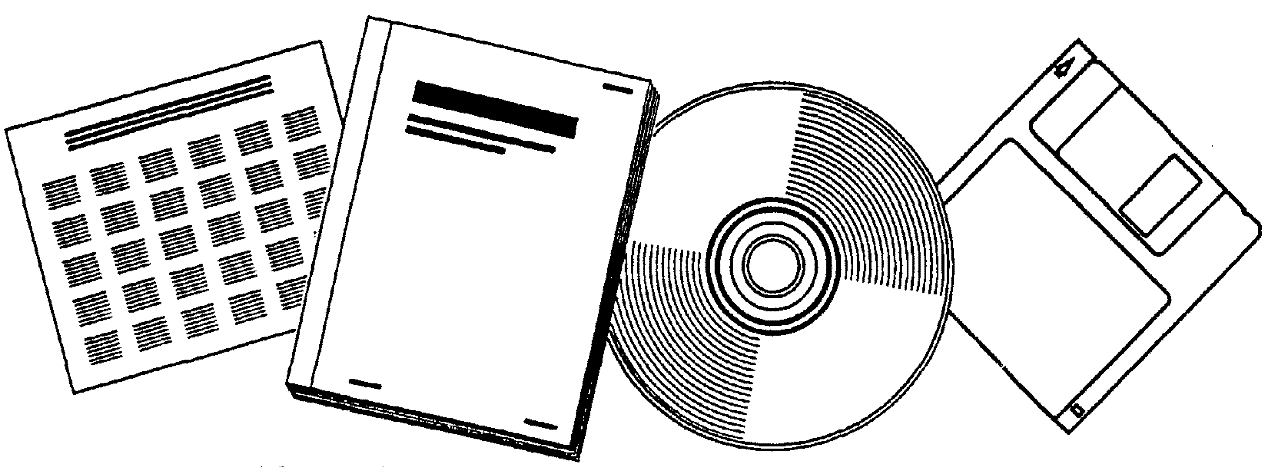
MECHANICALLY STABILIZED EARTH WALLS AND REINFORCED SOIL SLOPES DESIGN AND CONSTRUCTION GUIDELINES. PARTICIPANTS MANUAL. FHWA DEMONSTRATION PROJECT 82 GROUND IMPROVEMENT

EARTH ENGINEERING AND SCIENCES, INC., BALTIMORE, MD

OCT 96



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MECHANICALLY STABILIZED EARTH WALLS AND REINFORCED SOIL SLOPES DESIGN AND CONSTRUCTION GUIDELINES

PARTICIPANTS MANUAL

FHWA DEMONSTRATION PROJECT 82
GROUND IMPROVEMENT

OCTOBER 1996

Office of Engineering and
Office of Technology Applications
Federal Highway Administration
400 Seventh Street, SW
Washington, D.C. 20590

NOTICE

Conversion Factors Continued

Quantity	From English Units	To SI Units	Multiply By[7]
Moment of Mass	lb·ft	kg·m	0.1383
Moment of Inertia	$lb\cdot ft^2$	$kg.m^2$	0.042 14
Second Moment of Area	in^4	mm^4	416 200
Section Modulus	in^3	mm^3	16 390
Work	lb·ft	N·m	1.355 818
Energy	ft·lb	J	1.355 818
Power	ton (refrig) Btu/s hp (electric) Btu/h	kW kW W W	3.517 1.054 745.7 0.2931
Volume Rate of Flow	ft^3 cfm cfm mgd	m^3/s m^3/s L/s m^3/s	0.028 32 0.000 472 0.4719 0.0438
Temperature	°F	°C	(°F-32°)/1.8
Velocity, Speed	ft/s	m/s	<u>0.3048</u>
Acceleration	ft/s^2	m/s^2	<u>0.3048</u>
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Angular Momentum	$lb\cdot ft^2/s$	$kg\cdot m^2/s$	0.042 14
Plane Angle	°	°	no change

[7] Underline denotes exact conversion. All others conversion factors on this page are rounded to four significant figures.

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CONSULTANT: R. BERG P.E.
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16. Abstract

This manual is the reference text used for the FHWA Demonstration Project No. 82 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 MSEW 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 specification 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.

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SI CONVERSION FACTORS AND BASE UNITS

Quantity	From English Units	To SI Units	Multiply By[6]
Length	mile yard foot inch	km m m mm	1.609 0.9144 0.3048 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4047 0.404 0.836 0.092 645.2
Volume	acre foot cubic yard cubic foot cubic foot 1000 board feet gallon cubic inch	m^3 m^3 m^3 L (1000 cm^3) m^3 L (1000 cm^3) cm^3	1233 0.764 0.028 28.32 2.36 3.785 16.39
Mass	pound mass, lb-mass, lbm	kg	0.4536
Mass Density	lb-mass/ft^3	kg/m^3	16.02
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Force/Unit Length	lbs/ft kips/ft	N/m kN/m	14.59 14.59
Force/Unit Area, Pressure, Stress, Modulus of Elasticity.	lbs/in^2 kips/in^2 lbs/ft^2 kips/ft^2	kPa MPa Pa kPa	6.895 6.895 47.88 47.88
Force/Volume Unit Weight	lbs/ft^3 kips/ft^3	N/m^3 kN/m^3	157.1 157.1
Bending Moment, Torque, Moment of force	ft-lb ft-kip	N·m kN·m	1.356 1.356

[6] Underline denotes exact conversion. All others conversion factors on this page are rounded to four significant figures.

PREFACE

Engineers and specialty material suppliers have been designing reinforced soil structures for the past 25 years. During the last decade significant improvements have been made to design methods and in the understanding of factors affecting the durability of reinforcements.

In order to take advantage of these new developments, the FHWA has developed this manual in connection with Demonstration Project No. 82, Ground Improvement. The primary purpose of this manual is to support educational programs conducted by FHWA for transportation agencies. This program consists of (1) a workshop for geotechnical, structural, roadway and construction engineers and (2) technical assistance for project development in areas covered by this Demonstration Project on request to transportation agencies.

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

This Mechanically Stabilized Earth Walls (MSE) and Reinforced Soil Slopes (RSS), Design and Construction Guidelines Manual has evolved from the following AASHTO and FHWA references:

- *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.

- *Geosynthetic Design and Construction Guidelines*, by R.D. Holtz, B.R. Christopher, and R.R. Berg, FHWA HI-95-038.

- AASHTO, 1992 and 1996 Interims, Section 5.8.

- *Design and Construction Monitoring of Mechanically Stabilized Earth Structures*, by J.A. DiMaggio, FHWA, March 1994.

- AASHTO Bridge T-15 Technical Committee unpublished working drafts for the update of Section 5.8 of the AASHTO Bridge Design Specifications.

The authors recognize the efforts of Mr. Jerry A. DiMaggio, P.E. who was the FHWA Technical Consultant for this work, and served in the same capacity for most of the above referenced publications. Mr. DiMaggio's guidance and input to this and the previous works has been invaluable.

The authors further acknowledge the efforts of Mr. Tony Allen, Washington DOT, members of the AASHTO T-15 committee and the following Technical Working Group members who served as a review panel listed in alphabetical order:

Mr. Donald Bruce	-	ECO Geosystems Inc.
Mr. James Collin	-	The Collin Group
Mr. Albert DiMillio	-	FHWA
Mr. Richard Endres	-	Michigan DOT
Mr. John Hooks	-	FHWA
Mr. John Horvath	-	Manhattan College
Mr. Richard Sheffield	-	Mississippi DOT
Mr. Michael Simac	-	Ground Improvement Technologies
Mr. Ed Tavera	-	Louisiana DOT

Lastly, the authors wish to thank the clerical and computer graphics staff of Earth Engineering and Sciences, Inc. for their vital contributions and significant effort in preparing this manual.

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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) and at marginal sites with difficult subsurface conditions and other environmental constrains. Professionals charged with the responsibility of planning, designing, and implementing improvements and additions in such locations need to 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, and construction of Mechanically Stabilized Earth Walls (MSEW), and Reinforced Soil Slopes (RSS), and the monitoring of their long-term performance.

The design, construction and monitoring techniques for these structures have evolved over the last two 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 the first single, comprehensive document to integrate all design, construction, materials, contracting, and monitoring aspects required for successful project implementation.

This manual has been developed in support of FHWA's Demonstration Project No. 82 on the design and construction monitoring of MSEW retaining structures and RSS construction. Its principal function is to serve as a textbook and reference source to the materials presented in the demonstration project workshops, and as FHWA's primary guideline on this subject.

a. 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 steepened RSS.
- Design of steepened RSS over soft subgrades.
- Specifications and contracting approaches for both MSE walls and RSS construction.
- Construction monitoring and inspection.
- Design examples as case histories with detailed cost savings documented.
- A separate companion Manual addresses the long-term degradation of metallic and polymeric reinforcements. Sections of the 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 the Manual, several student exercises and workshop problems are included with solutions that demonstrate individual design aspects.

b. Source Documents

The majority of the material presented in this Manual was abstracted from FHWA RD89-043 *"Reinforced Soil Structures, Volume 1 Design and Construction Guidelines"*, *1992 AASHTO Specifications*, both Division 1, Design and Division II, Construction, and direct input from the AASHTO Bridge T-15 Technical Committee as part of their effort to update Section 5.8 of the AASHTO Bridge Specifications.

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

c. 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 (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 Mechanically Stabilized Earth that incorporate planar reinforcing elements in constructed earth-sloped structures with face inclinations of less than 70 degrees.

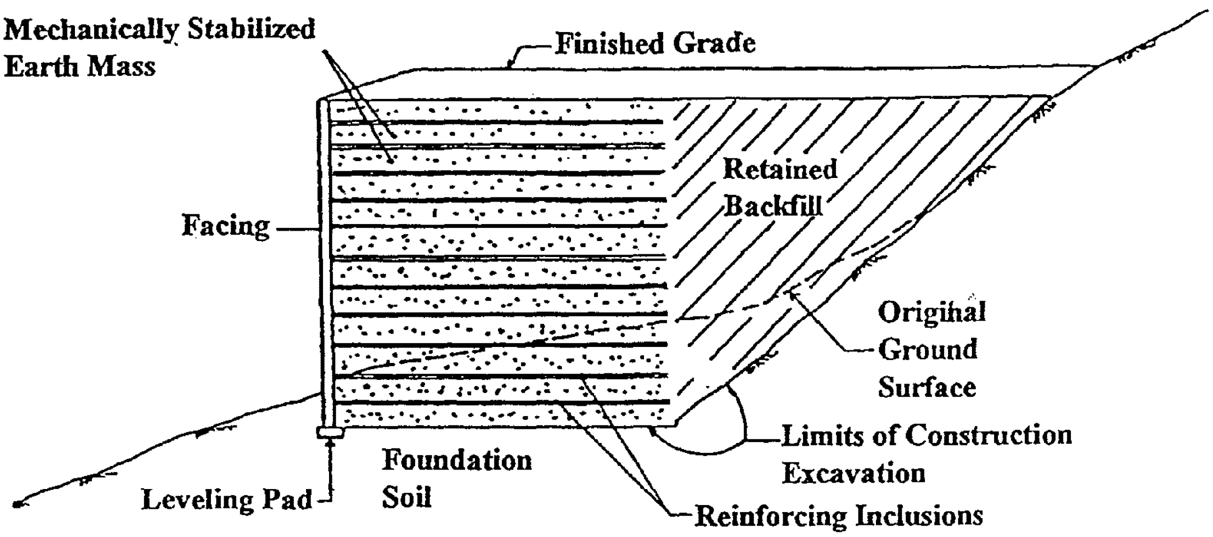
Geosynthetics is a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and grids (also known as geogrids).

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, metal sheets and plates, gabions, welded wire mesh, shotcrete, wood lagging and panels, 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 some type of erosion control material.

Retained backfill is the fill material located between the mechanically stabilized soil mass and the natural soil.

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

Generic cross sections of a mechanically stabilized soil mass in its geotechnical environment is shown in figures 1 and 4.



Mechanically Stabilized Earth Mass - Principal Elements

Figure 1. Generic cross section of a 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 (MSEW) and Reinforced Soil Slopes (RSS) 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 such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the inclusions can also withstand bending from shear stresses, providing additional stability to the system.

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 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 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. In the last 20 years, more than 20,000 Reinforced Earth structures representing over 70 million m^2 of wall facing have been completed in 37 countries. More than 7,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 28 meters.

Since the introduction of Reinforced Earth®, several other proprietary and nonproprietary systems have been developed and used. Table 1 provides a partial summary of some of the current systems by proprietary name, reinforcement type, and facing 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 remaining patents in force generally cover only the method of connection between the reinforcement and the facing.

For the first 20 years of use in the United States an articulating precast facing unit 2 to 2.25 m^2 generally square in shape, was the facing unit of choice. More recently, larger precast units of up to 5 m^2 have been used as have much smaller dry-cast units, generally in conjunction with geosynthetic reinforcements.

The use of geotextiles in MSE walls and RSS started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments 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. Since about 1980, the use of geotextiles in reinforced soil has increased significantly.

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.

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. The highest constructed RSS structure to date has been at 1H:1V to 33.5 m.

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

Current Usage

It is believed that MSE walls 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 700,000 m^2 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 MSE walls 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 MSE walls 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 has been reported that more than 200 such structures have been constructed in the United States, for highway applications to date. The current yearly usage for transportation- related applications is estimated at about 25 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, 30 to 40 RSS projects are being constructed yearly in connection with transportation related projects in the United States, with an estimated projected vertical face area of 70,000 m^2/year.

Table 1. Summary of reinforcement and face panel details for selected MSE wall systems.

System Name	Reinforcement Detail	Typical Face Panel Detail[1]
Reinforced Earth The Reinforced Earth Company 2010 Corporate Ridge McLean, VA 22102	Galvanized Ribbed Steel Strips: 4 mm thick, 50 mm wide. Epoxy-coated strips also available.	Facing panels are cruciform shaped precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
VSL Retained Earth VSL Corporation, 2840 Plaza Place Raleigh, NC 27612	Rectangular grid of W11 or W20 plain steel bars, 610 x 150 mm grid. Each mesh may have 4, 5 or 6 longitudinal bars. Epoxy-coated meshes also available.	Hexagonal and square precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.
Mechanically Stabilized Embankment Dept. of Transportation, Division of Engineering Services 5900 Folsom Blvd. P.O. Box 19128 Sacramento, CA 95819	Rectangular grid, nine 9.5 mm diameter plain steel bars on 610 x 150 mm grid. Two bar mats per panel (connected to the panel at four points).	Precast concrete; rectangular 3.81 m long, 610 mm high, 200 mm thick.
Georgia Stabilized Embankment Dept. of Transportation, State of Georgia No. 2 Capitol Square Atlanta, GA 30334-1002	Rectangular grid of five 9.5 mm plain steel bars on 610 x 150 mm grid 4 bar mats per panel	Precast concrete panel; rectangular 1.83 m wide, 1.22 m high, 200 mm thick with offsets for interlocking.
Hilfiker Retaining Wall Hilfiker Retaining Walls, P.O. Drawer L Eureka, CA 95501	Welded steel wire mesh, grid 50 x 150 mm of W4.5 x W3.5, W9.5 x W4, W9.5 x W4, and W12 x W5 in 2.43 m wide mats.	Welded steel wire mesh, wrap around with additional backing mat 6.35 mm wire screen at the soil face (with geotextile or shotcrete, if desired).
Reinforced Soil Embankment Hilfiker Retaining Walls, P.O. Drawer L Eureka, CA 95501	15 cm x 61 cm welded wire mesh: W9.5 to W20 - 8.8 to 12.8 mm diameter.	Precast concrete unit 3.8 m long, 610 mm high.
ISOGRID Neel Co. 6520 Deepford Street Springfield, VA 22150	Rectangular grid of W11 x W11 5 bars per grid	Diamond shaped precast concrete units, 1.5 by 2.5 m, 140 mm thick.
GENESIS Tensar Earth Technologies, Inc. 5775-B Glenridge Drive, Ste 400 Lakeside Center Atlanta, GA 30328	HDPE Geogrid	Keystone® Standard unit (200 mm high by 40 mm long face, 600 mm nominal depth); OR Keystone International Compact® (200 mm high by 450 mm long face, 300 mm nominal depth).
PYRAMID The Reinforced Earth® Company 2010 Corporate Ridge McLean, VA 22102	Galvanized WWM, size varies with design requirements or Grid of PVC coated, Polyester yarn (Matrex Geogrid)	Pyramid® unit (200 mm high by 400 mm long face, 250 mm nominal depth)
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 230 and 450 mm.	Precast concrete units, rectangular or wing shaped 1.82 m x 2.13 m x 140 mm.

[1]Additional facing types are possible with most systems.

Table 2. Representative list of Geotextile and Geogrid manufacturers and suppliers.[1]

Akzo Nobel Industrial Systems Ridgefield Business Center Suite 318, Ridgefield Court Asheville, NC 28802	Amoco Fabrics and Fibers Co. 900 Circle 75 Parkway, Suite 300 Atlanta, GA 30339	Bayex Inc. 14770 East Ave. P.O. Box 390 Albion, NY 14411-0390
Carthage Mills 4243 Hunt Road Cincinnati, OH 45242	Hoechst Celanese Corp. P.O. Box 5650 I-85 & Road 57 Spartanburg, SC 29304	Huesker, Inc. 11107 A S. Commerce Blvd. Charlotte,NC 28241
LINQ Industrial Fabrics, Inc. 2550 West 5th North Street Summerville, SC 29483	Nicolon Corporation 3500 Parkway Lane, Suite 500 Norcross, GA 30092	Reemay, Inc. 70 Old Hickory Blvd. Old Hickory, TN 37138
Spartan Technologies P.O. Box 1658 Spartanburg, SC 29304	Strata Systems, Inc. 425 Trible Gap Road Cummings, GA 30130	Synthetic Industries Construction Products Division 4019 Industry Drive Chattanooga, TN 37416
Tenax Corporation 4800 East Monument Street Baltimore, MD 21205	Tensar Earth Technologies 5775-B Glenridge Drive Suite 450, Lakeside Center Atlanta, GA 30328	Wellman, Inc. 2748 Tanager Ave. Commerce, CA 90040

[1] List is from the Industrial Fabrics Association International, Geotextile and Geomembrane Divisions membership list.

CHAPTER 2

SYSTEMS AND PROJECT EVALUATION

This chapter initially describes available MSEW and RSS systems and components, their application, advantages, disadvantages and relative costs.

Subsequently, it outlines required site and project evaluations leading to the establishment of site specific project criteria and details typical construction sequence for MSEW and RSS construction.

2.1 APPLICATIONS

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.

Some additional successful uses of MSE walls include:

- Temporary structures, which have been especially cost-effective for temporary detours necessary for highway reconstruction projects.

- Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is economical and can also result in savings of land because a vertical face can be used, which reduces construction time).

- Dams and seawalls, including increasing the height of existing dams.
- Bulk materials storage using sloped walls.

Representative uses of MSE walls for various applications are shown in figures 2 and 3.

Reinforced Soil Slopes, are cost-effective alternatives for new construction where the cost of fill, right-of-way, and other considerations may make a steeper slope desirable. However, even if foundation conditions are satisfactory, slopes may be unstable at the desired slope angle. Existing slopes, natural or manmade, may also be unstable as is usually painfully obvious when they fail. As shown in figure 4, multiple layers of reinforcement may be placed in the slope during construction or reconstruction to reinforce the soil and provide increased slope stability. Reinforced slopes are a form of mechanically stabilized earth that incorporate planar reinforcing elements in constructed earth sloped structures with face inclinations of less than 70 degrees. Typically, geosynthetics are used for reinforcement.

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 4a.
- To provide improved compaction at the edges of a slope, thus decreasing the tendency for surface sloughing as shown in figure 4b.

The principal purpose for using reinforcement is to construct an RSS 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 replacements for flatter unreinforced slopes and retaining walls. Roadways can also be widened over existing flatter slopes without encroaching on 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 5.

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.

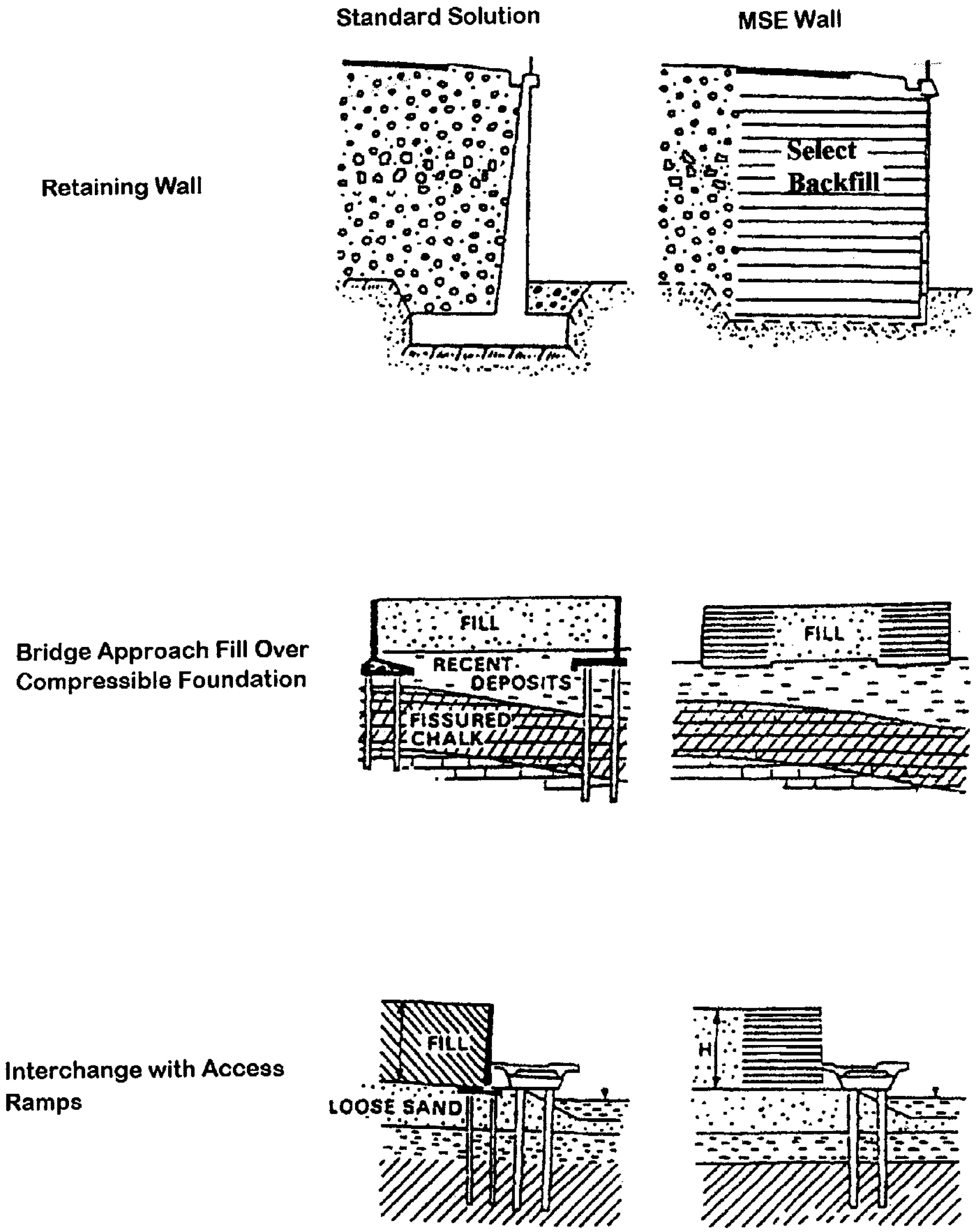


Figure 2. MSE Wall. Urban applications.

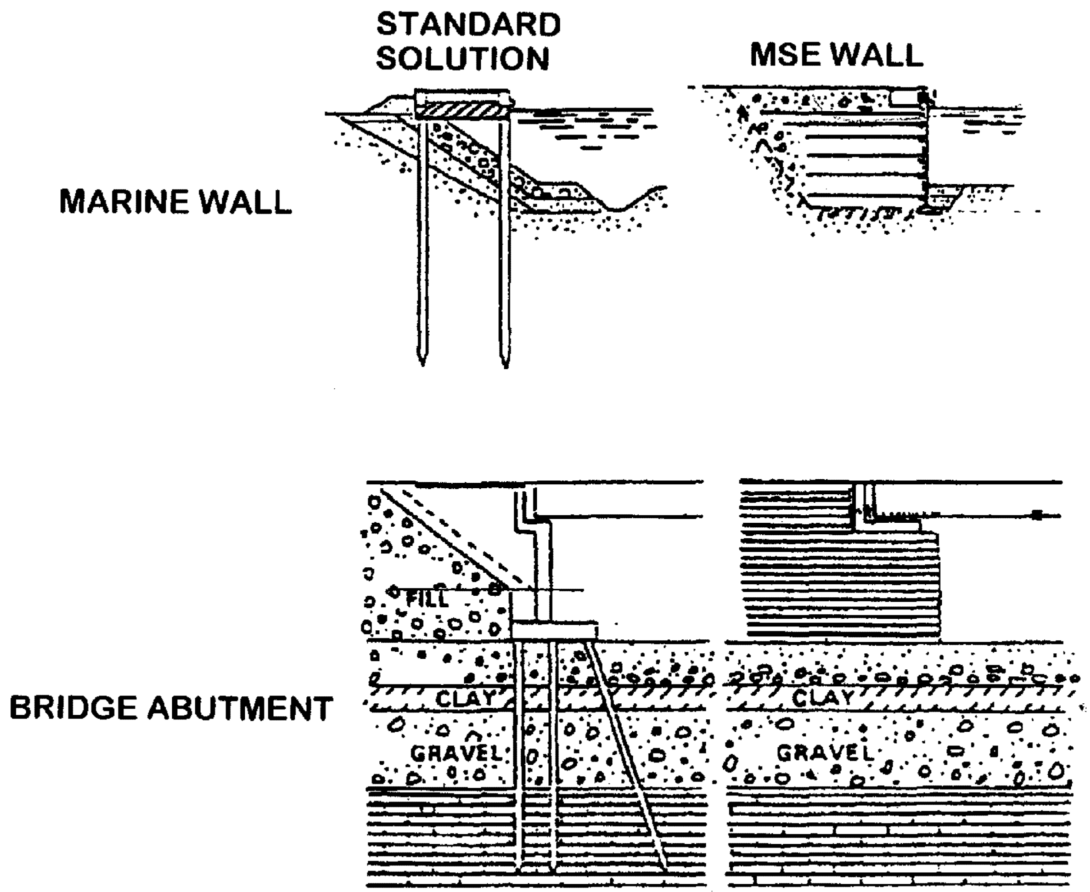


Figure 3. MSE wall applications, abutments, and marine.

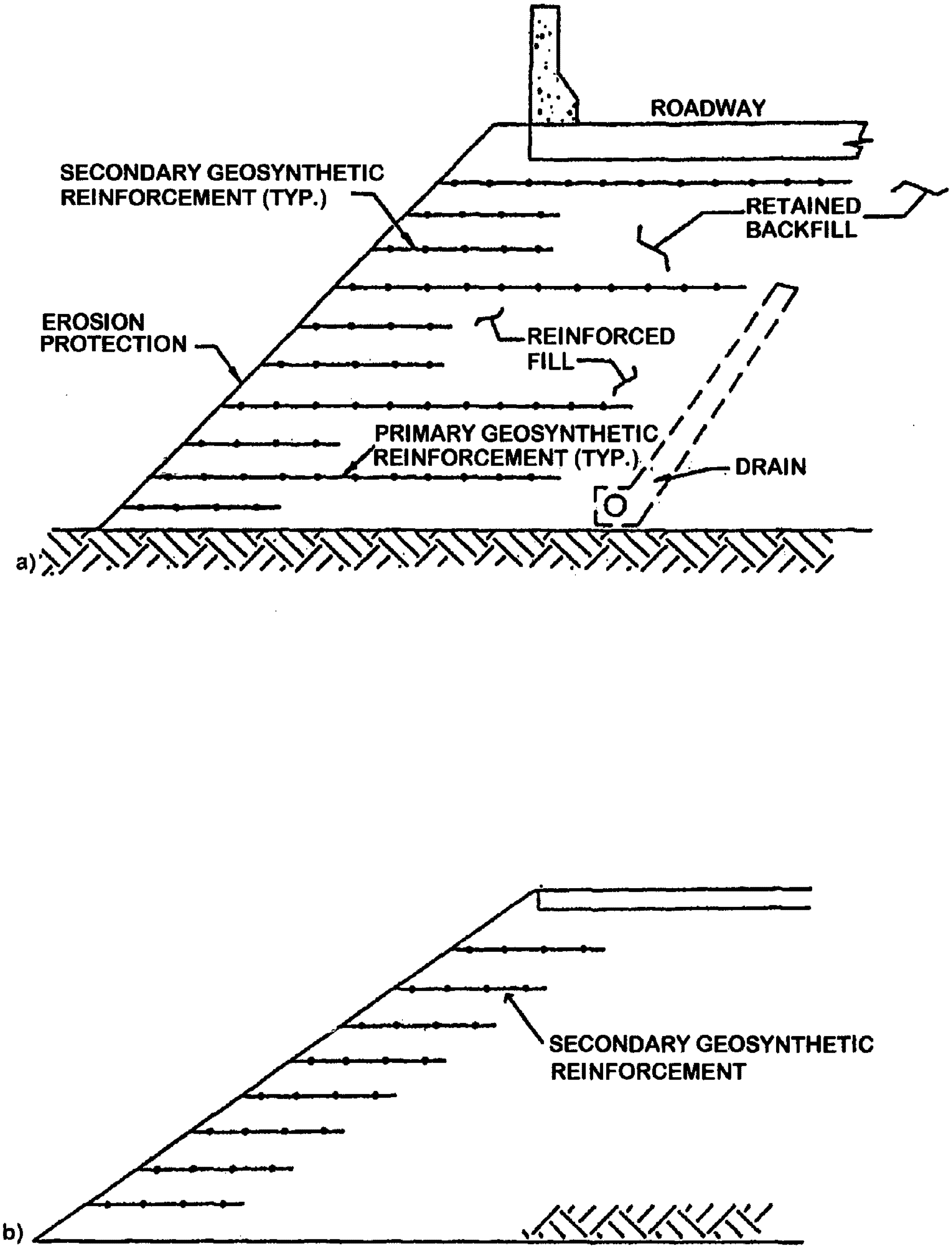


Figure 4. Slope reinforcement using geosynthetics to provide slope stability.

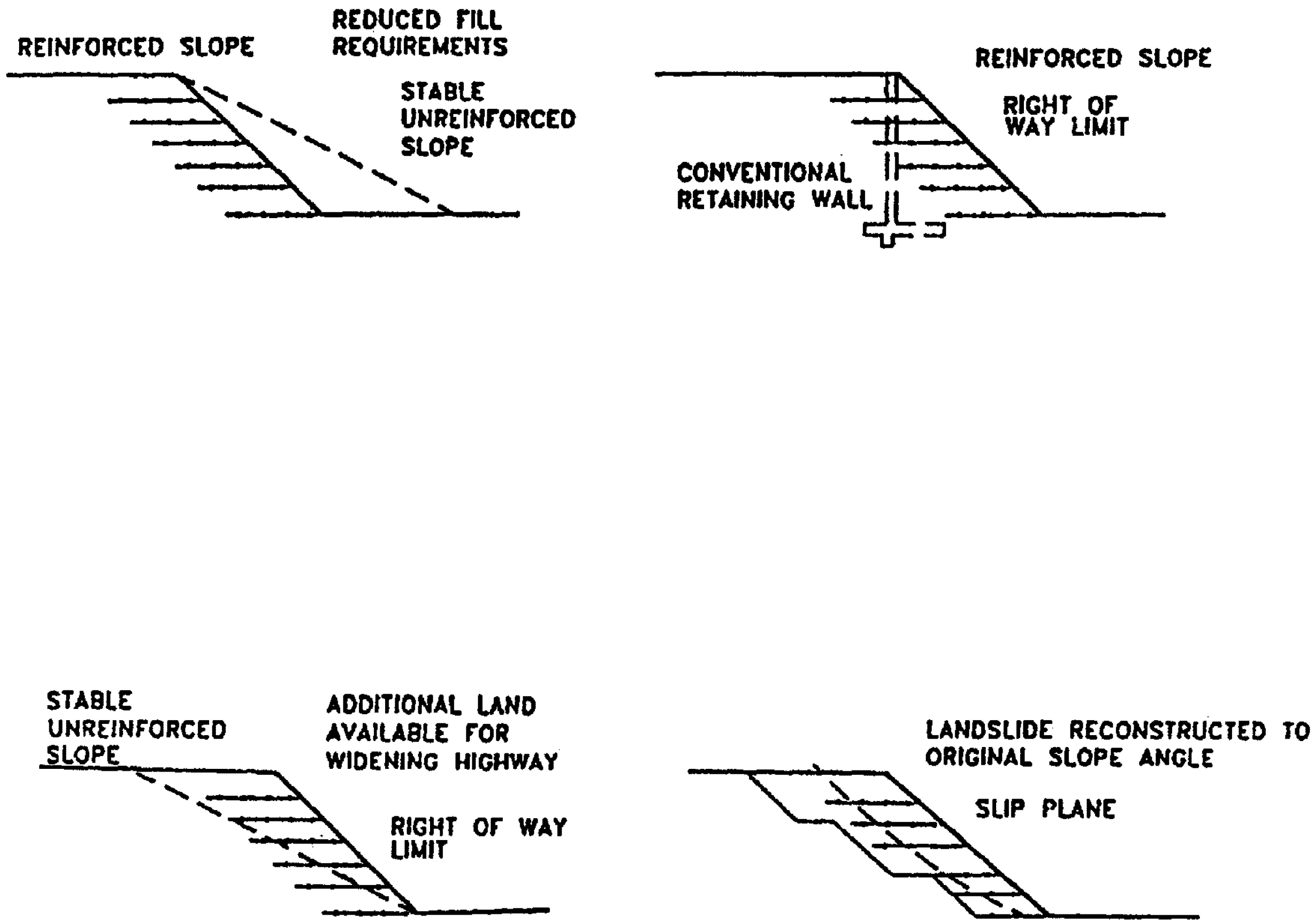


Figure 5. Application of reinforced soil slopes.

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 4a.

Other applications of reinforced slopes have included:

- Upstream/downstream face improvements to increased height of dams.
- Permanent levees.
- Temporary flood control structures.
- Decreased bridge spans.
- Temporary road widening for detours.
- Prevention of surface sloughing during periods of saturation.
- Embankment construction with wet, fine-grained soils.

2.2 ADVANTAGES AND DISADVANTAGES

a. 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 large construction equipment.
- Do not require experienced craftsmen with 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 25 m.

The relatively small quantities of manufactured materials required, rapid construction, and, competition among the developers of 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 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 absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, these structures have demonstrated a higher resistance to seismic loading than have rigid concrete 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 with advantage to blend in the environment.

b. Advantages of Reinforced Soil Slopes (RSS)

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the resulting material and rights-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 type structures. However, there are some maintenance issues that must be addressed such as mowing grass-faced, steep slopes.

In terms of performance, due to inherent conservatism in the design of RSS, they are actually safer than flatter slopes designed at the same factor of safety. As a result, there is a lower risk of long-term stability problems developing in the slopes. 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 though improved drainage, further improving long-term performance.

c. Disadvantages

The following general disadvantages may be associated with all soil reinforced structures:

- Require a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability.

- MSEW require select granular fill. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical). Requirements for RSS are typically less restrictive.

- Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as geosynthetics by ultra violet rays, and potential degradation of polymer reinforcement in the ground.

- Since design and construction practice of all reinforced systems are still evolving, specifications and contracting practices have not been fully standardized, especially for RSS.

- The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners and greater input from agencies geotechnical specialists in a domain often dominated by structural engineers.

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. 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 3 m and average foundation conditions. Modular block walls (MBW) are competitive with concrete walls at heights of less than 4.5 m.

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system. A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can absorb relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. A comparison of wall material and erection costs for several reinforced soil retaining walls and other retaining wall systems is shown in figure 6. Typical total costs range from \$160 to \$300 per m^2 of face, generally as function of height and cost of select fill.

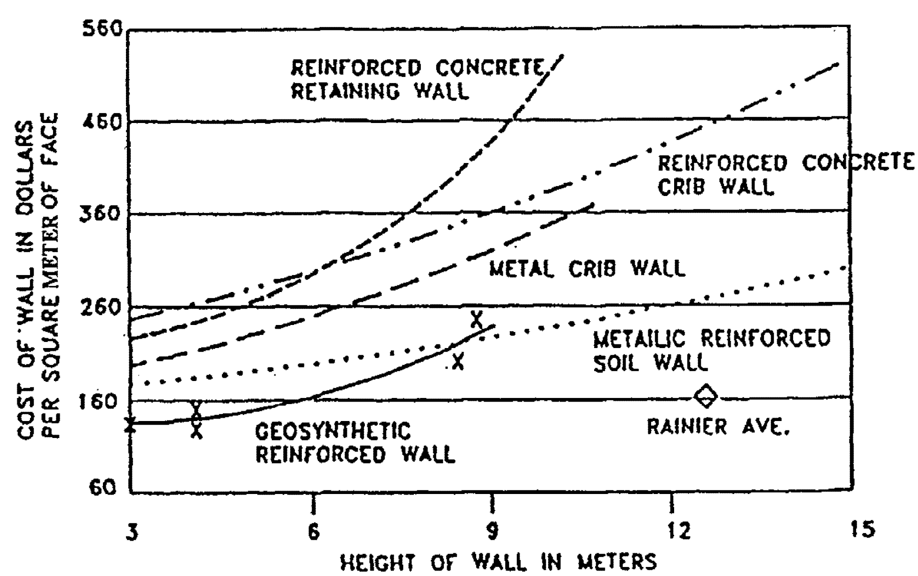


Figure 6. Cost comparison for retaining walls (from Geosynthetic Design and Construction Guidelines, 1995, H1-95-038).

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 - 20 to 30 percent of total cost.
- Facing system - 25 to 30 percent of total cost.
- Backfill materials including placement - 35 to 40 percent of total cost, where the fill is a select granular fill from an off site borrow source.

In addition, consideration must be given to the cost of excavation which may be somewhat greater than for other systems.

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 carried out to see if the 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 slope protection or facings 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 and 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.

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

- Backfill - 30 to 45 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 \$110 m^2 to \$260 m^2 as a function of height.

For applications in the 10 to 15 m height range bid costs of about \$170 m^2 have been reported.

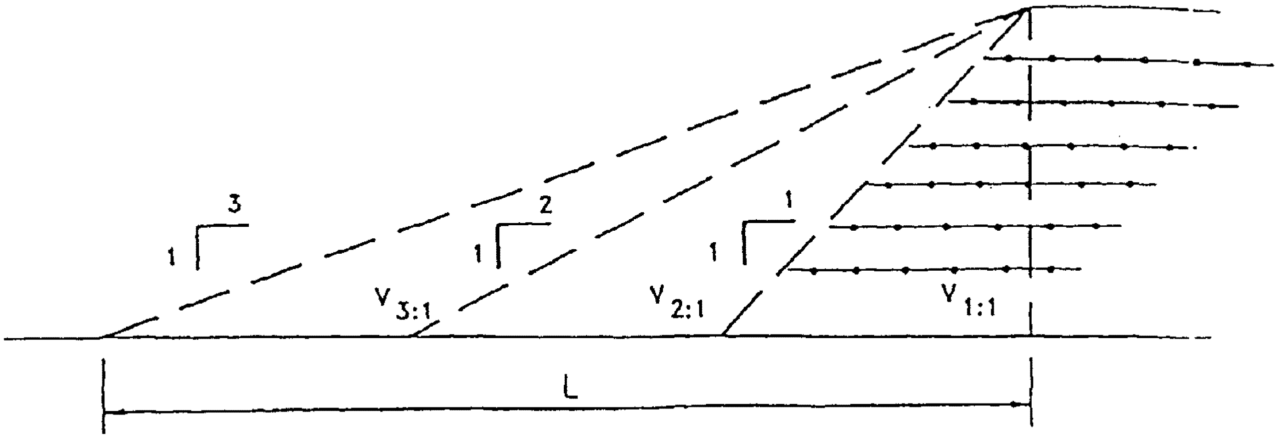
Figure 7 provides a rapid, first-order assessment of cost items for comparing a flatter unreinforced slope with a steeper reinforced slope.

2.4 DESCRIPTION OF MSE/RSS SYSTEMS

a. Systems Differentiation

Since the expiration of the fundamental process and concrete facing panel patents obtained by the Reinforced Earth Co. for MSEW systems and structures, the engineering community has adopted a generic term *Mechanically Stabilized Earth* 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.



$V_{3:1} = V$
$V_{2:1} = \frac{2}{3}V$
$V_{1:1} = \frac{1}{3}V$

$V_{3:1} = L$
$V_{2:1} = \frac{2}{3}L$
$V_{1:1} = \frac{1}{3}L$

<u>COST:</u>

3:1 = V_{SOIL} + L_{LAND} + Guardrail + Hydroseeding
2:1 = $\frac{2}{3}V_{SOIL}$ + $\frac{2}{3}L_{LAND}$ + Guardrail + Erosion Control + High Maintenance
1:1 = $\frac{1}{3}V_{SOIL}$ + $\frac{1}{3}L_{LAND}$ + Reinforcement + Guardrail + Erosion Control

Figure 7. Cost evaluation of reinforced soil slopes.

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 soil reinforced structure. Often technical assistance during the planning and construction phase is also included. Components marketed by commercial entities for integration by the owner in a coherent system are not classified as systems.

b. 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 150 mm.

- **Planar bidirectional.** Continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh. The mesh is characterized by element spacing of less than 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 or may be epoxy coated.

- **Nonmetallic reinforcements.** Generally polymeric materials consisting of polypropylene, polyethylene, or polyester.

The performance and durability considerations for these two classes of reinforcement vary considerably and are detailed in the companion Corrosion/Degradation document.

Reinforcement Extensibility

There are two classes of extensibility:

- **Inextensible.** The deformation of the reinforcement at failure is much less than the deformability of the soil.

- **Extensible.** The deformation of the reinforcement at failure is comparable to or even greater than the deformability of the soil.

c. 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. 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** summarized in Table 1 and illustrated in figure 8. The precast concrete panels have a minimum thickness of 140 mm and are of a cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required but will vary with the size of the panel. Vertically adjacent units are usually connected with shear pins.

- **Dry cast segmental blocks (MBW) units**. These are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units commonly ranges from 15 to 50 kg, with units of 35 to 50 kg routinely used for highway projects. Unit heights typically range from 100 to 200 mm for the various manufacturers. Exposed face length usually varies from 200 to 450 mm. Nominal width (dimension perpendicular to the wall face) of units typically ranges between 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) 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®, Versalock, Allen etc. They are illustrated in figure 9.

- **Metallic Facings**. The original Reinforced Earth® system had facing elements of galvanized steel sheet formed into half cylinders. Although precast concrete panels are now usually used in Reinforced Earth walls, metallic facings may be appropriate in structures where difficult access or difficult handling requires lighter facing elements.

- **Welded Wire Grids**. Wire grid can be bent up at the front of the wall to form the wall face. This type of facing is used in the Hilfiker, Tensar, and Reinforced Earth wire retaining wall systems.

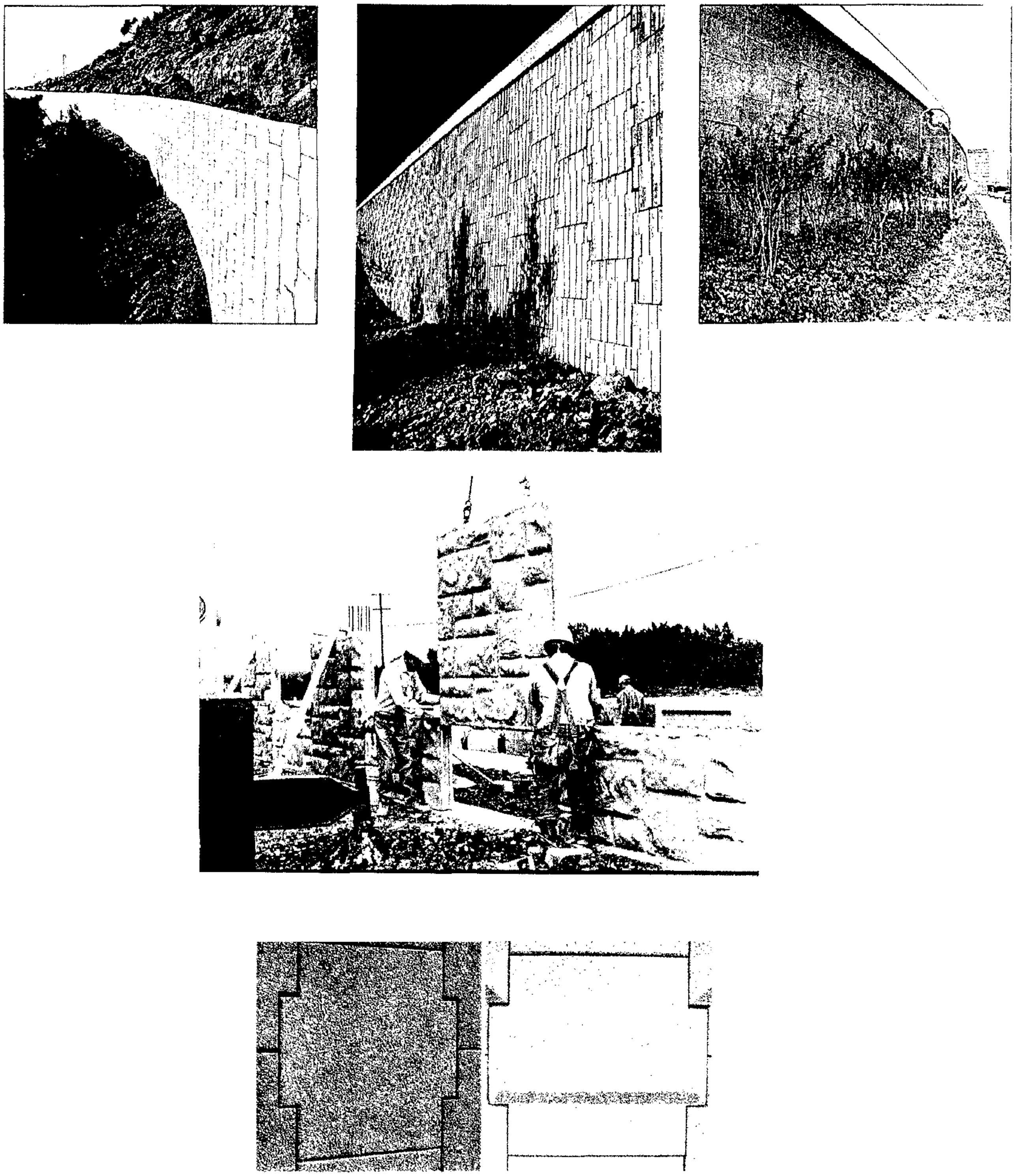


Figure 8. MSE wall surface textures.

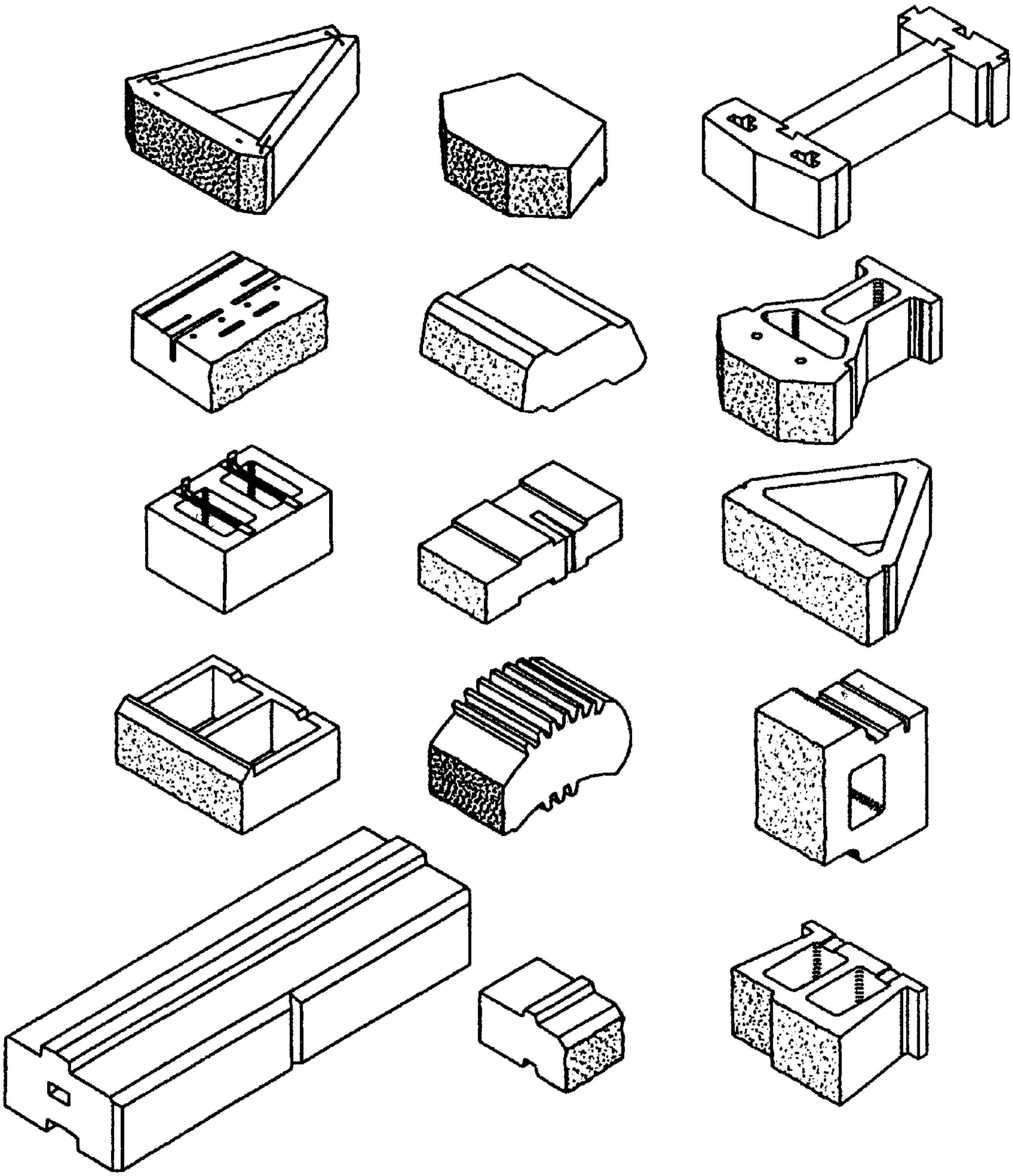


Figure 9. Examples of commercially available MBW units.
(from Design Manual for Segmental Retaining Walls)

- **Gabion Facing**. Gabions (rock-filled wire baskets) can be used as facing with reinforcing elements consisting of welded wire mesh, welded bar-mats, geogrids, geotextiles or the double-twisted woven mesh placed between or connected to the gabion baskets.

- **Geosynthetic Facing.** Various types of geotextile reinforcement are looped around at the facing to form the exposed face of the retaining wall. These faces are susceptible to ultraviolet light degradation, vandalism (e.g. target practice) and damage due to fire. Alternately, a geosynthetic grid 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.

- **Postconstruction 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 attaching prefabricated facing panels made of concrete, wood, or other materials. This 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. The greatest advantages of such facings are low cost, ease of installation, design flexibility, good drainage (depending on the type of backfill) 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.

Recently introduced dry cast segmental block MBW facings raise some concerns as to their durability in aggressive freeze-thaw environments because their water absorption capacity can be significantly higher than that of wet-cast concrete. Historical data provide little insight as their usage history is less than a decade. 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 become an aggressive aging media for some geosynthetic products potentially used as reinforcements. Freeze-thaw durability is enhanced for products produced at higher compressive strengths and/or sprayed with a posterection sealant.

The outward faces of slopes in RSS structures are usually vegetated if 1:1 or flatter. The vegetation requirements vary by geographic and climatic conditions and are therefore, project specific. Details are outlined in chapter 6, section 6.5.

d. Reinforced Backfill Materials

MSEW Structures

MSE walls require high quality backfill 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 backfill is still critical. These performance requirements generally eliminate soils with high clay contents.

From a reinforcement capacity point of view, lower quality backfills could be used for MSEW structures; however, a high quality granular backfill has the advantages of being free draining, 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.

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 wet-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 less durability concerns for the reinforcement, and is easier to handle, place and compact, which speeds up construction.

e. Miscellaneous Materials of Construction

Walls using precast concrete panels require bearing pads in their horizontal joints that provide some compressibility and movement between panels and preclude concrete to concrete contact. These materials are either neoprene or SBR rubber.

All joints are covered with a polypropylene (PP) geotextile strip to prevent the migration of fines from the backfill. Vertical joints, if large, may be filled in addition with a synthetic foam. The compressibility of the horizontal joint material should be a function of the wall height. Walls with heights greater than 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.

2.5 SITE EVALUATION

a. 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.

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 construction slopes that may be required.

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

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

b. 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 ground water-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 which 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 Soil Conservation Service, the U.S. Department of Agriculture, and local planning boards or county offices.

c. **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 1988 AASHTO "*Manual on Foundation Investigations*", should be reviewed.

The following minimum guidelines are recommended for the subsurface exploration for potential MSE applications:

- Soil borings should be performed at intervals of:

  - 30 m along the alignment of the soil-reinforced structure
  - 45 m along the back of the reinforced soil structure

  The width of the MSE wall or slope structure may be assumed as 0.8 times the anticipated height.

- 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 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 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 D-3441, 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 indicted 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 data.

The development and implementation of an adequate subsurface investigation program 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. 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.

d. Laboratory Testing

Soil samples should be visually examined and appropriate tests performed for classification according to the Unified Soil Classification System (ASTM D 2488-69). 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, compressive strength, 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 compression 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 making 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 compaction 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.
- Electrical resistivity.
- Salt content including sulfate, sulfides, and chlorides.

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

2.6 PROJECT EVALUATION

a. 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 outward face. These factors again may create an initial difficulty in adequate technical evaluation. 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.

b. 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.

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 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 wick drains.
- Construction of stone 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 staging 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 should not be considered where differential settlements are anticipated.

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

c. 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.

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.[1] Alkaline regimes characterized by pH > 10 will cause accelerated loss of galvanization. Under these conditions, bare steel reinforcements could be considered.

Certain in situ regimes have been identified as being potentially aggressive for geosynthetic reinforcements, although at present research is being conducted to quantify the degree of degradation and the specific conditions necessary.

For additional specific discussions on the potential degradability of reinforcements, refer to the companion Corrosion/Degradation reference document and chapter 3, section 3.5.

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

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.

d. Size and nature of structure

Theoretically there is no upper limit to the height of MSEW that can be constructed. Structures in excess of 25 m have been successfully constructed with steel reinforcements although such heights for transportation-related structures are rare. RSS embankments have been constructed to equal heights.

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 bearing capacity in the reinforced fill is usually limited to 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 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 3 to 4 meters 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 meters of face) has little impact on economy compared with other retaining wall types. However, the unit cost for small projects of less than 300 m^2 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 always more economical than vertically faced MSEW structures.

e. 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 400 m^2) 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.

f. Questionable Applications

The current AASHTO *Interim Specifications for Highway Bridges*, indicates 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. A similar limitation should be considered for RSS structures.

FHWA RD89-043 further suggests that, based on current experience, the following additional limitations may be warranted:

- MSEW should be limited to 30 m in height for inextensible steel reinforcements.
- MSEW should be limited to 15 m in height for extensible geosynthetic reinforcements.

2.7 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.

a. 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.

b. 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 vertical face and cannot accommodate small, uniform front batters. Currently, the size of panels commercially produced varies from 1.8 to 4.5 m^2. Full height panels may be considered for walls up to 4 to 5 m in height on foundations that are not expected to settle. The precast concrete panels can be manufactured with a variety of surface textures and geometrics, as shown in figure 8.

MBW facings are available in a variety of shapes and textures as shown in figure 9. They range in facial area from 0.05 to 0.1 m^2. An integral feature of this type of facing is a front batter ranging from nominal to 15 degrees.

At more remote locations, gabion, timber faced, or vegetated MSE may be considered.

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

For RSS structures, the choice of slope facing may be controlled by climatic and regional factors. For structures of less than 10 m height with slopes of 1:1 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 chapter 6, section 6.5.

c. Performance Criteria

Performance criteria for MSE structures with respect to design requirements are governed by design practice or codes such as contained in Article 5.8 of the 1994 Interim AASHTO Specifications for Highway Bridges. These requirements consider the required margins of safety with respect to failure modes. They are equal for all types of MSEW structures. No specific AASHTO guidance is presently available for RSS structures.

With respect to lateral wall displacements, no method is presently available to definitely 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 10.

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. It further suggests that the anticipated construction deformation of MSE structures constructed with polymeric reinforcements (extensible) is approximately three times greater than if constructed with metallic reinforcements (inextensible).

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.

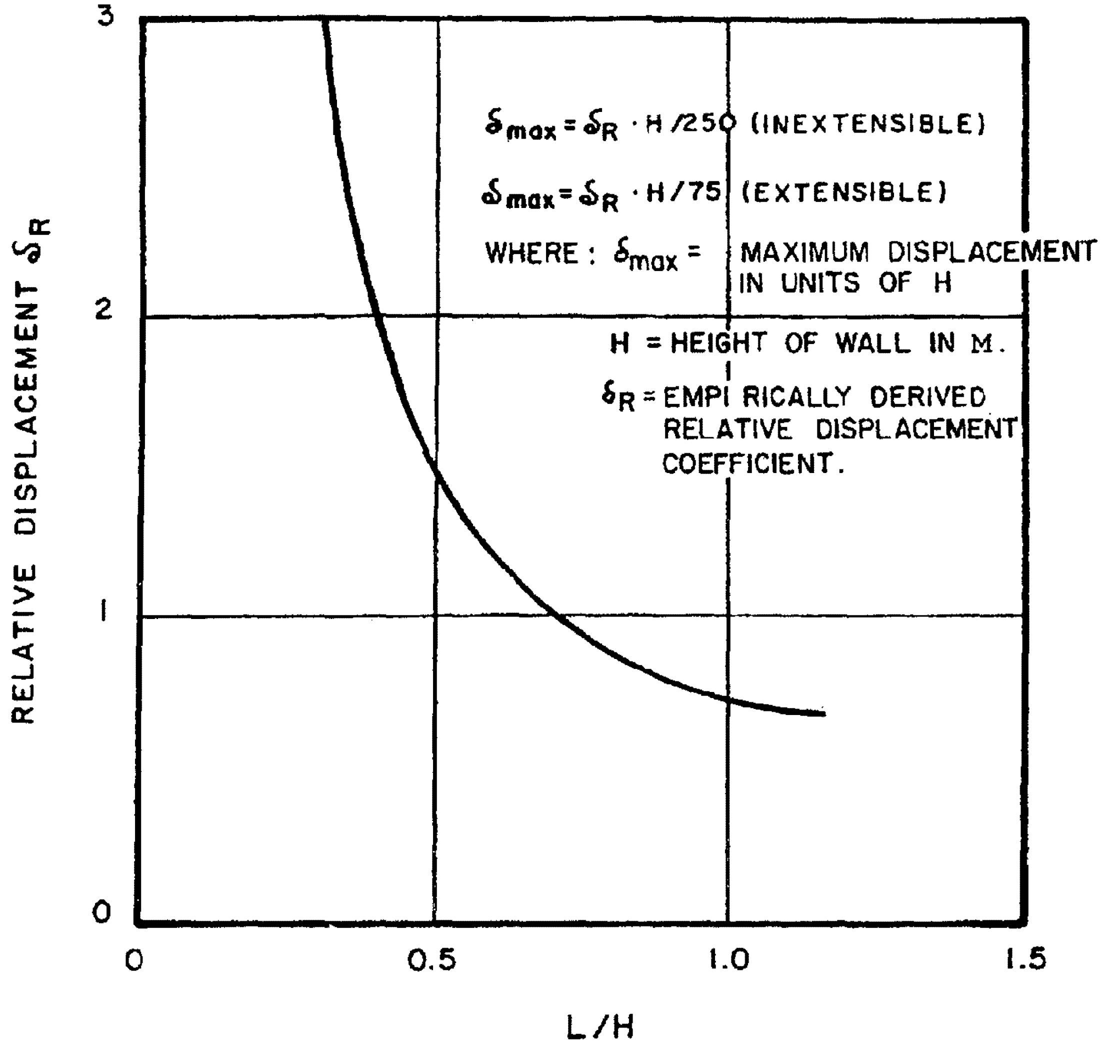
Recommended factors of safety with respect to failure modes are as follows:

- External Stability

Sliding	:	F.S. $\geq$ 1.5 (MSEW); 1.3 (RSS)
Eccentricity e, at Base	:	$\leq$ L/6 in soil L/4 in rock
Bearing Capacity	:	F.S. $\geq$ 2.5
Deep Seated Stability	:	F.S. $\geq$ 1.3
Seismic Stability	:	F.S. $\geq$ 75% of static F.S. (All failure modes)

- Internal Stability

Pullout Resistance (MSEW and RSS)	:	F.S. $\geq$ 1.5
Internal Stability for RSS	:	F.S $\geq$ 1.3



NOTE: INCREASE RELATIVE DISPLACEMENT 25% FOR EVERY 20 kPa OF SURCHARGE.

Based on 6 m high walls, relative displacement increase approximately 25% for every 20 kPa of surcharge. Experience indicates that for higher walls, the surcharge effects may be greater.

Note that actual displacements will also depend on soil characteristics, compaction effort and contractor workmanship.

Figure 10. Empirical curve for estimating probable anticipated lateral displacement during construction for MSE walls.

Allowable Tensile Strength		
for steel strip reinforcement	:	0.55 F_y
for steel grid reinforcement	:	0.48 F_y (connected to rigid facings)
Allowable Tensile Strength for geosynthetic reinforcements	:	T_a - See design life, below

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.

- **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.

- **External loads**. The external loads may be soil surcharges required by the geometry, adjoining footing loads, line loads as from traffic, and/or traffic impact loads. Traffic line loads and impact loads are applicable where the traffic lane is located horizontally from the face of the wall within a distance less than one half the wall height. The magnitude of the minimum loads outlined in Articles 3.20.3 and 5.8 of current AASHTO, is a uniform load equivalent to 0.6 m of soil over the traffic lanes.

- **Wall embedment**. The minimum embedment depth for walls from adjoining finished grade to the top of the leveling pad should be based on bearing capacity, settlement and stability considerations. Current practice based on local bearing capacity considerations, recommends the following embedment depths:

Slope in Front of Wall	Minimum to Top of Leveling Pad
horizontal (walls)	H/20
horizontal (abutments)	H/10
3H:1V	H/10
2H:1V	H/7
3H:2V	H/5

Larger values may be required, depending on depth of frost penetration, shrinkage and swelling of foundation soils, seismic activity, and scour. Minimum in any case is 0.5 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 backfill, hence reducing the overall wall height.

A minimum horizontal bench 1.2 m wide shall be provided in front of walls founded on slopes.

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

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

- **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.

  The peak horizontal ground acceleration for each site can be obtained from Section 3 of AASHTO Division 1-A, Seismic Design. For sites where the Acceleration Coefficient "A" in AASHTO is less or equal to 0.05, static design considerations govern and dynamic performance or design requirements may be omitted.

  For sites where the Acceleration Coefficient is greater than 0.29, significant total lateral structure movements may occur, and a seismic design specialist should review the stability and potential deformation for the structure. All sites where the "A" coefficient is greater than 0.05 should be designed/checked for seismic stability. For RSS structures, seismic analyses should be made for all sites.

- **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 settlement are anticipated (less 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. Guidance on minimum joint width and limiting differential settlements that can be tolerated is presented in table 3.

Table 3. Relationship between joint width and limiting differential settlements for MSE precast panels.

Joint Width	Limiting Differential Settlement
20 mm	1/100
13 mm	1/200
6 mm	1/300

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 settlements perpendicular to the wall face are anticipated, the reinforcement connection must allow for vertical movement or the reinforcement placed on a sloping fill surface which is higher at the back end of the reinforcement to compensate for the greater vertical settlement. This latter construction technique, however, requires that surface drainage be carefully controlled after each day's construction.

Alternately, where significant differential settlements are anticipated, ground improvement techniques may be warranted to limit the settlements, as outlined in geological conditions.

d. Design Life

MSE walls shall 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 which support 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 be essentially maintenance free throughout their design service life.

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

2.8 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.

a. Construction of MSEW systems with precast facings

The construction of MSEW systems with a precast 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 dynamic compaction, stone columns, wick drains, or other foundation stabilization/improvement methods 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 300 wide and 150 mm thick and is used for MSEW construction only, where concrete panels are subsequently erected. A gravel pad has been often substituted for MBW construction.

  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, metal facing panels, or dry cast modular blocks.

  The first row of facing panels may be full, or half-height panels, depending upon the type of facing used. The first tier of panels must be shored up to maintain stability and alignment. For construction with modular dry-cast blocks, full sized blocks are used throughout with no shoring.

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

- **Placement and compaction of backfill 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 300 mm. Reinforced backfill should be dumped into or parallel to the rear and middle of the reinforcement and bladed toward the front face. Random fill placement behind the reinforced volume should proceed simultaneously.

- **Placement of the first layer of reinforcing elements on the backfill.** The reinforcements are placed and connected to the facing panels, when the fill has been brought up to the level of the connection they are generally placed perpendicular to back of the facing panels. More detailed construction control procedures associated with each construction step are outlined in chapter 9.

- **Placement of the backfill over the reinforcing elements to the level of the next reinforcement layer and compaction of the backfill.** 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 backfill has been completed to its final grade.

  A complete sequence is illustrated in figures 11 through 13.

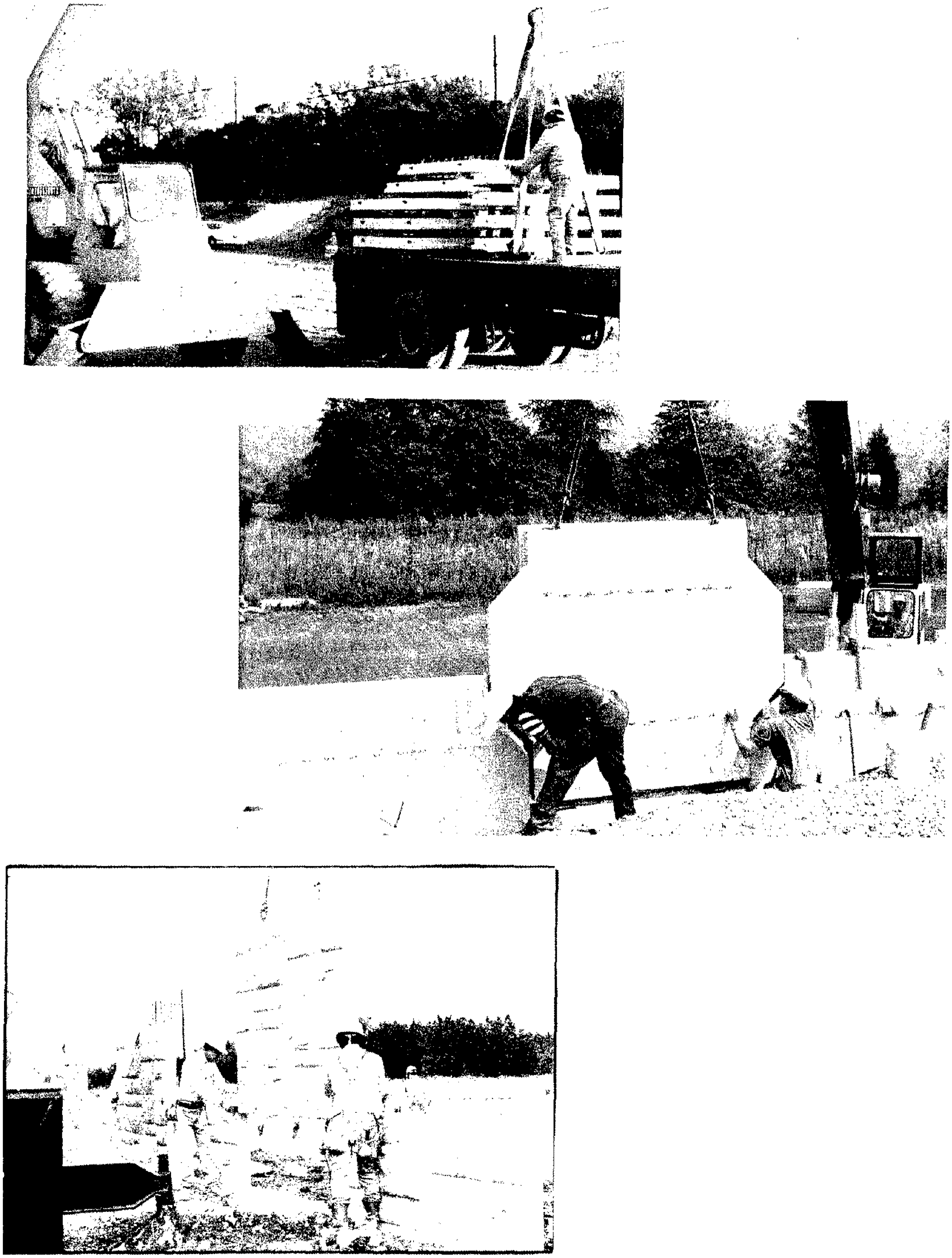
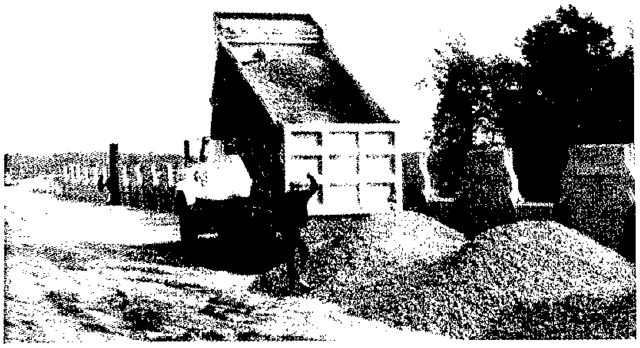
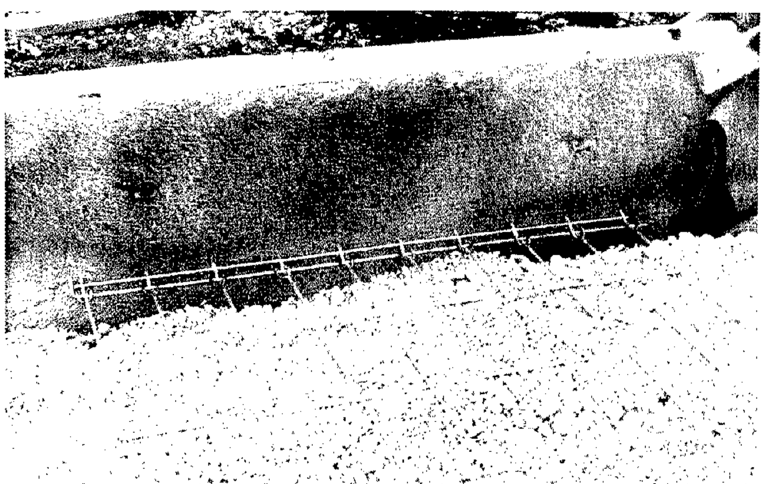


Figure 11. Erection of precast panels.





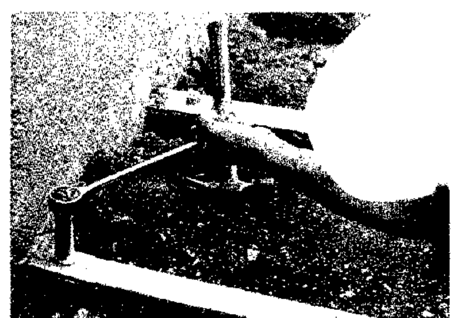
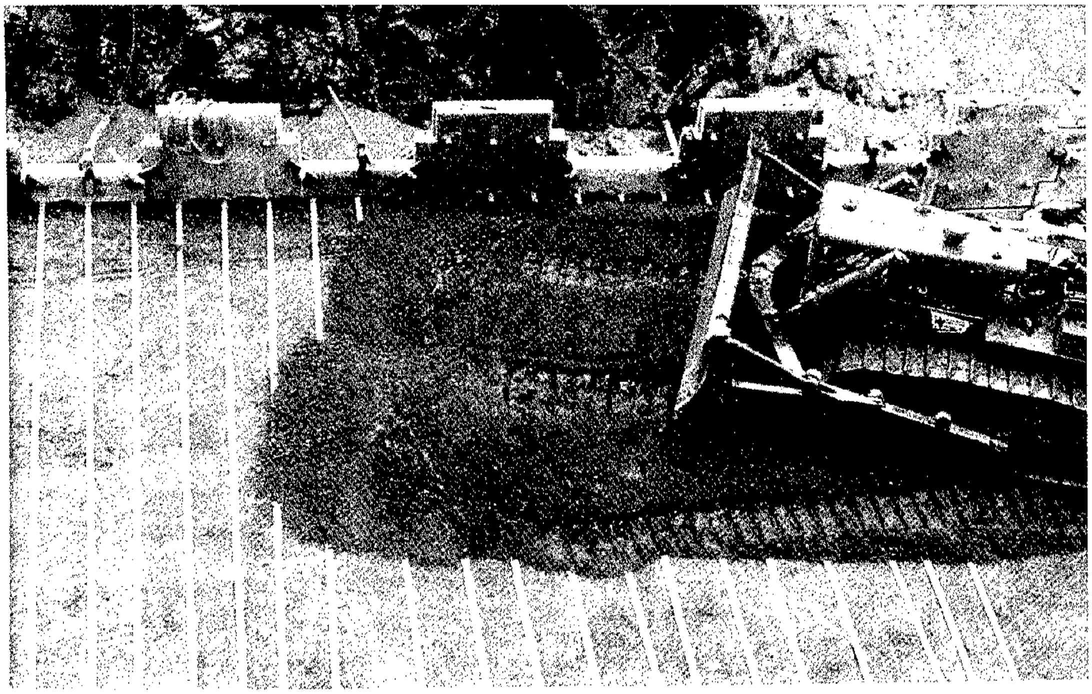


Figure 12. Fill spreading and reinforcement connection.



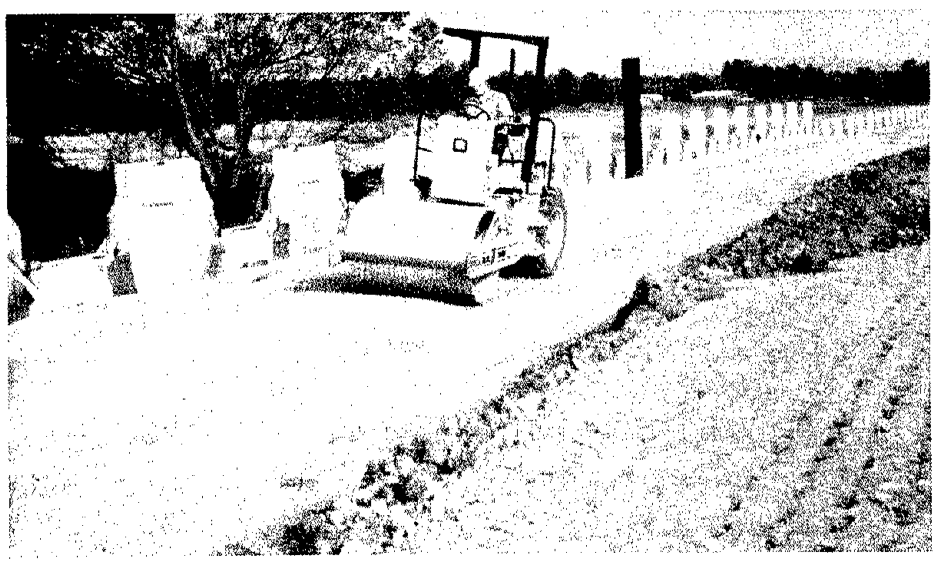


Figure 13. Compaction of backfill.

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 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. It is often convenient to unroll the reinforcement with the roll or machine direction parallel to the face. If this is done, then the cross machine tensile strength must be greater than the design tension requirements.

  Secure reinforcement with retaining pins to prevent movement during reinforced fill placement.

  Overlap adjacent sheets a minimum of 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 14. For temporary support of forms at the face, form holders should be placed at the base of each layer at 1.20 m horizontal intervals. Details of temporary form work are shown in figure 15. These supports are essential for achieving good compaction. When using geogrids or wire mesh, it may be necessary to use a geotextile to retain the backfill material at the wall face.

  When compacting backfill within 1 m of the wall face, a hand-operated vibratory compactor is recommended.

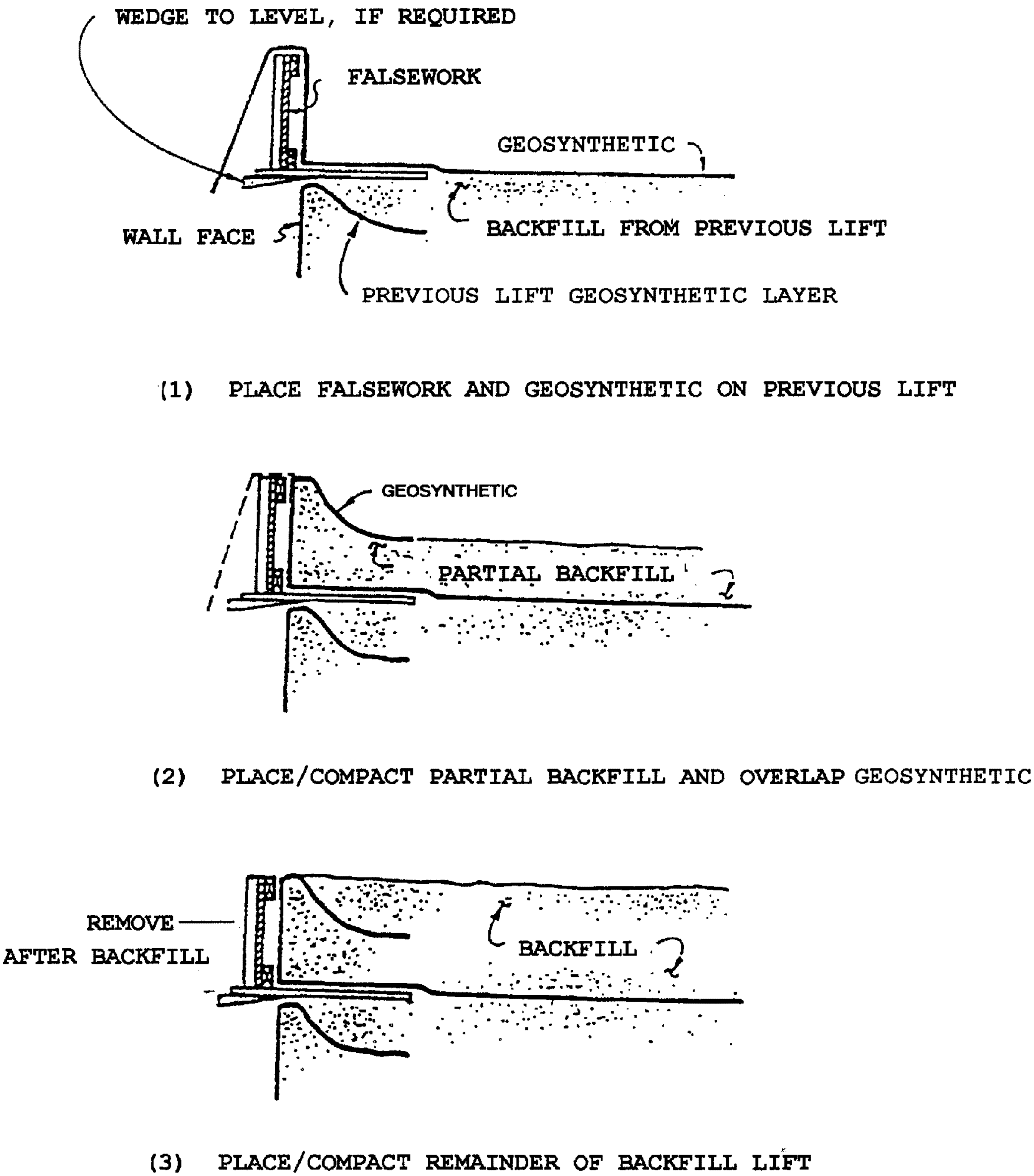
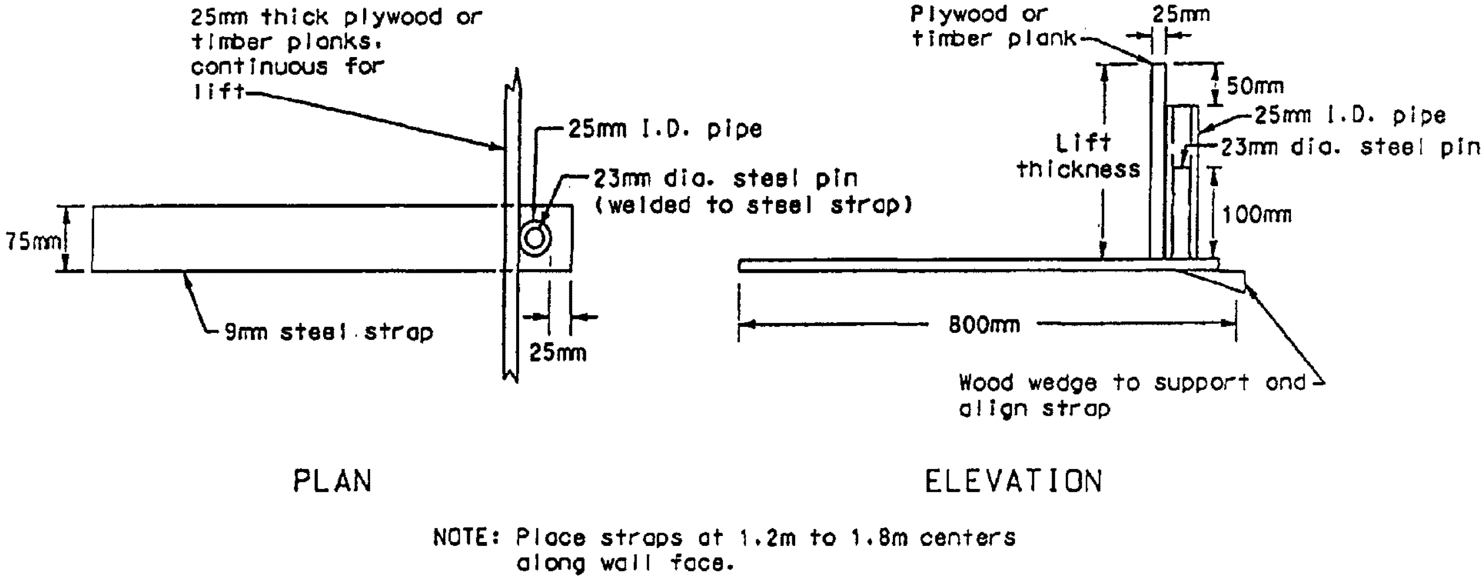


Figure 14. Lift construction sequence for geosynthetic faced MSE walls.



GEOTEXTILE TEMPORARY
FORM SYSTEM DETAIL

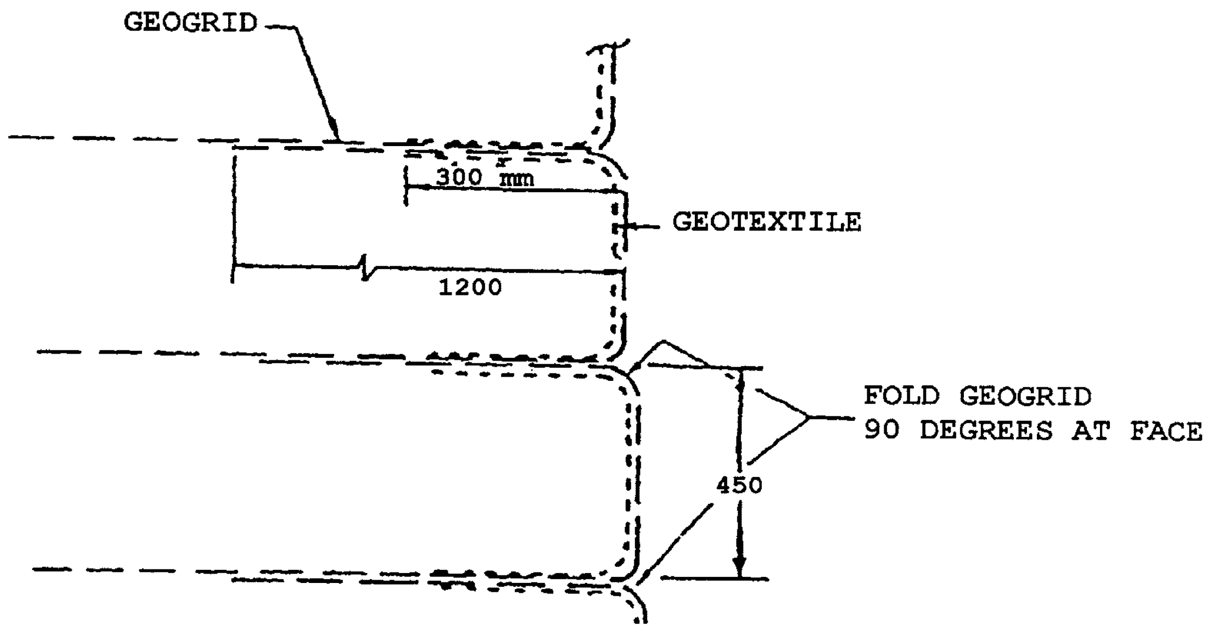


Figure 15. Typical geosynthetic face construction detail.

The return-type method or successive layer tie method as shown in figure 15 can be used for facing support. In the return method, the reinforcement is folded at the face over the backfill material, with a minimum return length of 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.). Figure 16 shows some alternative facing systems for flexible faced walls and slopes.

c. 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.
- Place reinforcement layer.
- Place and compact backfill on reinforcement.
- Construct face. Details of the available methods are outlined in chapter 6, construction.
- Place additional reinforcement and backfill.

Key stages of construction are illustrated in figure 17.

2.9 PROPRIETARY ASPECTS

a. Materials

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

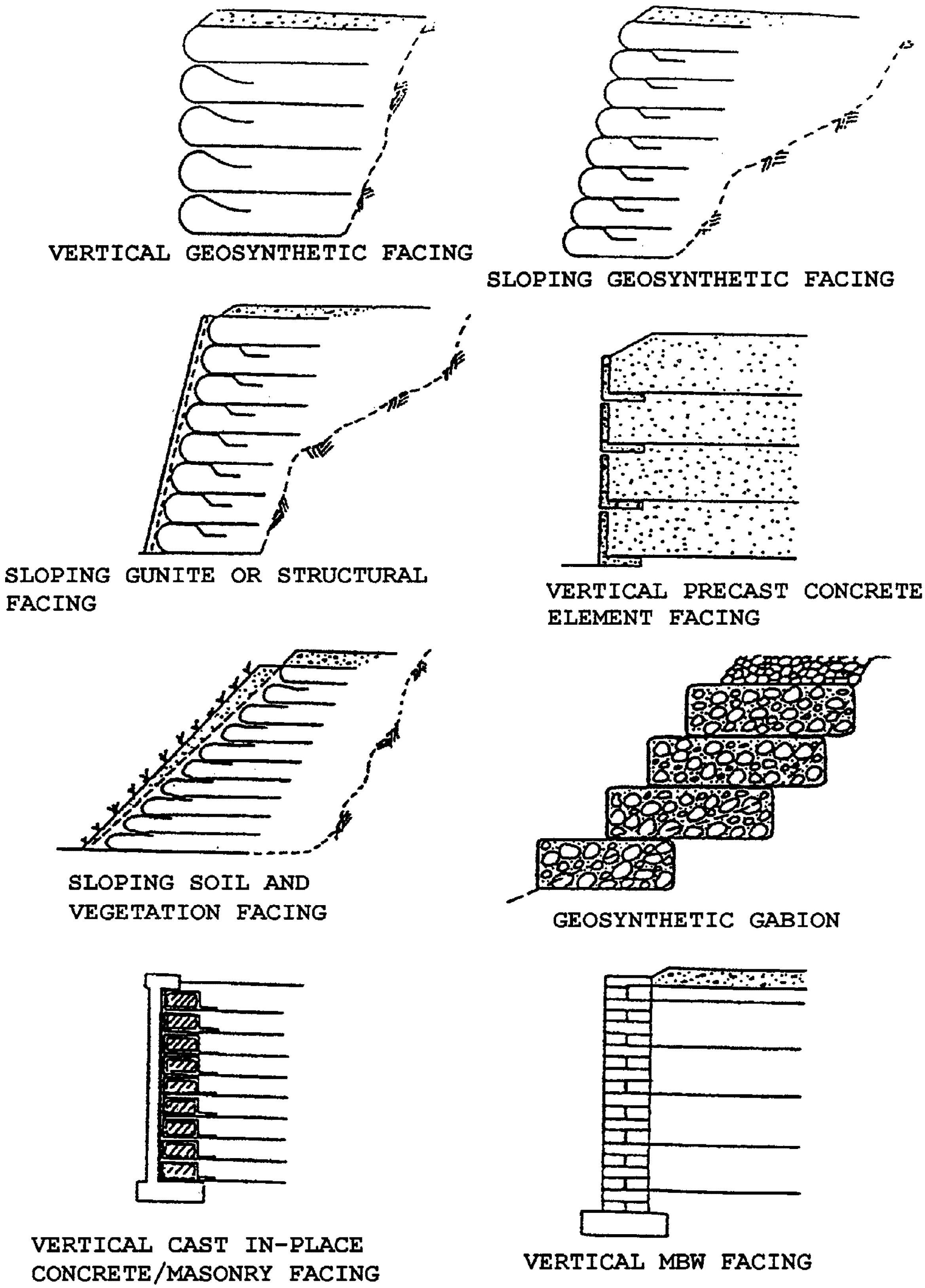
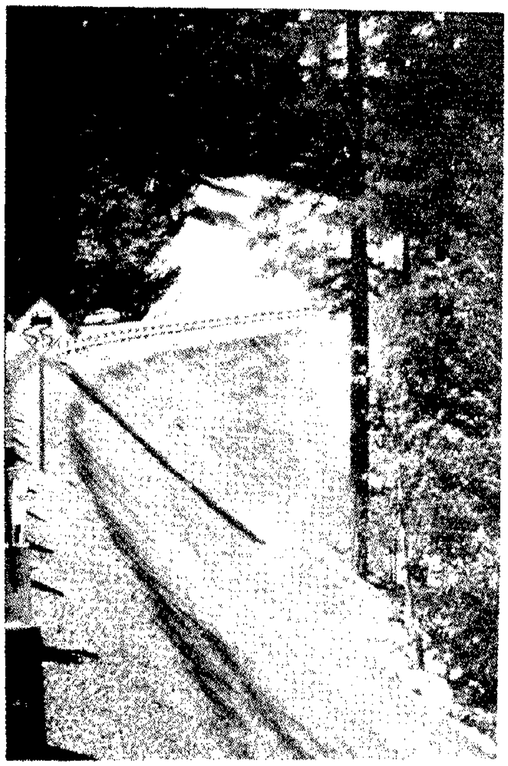


Figure 16. Types of geosynthetic reinforced soil wall facing.

a.



c.

b.

Figure 17. Reinforced slope construction; a) geogrid and fill replacement; b) soil fill erosion control mat facing; and c) finished, vegetated 1:1 slope.

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

- Ribbed planar reinforcing strips due to expire in September 1996 issued to the Reinforced Earth Company.

- 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.

b. Special Applications

A number of patents appear to 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.

CHAPTER 3

SOIL REINFORCEMENT PRINCIPLES AND SYSTEM DESIGN PROPERTIES

This chapter outlines the fundamental soil reinforcement principle that governs structure behavior, and develops system design parameters which are used for specific MSEW and RSS design, detailed in chapters 4, 5 and 7.

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 backfill, retained fill 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 backfill. 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. 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 backfill 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 backfill zone. The retained soil is the source of earth pressures that the reinforced mass must resist.

A drainage system below and behind the reinforced backfill is also an important component especially when using poorly draining backfill.

3.2 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.

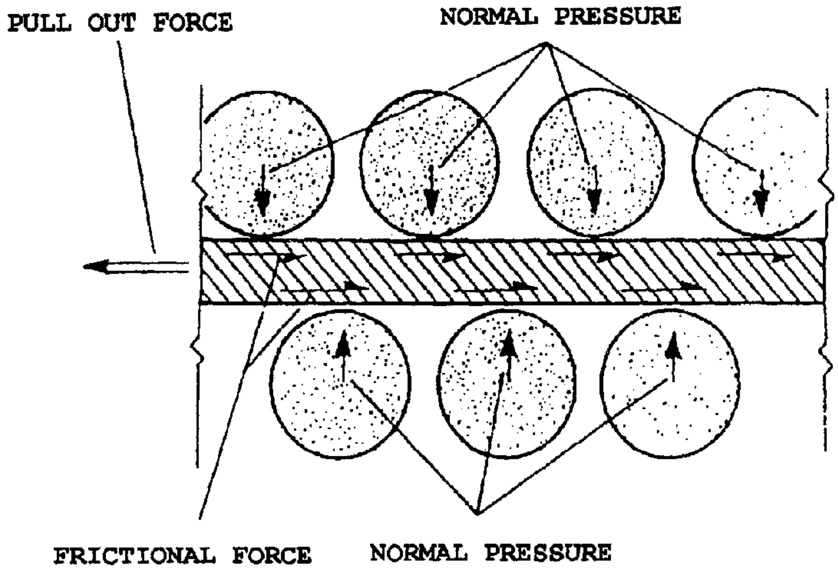
- Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

Stress Transfer Mechanisms

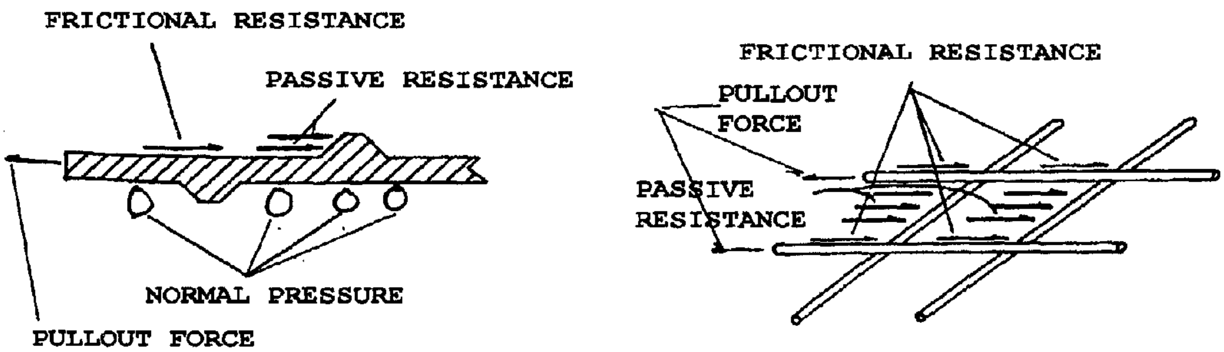
Stresses are transferred between soil and reinforcement by friction (figure 18a) and/or passive resistance (figure 18b) depending on reinforcement geometry:

Friction develops at locations where there is a relative shear displacement and corresponding shear stress between soil and reinforcement surface. Reinforcing elements where friction is important should be aligned with the direction of soil reinforcement relative movement. Examples of such reinforcing elements are steel strips, longitudinal bars in grids, geotextile 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 rigid geogrids, bar mat, and wire mesh reinforcements. The transverse ridges on "ribbed" strip reinforcement also provide some passive resistance.



A) FRICTIONAL STRESS TRANSFER BETWEEN SOIL AND REINFORCEMENT SURFACES.



B) SOIL PASSIVE (BEARING) RESISTANCE ON REINFORCEMENT SURFACES

Figure 18. Stress transfer mechanisms for soil reinforcement.

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.

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 carried by the reinforcement in two ways: in tension or in 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 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 has been recently developed, and is detailed below.

a. Evaluation of Pullout Performance

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

- Pullout capacity, i.e., the pullout resistance of each reinforcement should be adequate to resist the design working tensile force in the reinforcement with a specified factor of safety.

- 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, i.e., interface friction and passive soil resistance against transverse elements of composite 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 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 an issue of the type of reinforcement. Table 4 provides, for generic reinforcement types, the basic aspects of 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.

b. Estimate of the Reinforcement Pullout Capacity in RSS and 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 mass. 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, and it is, therefore, difficult to compare the pullout performance of different reinforcements for a specific application.

Table 4. **Basic aspects of 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	Frictional		
smooth		1.2 mm	Noncreeping
ribbed		12 mm	
Extensible composite plastic strips	Frictional	Dependent on reinforcement extensibility	Dependent on reinforcement structure and polymer creep
Extensible sheets	Frictional	Dependent on reinforcement extensibility (25 to 100 mm)	Dependent on reinforcement structure and polymer creep characteristics
geotextiles			
Inextensible grids			
bar mats	Passive + frictional	12 to 20 mm	Noncreeping
welded wire meshes	Frictional + passive	12 to 20 mm	Noncreeping
Extensible grids			
geogrids	Frictional + passive	Dependent on extensibility (25 to 50 mm)	Dependent on reinforcement structure and polymer creep characteristics
woven meshes	Frictional + passive	25 to 50 mm	Noncreeping

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

$$P_r = F^* \cdot \alpha \cdot \sigma_v' \cdot L_e \cdot C \tag{1}$$

where:

	$L_e \cdot C$	=	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 strips, grids, and sheets
	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)
	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, 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 or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, $\alpha = 0.6$ is recommended.

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 A. 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* = Passive Resistance + Frictional Resistance

$$\text{or, } F^* = F_q \cdot \alpha_\beta + \tan \rho \qquad (2)$$

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 2 are summarized in table 5 and figure 19 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, should be limited to reinforcement/backfill combinations, where this data is sparse or non existent. Where applicable, laboratory pullout tests should be made in a device consisting of a test box with the following dimensions: 760 mm wide, 1210 mm long, and 450 mm deep. The reinforcement samples should be horizontally embedded between two, 150-mm layers of soil. The reinforcement specimen should be pulled horizontally out the front of the box through a split removable door. The test normal load should be applied vertically to the sample by pressurizing an air bag placed between a cover plate and a reaction plate resting on the soil. The pullout movement should be approximately 1.0 mm per minute and monitored using dial gauges mounted to the front of the specimen. Note that this test procedure provides a short-term pullout capacity and does not account for soil or reinforcement creep deformations, which may be of significance in RSS structures utilizing fine grained backfills.

Table 5. Summary of pullout capacity design parameters.

Reinforcement Type	S_{opt}	Grid Spacing	Tan ρ	F_q	α_β	α
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	$t/(2S_t)$	1.0
Extensible grids:						
(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	$\frac{(f_b t)}{(2S_t)}$	0.8
(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:

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 backfill as specified in AASHTO Article 7.3.6.3 of Division II is used. For backfill outside these limits, tests must be run.

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.

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.

NA means "not applicable". ρ is the interface friction angle mobilized along the reinforcement. S_{opt} is the optimum transverse grid element spacing to mobilize maximum pullout resistance. 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. d_{50} is the backfill grain size at 50% passing by weight. α is the scale effect correction factor. Definition of the geometric variables are illustrated in figure 19.

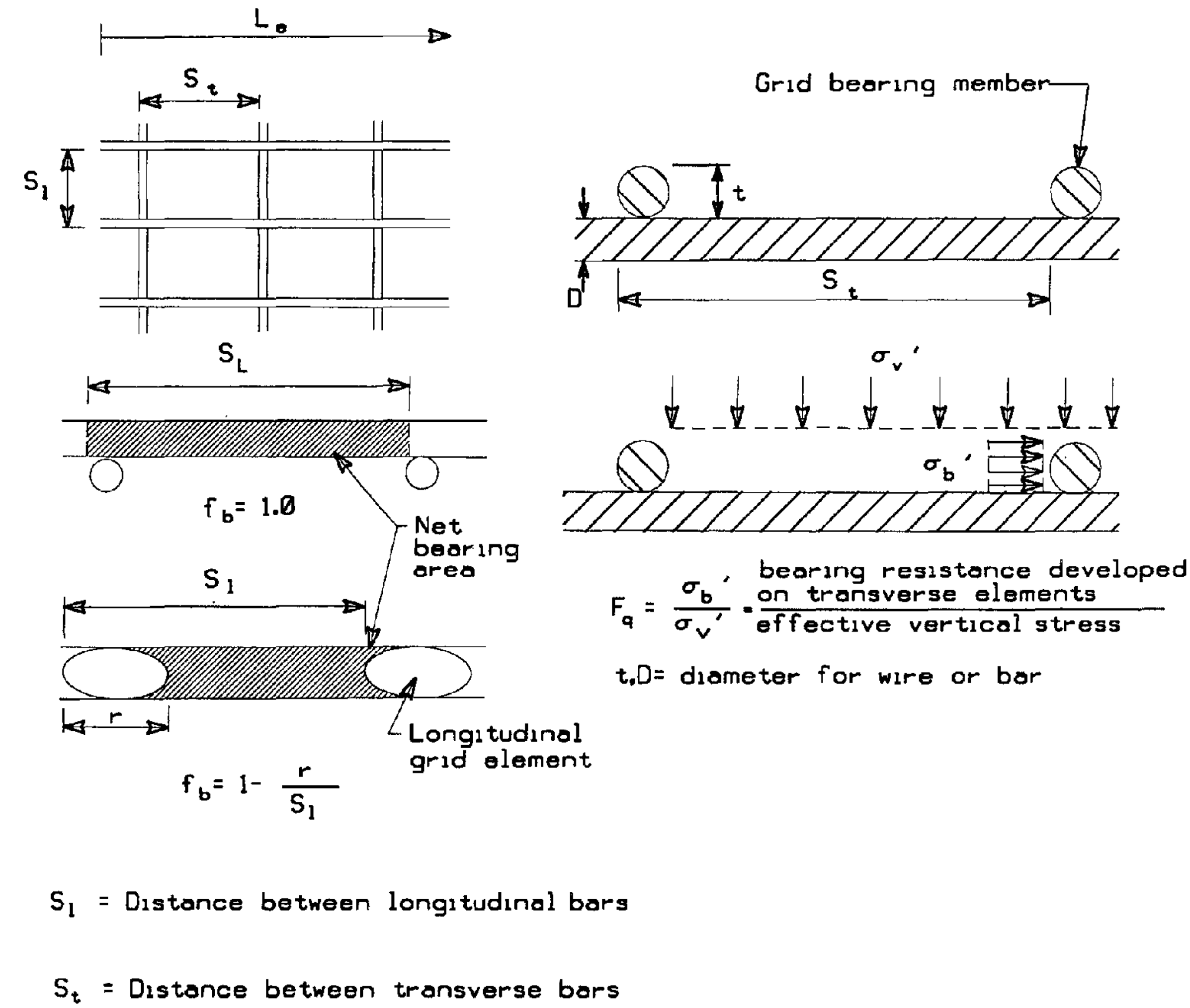


Figure 19. Definition of grid dimensions for calculating pullout capacity.

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 600 mm. The pullout resistance achieved at a maximum deformation of 20 mm as measured at the front of the embedded section for inextensible reinforcements and 15 mm as measured at the end of the embedded sample for extensible reinforcements. This allowable deflection criteria 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 backfill 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.

A summary of the procedures for evaluating laboratory tests to obtain pullout design parameters is outlined in appendix A 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 these semi empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

For steel ribbed reinforcement, the friction factor is commonly taken as:

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

$$F^* = \tan \phi \text{ at a depth of 6 m and below} \quad (4)$$

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 of 4 should be assumed, for backfills meeting the requirements of section 3.4 of this chapter.

For steel grid reinforcements the Pullout Resistance Factor F*, is a function of a bearing or embedment factor (F_q), applied over the contributing bearing α_β, as follows:

$$F^* = F_q\,\alpha_\beta = 40\,\alpha_\beta = 20\,(t/S_t) \text{ at the top of the structure} \quad (5)$$
$$F^* = F_q\,\alpha_\beta = 20\,\alpha_\beta = 10\,(t/S_t) \text{ at a depth of 6 m and below} \quad (6)$$

The grid spacing between transverse grid elements, S_t as shown on figure 19, shall be uniform throughout the length of the reinforcement rather than having transverse grid members concentrated only in the resistant zone.

For geosynthetic sheet reinforcement, the friction F* is commonly taken as:

$$F^* = 2/3 \tan\phi \quad (7)$$

For geogrid reinforcement, the friction factor, often referred as an Interaction Factor (C_i) is commonly taken as:

$$F^* = 0.8 \tan\phi \quad (8)$$

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 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 ESTABLISHMENT OF ENGINEERING PROPERTIES BASED ON SITE EXPLORATION AND TESTING

a. Foundation Soils

Determination of engineering properties for foundation soils should be focused on establishment of bearing capacity, 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 capacity in accordance with Article 4.4.7 for soil and 4.4.8 for rock in 1992 AASHTO. The effects of load inclination and footing shape may be omitted and the minimum Factor of Safety may be taken as 2.5 for Group I loading.

For foundation settlement determinations, the results of conventional settlement analyses using either laboratory time-settlement data, coefficients of consolidation, in conjunction with approximate value for compression index obtained from correlations to soil index tests (moisture content, atterberg limits) may 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 wick drains, stone columns, dynamic compaction, and the use of lightweight fill to reduce settlement. Of particular concern, are situations where the MSEW structure may terminate adjacent to a 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 system suppliers. Evaluations of this type are the responsibility of agency engineers or consultant geotechnical designers.

b. Reinforced Backfill Soil

The selection criteria of reinforced backfill should consider long-term performance of the completed structure, construction phase stability and the degradation environment created for the reinforcements. Much of our knowledge and experience with MSE 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 structures. Many agencies have adopted conservative backfill 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 reinforced backfill property criteria cannot completely replace a reasonable degree of construction control and inspection.**

In general, these select backfill materials will be more expensive than lower quality materials. The specification criteria for each application (walls and slopes) are somewhat different 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 backfills. Detailed project backfill specifications, which **uniformly apply to all MSE systems, should be provided by the contracting agency.**

The following requirements are consistent with current practice:

Select Granular Fill Material for the Reinforced Zone. All backfill material used in the structure volume for MSEW structures shall be reasonably free from organic or other deleterious materials and shall conform to the following gradation limits as determined by AASHTO T-27.

1)

U.S. Sieve Size	Percent Passing
4 in (102 mm)[a]	100
No. 40 (0.425 mm)	0-60
No. 200 (0.075 mm)	0-15

Plasticity Index (PI) shall not exceed 6.

[a]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 19 mm for geosynthetics, and epoxy and PVC coated reinforcements unless tests are or have been performed to evaluate the extent of construction damage anticipated for the specific fill material and reinforcement combination.

2) **Soundness.** The materials shall be substantially free of shale or other soft, poor durability particles. The material shall have a magnesium sulfate soundness loss (or a sodium sulfate value less than 15 percent after five cycles) of less than 30 percent after four cycles. Testing shall be in accordance with AASHTO T-104.

The fill material must be free of organic matter and other deleterious substances, as these materials not only enhance corrosion but also result in excessive settlements. The compaction specifications should include a specified lift thickness and allowable range of moisture content with reference to optimum. The compaction requirements of backfill are different in close proximity to the wall facing (within 1.5 to 2 m). Lighter compaction equipment is 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 backfill material of good quality in terms of both friction and drainage, such as crushed stone is recommended close to the face of the wall to provide adequate strength and tolerable settlement in this zone.

For RSS structures, less select backfill can be used. The following guidelines are provided as recommended backfill requirements for RSS construction:

Sieve Size		Percent Passing
20 mm*		100 - 75
No. 4	(4.76 mm)	100 - 20
No. 40	(0.425 mm)	0 - 60
No. 200	(0.075 mm)	0 - 50

Plasticity Index (PI) ≤ 20 (AASHTO T-90)

Soundness: 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.

* The maximum fill size can be increased (up to 100 mm) provided field tests have been or will be performed to evaluate potential strength reduction due to construction damage. In any case, geosynthetic strength reduction factors for site damage should be checked in relation to the maximum particle size to be used and the angularity of the larger particles.

Backfill compaction should be based on 95% of AASHTO T-99, and ±2% of optimum moisture, w_{opt}.

Fill materials outside of these gradation and plasticity index requirements have been used successfully; however, long-term (> 5 years) performance field data is not available. Performance monitoring is recommended if backfill soils fall outside of the requirements listed above, as detailed in chapter 9.

The reinforced fill criteria outlined above represent materials that have been successfully used throughout the United States and resulted in excellent structure performance. For MSE walls, a lower bound frictional strength of 34 degrees would be consistent with the specified fill, although some nearly uniform fine sands meeting the specifications limits may exhibit friction angles of 31 to 32 degrees. Higher values may be used if substantiated by laboratory direct shear or triaxial test results for the site specific material used or proposed.

For RSS structures, where a considerably greater percentage of fines (Minus #200 sieve) is permitted, lower bound values of frictional strength equal to 28 to 30 degrees would be reasonable. Again, significant economy could be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value.

c. Retained Fill

The key engineering properties required are strength and unit weight based on evaluation and testing of subsurface data. Friction angles (ϕ) and unit weight (γ_T) may be determined from either drained direct shear tests or consolidated drained triaxial tests. 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. In addition, the position of groundwater levels above the proposed base of construction must be determined in order to plan an appropriate drainage scheme. For most retained fills lower bound frictional strength values of 28 to 30 degrees are reasonable for granular and low plasticity cohesive soils. For highly plastic retained fills (PI > 40), even lower values would be indicated and should be evaluated for both drained and undrained conditions.

d. Electrochemical Properties

The design of buried steel elements of MSE structures is predicated on backfills 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 limits are shown in table 6 as follows:

Table 6. Recommended electrochemical properties for backfills when using steel reinforcement.

Property	Criteria	Test Method
Resistivity	>3000 ohm-cm	AASHTO T-288-91
pH	>5<10	AASHTO T-289-91
Chlorides	100 PPM	AASHTO T-291-91
Sulfates	200 PPM	AASHTO T-290-91
Organic Content	1% max.	AASHTO T-267-86

Reinforced fill soils must meet the indicated criteria to be qualified for use in MSE construction using steel reinforcements.

Where geosynthetic reinforcements are planned, the limits for electrochemical criteria would vary depending on the polymer. Tentative limits, based on current research are shown in table 7.

Table 7. Recommended electrochemical properties for backfills when using geosynthetic reinforcements.

Base Polymer	Property	Criteria	Test Method
Polyester (PET)	pH	>3<9	AASHTO T-289-91
Polyolefin (PP & HDPE)	pH	>3	AASHTO T-289-91

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:

a. 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). 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.

- **Geotextiles and geogrids**. A layer of geotextile or geogrid is characterized by the width of the geosynthetic and the center-to-center horizontal distance between elements. The cross-sectional area is not needed since the strength is expressed by a tensile force per unit width rather than by 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.

$$R_c = b/S_h \tag{9}$$

where: b = the gross width of the strip, sheet or grid; and
S_h = center-to-center horizontal spacing between strips, sheets, or grids

(R_c = 1 in the case of continuous reinforcement, i.e., each reinforcement layer covers the entire horizontal surface of the reinforced soil mass.)

b. Strength Properties

Steel Reinforcement

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:

$$E_c = E_n - E_R$$

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.

Therefore, the allowable tensile force per unit width of reinforcement, T_a, is obtained as follows:

$$T_a = FS \frac{A_c F_y}{b} \tag{11}$$

where:	FS	=	0.55 for strips and 0.48 for grids with rigid facing elements (Note: 0.55 F may be used for grids with flexible facings)
	b	=	the gross width of the strip, sheet or grid
	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 allowable tensile stress for steel reinforcements and connections for permanent structures is developed in accordance with Article 10.32, in particular Table 10.32.A of AASHTO. These requirements result in an allowable tensile stress for steel strip reinforcement, in the wall backfill away from the wall face connections, of 0.55 F_y. For grid reinforcing members connected to a rigid facing element (e.g., a concrete panel or block), the allowable tensile stress is reduced to 0.48 F_y. Transverse and longitudinal grid members are sized in accordance with ASTM A-185. For temporary structures (i.e., design lives of 3 years or less), the allowable tensile stress may be increased by 40 percent. The global safety factor of 0.55 applied to F_y for permanent structures accounts for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonuniformities, and uncertainties in long-term reinforcement strength. Safety factors less than 0.55, such as the 0.48 factor applied to grid members with rigid concrete facing, to account for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars.

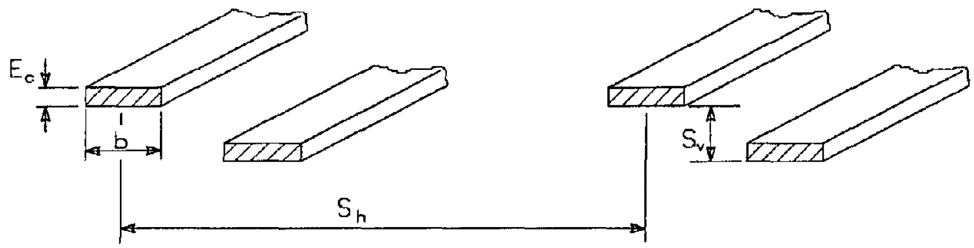
The quantities needed for determination of A_c for steel strips and grids are shown in figure 20. The use of hardened and otherwise low strain steels may increase the potential for catastrophic failure, therefore, a lower allowable material stress may be warranted with such materials.

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 its effectiveness. When PVC or epoxy coatings are used, the maximum particle size of the backfill should be restricted to 19 mm or less to reduce the potential for construction damage. For a more detailed discussion of requirements, refer to the Corrosion/Degradation document.

Several State transportation departments have used resin-bonded epoxy coated steel reinforcing elements. The effectiveness of these coatings in MSEW structures has not been sufficiently demonstrated and their widespread use cannot be presently endorsed. If used a minimum coating thickness of 0.41 mm (16 mils) is recommended applied in accordance with ASTM A-884 for grid reinforcement and AASHTO M-284 for strip reinforcement. 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.

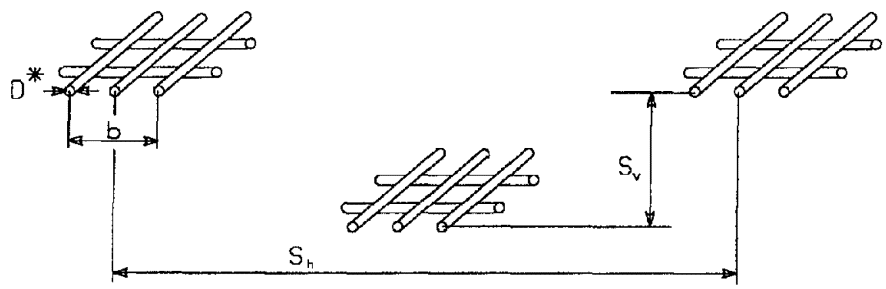
Extensive studies have been made to determine the rate of corrosion of galvanized mild steel bars or strips buried in different types of soils commonly used in reinforced soil. Based on these studies, deterioration of steel strips, mesh, bars and mats can be estimated and accounted for by using increased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

The majority of MSE walls constructed to date have used galvanized steel and backfill materials with low corrosive potential. 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. After the zinc is oxidized (consumed), corrosion of the base metal starts.



$A_c = b\,E_c$

E_c = strip thickness corrected for corrosion loss.



A_c = (No. of longitudinal bars) $\cdot\ \pi \dfrac{D^{*2}}{4}$

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

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

$$T_{max} \leq T_a R_c = \frac{FS\ A_c\ F_y\ R_c}{b}$$

Where T_a = allowable long-term tensile strength of reinforcement (strength/unit reinforcement width)

FS = factor of safety

F_y = yield strength of steel

R_c = reinforcement coverage ratio = $\dfrac{b}{S_h}$

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

T_{max} = maximum load applied to reinforcement (load/unit wall width).

Figure 20. Parameters for metal reinforcement strength calculations.

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

Corrosion Rates - mildly corrosive backfill

For zinc
15 μm/year (first 2 years)
4 μm/year (thereafter)

For residual carbon steel
12 μm/year (thereafter)

The designer of an MSE structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of the United States, it can be expected that deicing salts might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 2.5 m of the reinforced backfill (as measured from the roadway surface) 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.

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.)

- Structures exposed to stray currents, such as from nearby underground power lines, and structures supporting or located adjacent to electrical railways.

- The use of metal reinforcing elements that are not galvanized with at least 610 g/m^2 coating in accordance with AASHTO M-111 for strip reinforcement and ASTM A-641 for grid reinforcement.

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

Geosynthetic Reinforcement

Selection of T_a for geosynthetic reinforcement is more complex than for steel. The tensile properties of geosynthetics are affected by environmental 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, and the details of polymer behavior for in-ground use are not completely understood.

Ideally, T_a should be determined by thorough consideration of allowable elongation, creep potential and all possible strength degradation mechanisms.

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. Temperatures can be as high as 50° C compared with the normal range of in-ground temperature of 12° C in cold and temperate climates to 30° C in arid desert climates.

Degradation most commonly occurs from mechanical damage, long-term time dependent degradation caused by stress (creep), deterioration from exposure to ultraviolet light, and chemical or biological interaction with the surrounding environment.

Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

Typically, polyester products (PET) are susceptible to aging strength reductions due to hydrolysis (water availability) and high temperatures. Hydrolysis and fiber dissolution are accelerated in alkaline regimes, below or near piezometric water levels or in areas of substantial rainfall where surface water percolation or capillary action ensures water availability over most of the year.

Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen) and or high temperatures. The level of oxygen in reinforced fills is a function of soil porosity, ground water location and other factors not yet fully understood. However, it is considerably less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in the ground should proceed at a slower rate than those used above ground. Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the backfill as found in acid sulphate soils, slag fills, other industrial wastes or mine tailings containing transition metals. 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 artificial relative resistance of polymers to these identified regimes is shown in table 8.

Table 8. Anticipated resistance of polymers to specific environments

Soil Environment	Polymer		
	PET	PE	PP
Acid Sulphate Soils	NE	?	?
Organic Soils	NE	NE	NE
Saline Soils pH < 9	NE	NE	NE
Calcareous Soils	?	NE	NE
Modified Soils/Lime, Cement	?	NE	NE
Sodic Soils, ph > 9	?	NE	NE
Soils with Transition Metals	NE	?	?

NE = No Effect
? = Questionable Use, Exposure Tests Required

Polymeric reinforcements may therefore be chosen consistent with the preliminary data shown in table 8.

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. 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.

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

Damage during backfilling operations is a function of the severity of loading imposed on the geosynthetic during construction operations and the size and angularity of the backfill. For MSEW and RSS construction, light weight, low strength geotextiles should be avoided to minimize damage with ensuing loss of strength.

For geosynthetic reinforcements, the design life is achieved by developing an allowable design load which considers all time dependent strength losses over the design life period as follows:

$$T_a = \frac{T_{ULT}}{RF \cdot FS} \tag{12}$$

where T_a is the design long term reinforcement tension load for the limit state, T_{ult} the ultimate geosynthetic tensile strength and RF is the product of all applicable reduction factors.

The geosynthetic material strength more specifically is:

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \cdot RF_D \cdot RF_{ID}} \tag{13}$$

where:

T_{al} = Long-term tensile strength on a load per unit width of reinforcing basis.

T_{ULT} = Ultimate (or yield tensile strength) from wide strip tensile strength tests (ASTM D 4595 or GR1:GG1 for geogrids), based on minimum average roll value (MARV) for the product.

RF_{CR} = Creep Reduction Factor is the ratio of the ultimate strength (T_{ULT}) to the creep limit strength obtained from laboratory creep tests for each product. Typical ranges of reduction factors as a function of polymer type, are indicated below:

Polymer Type	Creep Reduction Factors
Polyester	2.5 to 2.0
Polypropylene	5 to 4.0
Polyethylene	5 to 2.5

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

RF_{ID} = Installation Damage reduction factor. It can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight.

FS = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used, with one notable exception. The 1994 Interims to the AASHTO Specifications for Highway Structures (1992), states that a minimum factor of safety of 1.78 is to be used.

For RSS structures, it is taken as 1.0, as the required factor of safety, is accounted in the stability analysis.

The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, but RF should not be less than 3.0 for permanent structures, briefly summarized as follows:

Creep Reduction Factor, RF_{CR}.

This reduction factor is obtained from long term laboratory creep testing as detailed in appendix B. 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. The creep reduction factor is the ratio of the ultimate load to the maximum sustainable load within the design life.

Durability Reduction Factor, RF_D.

The protocol for testing to obtain this reduction factor is under development. In general, it consists 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 pH's 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 document.

Installation Damage Reduction Factor, RF_{ID}.

A protocol for field testing for this reduction factor is detailed in the companion Corrosion/Degradation document and in ASTM D-5818. The protocol requires that the geosynthetic material is subjected to a backfilling 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 270 g/m^2 for geotextiles is recommended to minimize installation damage. This roughly corresponds to a Class 1 geotextile as specified in AASHTO M-288-96. For more detailed explanations, see the companion Corrosion/Degradation document.

Factor of Safety, FS.

This is a global factor of safety which accounts for uncertainties in externally applied loads, structure geometry, fill properties, potential for local overstress due to load nonuniformity and uncertainties in long-term reinforcement strength. For limit state conditions, a F.S. of 1.5 has been traditionally used. This is lower than the implied current F.S. of 1.82 (1/0.55 F_y) for steel reinforcements due to the ductile nature of geosynthetics systems versus the brittle nature of steel systems at failure.

The recommended F.S. of 1.5 can be further justified by considering the following:

- For geosynthetic reinforcements, the backfill soil controls the amount of strain in the reinforcement which for granular backfills 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 allowable strength.

- Measurement of stress levels in structures, has consistently indicated lower stress levels than used for design as developed in chapter 4.

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 allowable tensile strength T_a, may be evaluated without product specific data, as:

$$T_a = \frac{T_{ULT}}{7 \cdot FS} \qquad (14)$$

Further, this reduction factor RF = 7, should be limited 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 < 30° C

- Maximum backfill particle size of 19 mm.

- Maximum MSEW height is 10 m and

- Maximum RSS height is 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.

The total reduction factor of 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 9.

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 4 to 6.

For temporary applications not having severe consequences should poor performance or failure occur, a default value for RF of less than 3 could be considered.

Table 9. Minimum requirements for use of default reduction factors for primary geosynthetic reinforcement.

Type	Property	Test Method	Criteria to Allow Use of Default RF
Polypropylene	UV Oxidation Resistance	ASTM D-4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyethylene	UV Oxidation Resistance	ASTM D-4355	Min. 70% strength retained after 500 hrs. in weatherometer
Polyester	Hydrolysis Resistance	Intrinsic Viscosity Method (ASTM D-4603) with Correlation or Determine Directly Using Gel Permeation Chromatography	Min. Number (Mn) Molecular Weight of 25,000
Polyester	Hydrolysis Resistance	ASTM D-2455	Max. Carboxyl End Group Number of 30
All Polymers	Survivability	Weight per Unit Area, ASTM D-5261	Min. 270 g/m^2
All Polymers	% Post Consumer Recycled Material by Weight	Certification of Material used	Maximum 0%

The 1994 Interim AASHTO specification recommends that the allowable tensile strength (T_{al}) be the lesser of the Limit State condition (T_a) outlined above, or from a Serviceability State determination which limits the total strain on the structure to 5 percent for MSE walls and 10 percent for RSS structures. Methods for this determination have varied widely with no present consensus on an appropriate method capable of modeling the strains in the structure. *Therefore, until an appropriate method of determination is agreed upon, it is recommended this requirement be dropped.*

CHAPTER 4

DESIGN OF MSE WALLS

This chapter details general and simplified design guidelines common to all MSEW systems. It is limited to MSE walls having a near-vertical face, and uniform length reinforcements. Design guidelines for complex structures, or structures with unusual features are covered in chapter 5.

This chapter is organized sequentially as follows:

- Overview of design methods.
- Sizing for external stability.
- Sizing for internal stability.
- Design details.
- Design example.

4.1 DESIGN METHODS

Since the development of soil reinforcement concepts and their application to MSEW structure design, a number of design methods have been proposed, used, and refined. Current practice consists of determining the geometric and reinforcement requirements to prevent internal and external failure using limit equilibrium methods of analysis.

External stability evaluations for MSEW structures treat the reinforced section as a composite homogeneous soil mass and evaluate the stability according to conventional failure modes for gravity type wall systems. Differences in the present practice exist for internal stability evaluations which determines the reinforcement required, principally in the development of the internal lateral stress and the assumption as to the location of the most critical failure surface.

Internal stability is treated as a response of discrete elements in a soil mass. This suggests that deformations are controlled by the reinforcements rather than total mass, which appears inconsistent given the much greater volume of soil in such structures. Therefore, deformation analyses are generally not included in current methods.

Given the availability of different methods and research in the last decade, general agreement has been reached that a complete design approach should consist of the following:

- Working Stress analyses.
- Limit Equilibrium analyses.
- Deformation Evaluations.

a. Analysis of Working Stresses for MSEW structures

An analysis of working stresses consists of:

- Selection of reinforcement location and a check that stresses in the stabilized soil mass are compatible with the properties of the soil and inclusions.
- Evaluation of local stability at the level of each reinforcement and prediction of progressive failure.

b. Limit Equilibrium Analysis

A limit equilibrium analysis consists of a check of the overall stability of the structure. The types of stability that must be considered are external, internal, and combined:

- External stability involves the overall stability of the stabilized soil mass considered as a whole and is evaluated using slip surfaces outside the stabilized soil mass.
- Internal stability analysis consists of evaluating potential slip surfaces within the reinforced soil mass.

In some cases, the critical slip surface is partially outside and partially inside the stabilized soil mass, and a combined external/internal stability analysis may be required.

c. Deformation Evaluations

A deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to horizontal and vertical displacement. In addition, the influence and variations in the type of reinforcement on the performance of the structure can be evaluated. Horizontal deformation analyses are the most difficult and least certain of the performed analyses. In many cases, they are done only approximately or it is simply assumed that the usual factors of safety against external or internal stability failure will ensure that deformations will be within tolerable limits. Vertical deformation analyses are obtained from conventional settlement computations, with particular emphasis on differential settlements, longitudinally along the wall face, and transversely from the face to the end of the reinforced soil volume. The results may impact the choice of facing, facing connections or backfilling sequences.

d. Design Methods, Inextensible Reinforcements

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

The state of stress for **external stability**, is assumed to be equivalent to a Coulomb state of stress with a wall friction angle δ equal to zero. For **internal stability** a variable state of stress varying from a multiple of K_a to an active earth pressure state, K_a are used for design. Recent 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 from the surface. The results from these efforts have been synthesized in a *simplified coherent gravity method*, which will be used throughout this manual.

e. Design Methods, Extensible 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 coherent gravity 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.

4.2 SIZING FOR 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 21. They include:

- Sliding on the base.
- Limiting the location of the resultant of all forces (overturning).
- Bearing capacity .
- Deep seated stability (rotational slip-surface or slip along a plane of weakness).

Due to the flexibility and satisfactory field performance of MSE walls, the adopted values for the factors of safety for external failure are in some cases lower than those used for reinforced concrete cantilever or gravity walls. For example, the factor of safety for overall bearing capacity is 2.5 rather than a higher value, which is used for more rigid structures.

Likewise, the flexibility of MSE walls should make the potential for overturning failure highly unlikely. However, overturning criteria (maximum permissible eccentricity) aid in controlling lateral deformation by limiting tilting and, as such, should always be satisfied.

External stability computational sequences are schematically illustrated as follows:

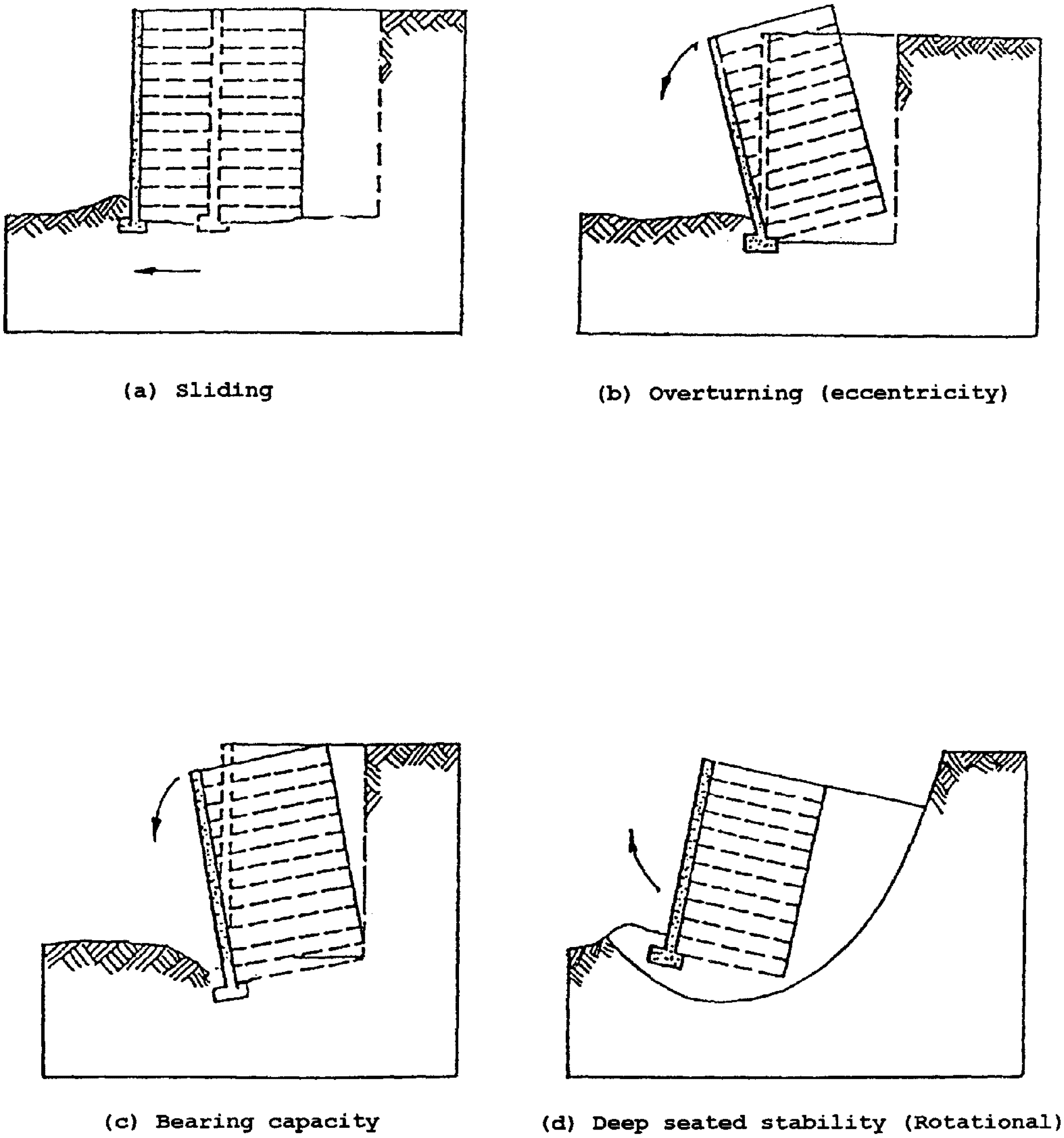
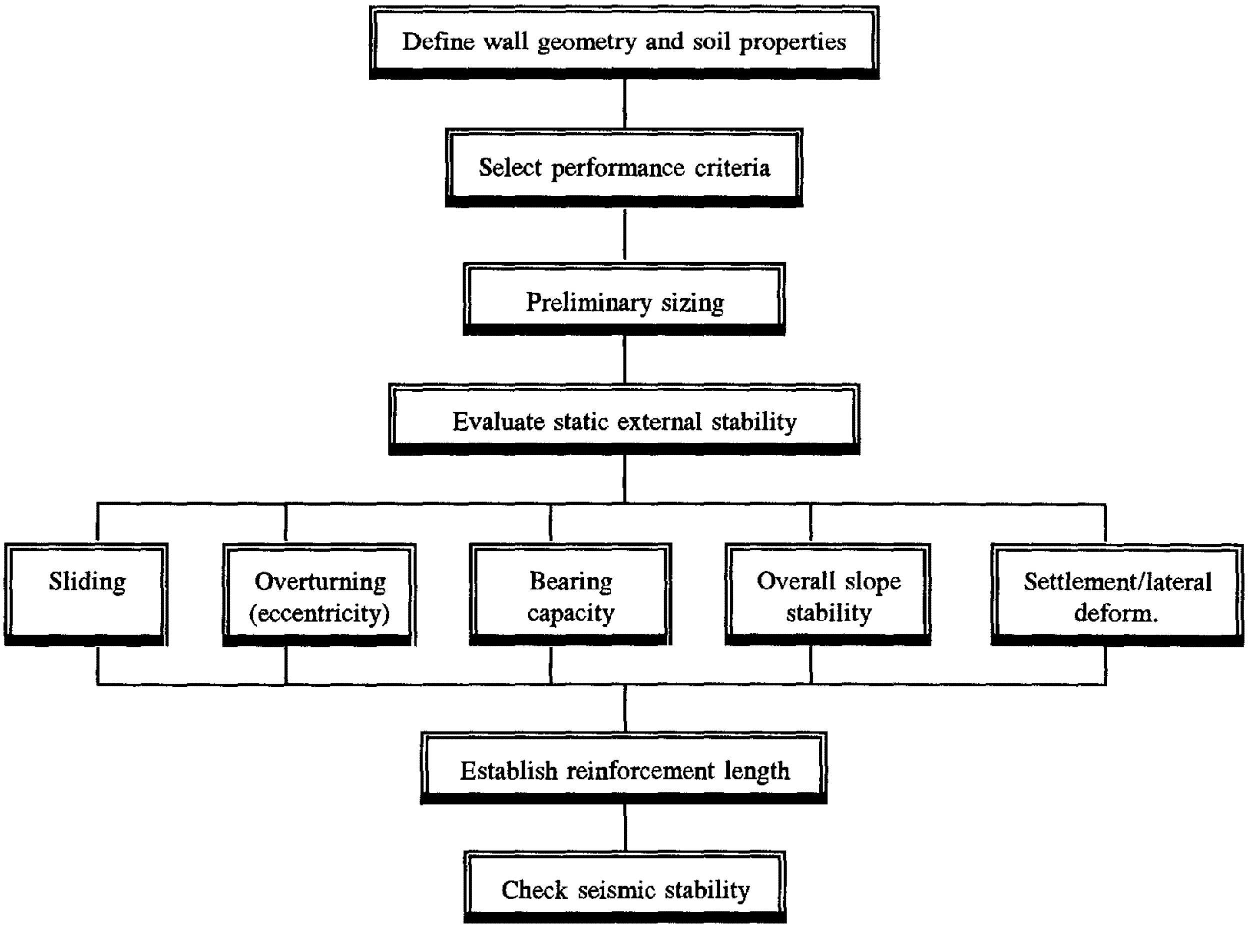


Figure 21. Potential external failure mechanisms for a MSE wall.



Each of the sequential steps are discussed as follows:

a. Define wall geometry and soil properties

The following must be defined or established by the designer:

- Wall height, batter.
- Soil surcharges, live load surcharges, dead load surcharges, etc.
- Seismic loads.
- Engineering properties of foundation soils (γ, c, ϕ).
- Engineering properties of the reinforced soil volume (γ, c, ϕ).

- Engineering properties of the retained fill (γ, c, ϕ).

- Groundwater conditions.

b. Select performance criteria

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

- External stability factors of safety (Sliding, bearing capacity location of resultant force).

- Global stability factor of safety.

- Maximum differential settlement.

- Maximum horizontal displacement.

- Seismic stability factor of safety.

- Design life.

c. Preliminary Sizing

The process of sizing the structure begins by adding the required embedment, established under Project Criteria (Section 2.7.c), to the wall height in order to determine the design heights for each section to be investigated. Since the structure is constructed from the bottom up, this condition may prevail at least to the end of construction.

A preliminary length of reinforcement is chosen that should be greater of 0.7H and 2.5 m, where H is the design height of the structure. Structures with sloping surcharge fills or other concentrated loads, as in abutment fills, generally require longer reinforcements for stability, often on the order of 0.8H to as much as 1.1H. Special structures with lesser reinforcement lengths at the base are covered in chapter 5.

d. Earth Pressures for External Stability

Stability computations for walls with a vertical face are made by assuming that the MSE wall mass acts as a rigid body with earth pressures developed on a vertical pressure plane arising from the back end of the reinforcements, as shown in figures 22 to 24.

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

$$K_a = \tan^2 (45 - \frac{\phi}{2}) \tag{15}$$

for vertical wall with a surcharge slope from:

$$K_a = \cos \beta \left[\frac{\cos \beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos \beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right] \tag{16}$$

where β = surcharge slope angle.

For broken back surcharge conditions, the angle I (see figure 24) is substituted for the infinite surcharge slope angle β.

For an inclined front face greater than 10 degrees, the coefficient of earth pressure can be calculated from the general Coulomb case as:

$$K_a = \frac{\sin^2 (\theta + \phi)}{\sin^2\theta \, \sin(\theta - \delta) \left[1 + \sqrt{\frac{\sin(\phi + \delta) \, \sin(\phi - \beta)}{\sin(\theta - \delta) \, \sin(\theta + \beta)}} \right]^2} \tag{17}$$

where θ is the face inclination from a horizontal, and β the surcharge slope angle. The wall friction angle δ is assumed to be equal to β.

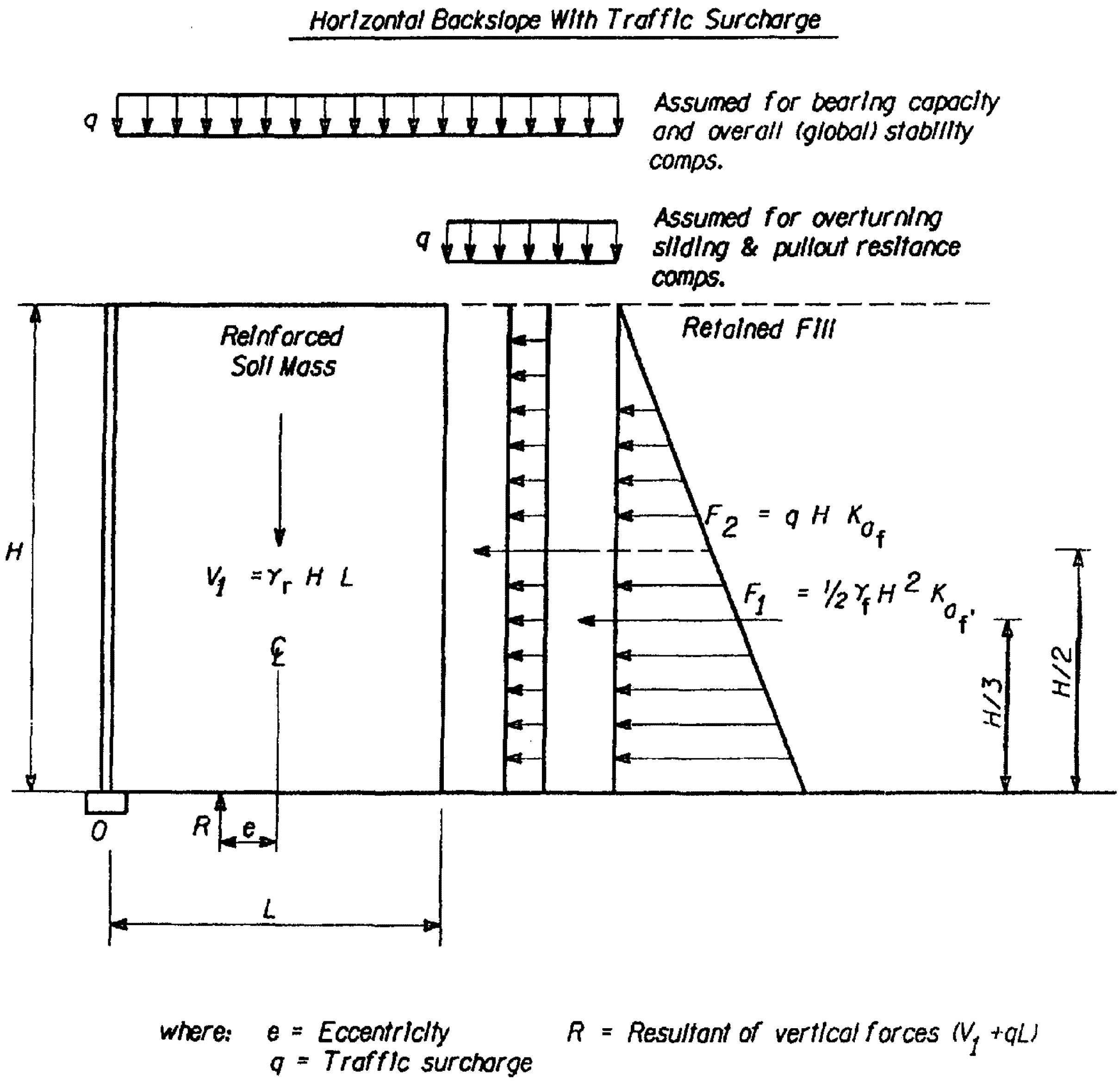
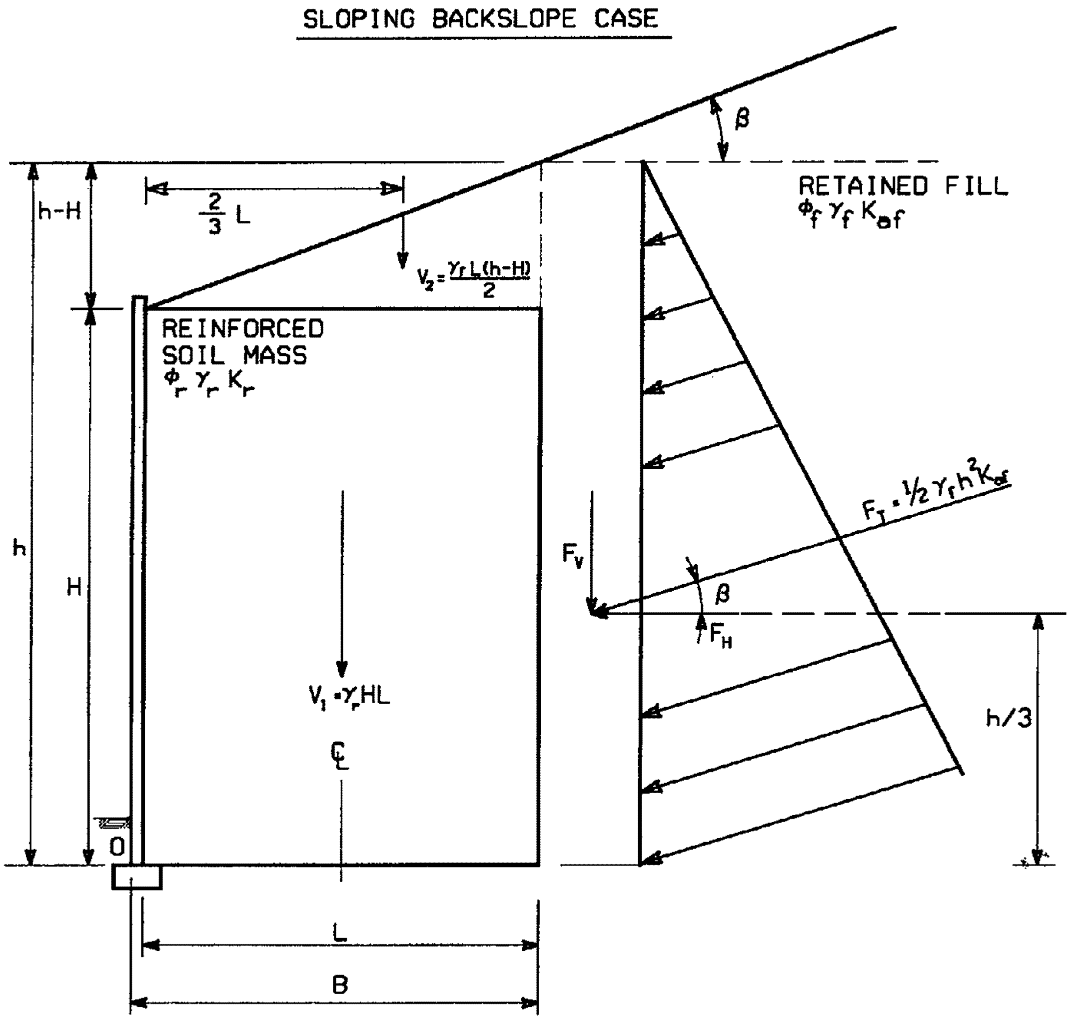
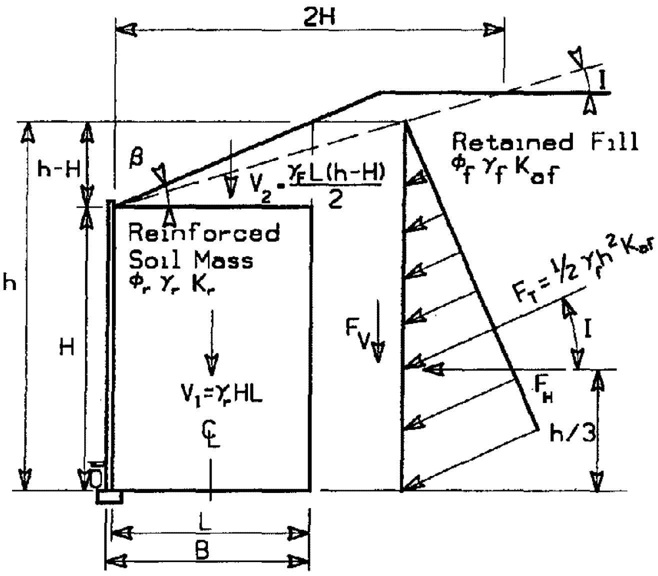


Figure 22. External analysis: earth pressures/eccentricity. Horizontal backslope with traffic surcharge.



Note: For relatively thick facing elements (e.g., segmental concrete facing blocks) it may be desireable to include the facing dimensions and weight in sliding and overturning calculations (i.e. use "B" in lieu of "L").

Figure 23. External analysis: earth pressure/eccentricity. Sloping backfill case.



$F_H = F_T \cos(I)$

$F_V = F_T \sin(I)$

FOR INFINITE SLOPE $I = \beta$

K_a For Retained Fill Using $\delta = \beta = I$

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

Note: For relatively thick facing elements (e.g., segmental concrete facing blocks) it may be desireable to include the facing dimensions and weight in sliding and overturning calculations (i.e. use "B" in lieu of "L").

Figure 24. External analysis: earth pressure/eccentricity. Broken backslope case.

Vertical Pressure Computations

Computations for vertical stresses at the base of the wall defined by the height h are shown on figure 25. It should be noted that the weight of any wall facing is typically neglected in the calculations. Calculation steps for the determination of a vertical bearing stress are:

(1) Calculate $F_T = \frac{1}{2} K_{af\,(\phi,\,\beta)}\, \gamma_f\, h^2$ (18)

(2) Calculate eccentricity, e, of the resulting force on the base by summing the moments of the mass of the reinforced soil section about the center line of mass. Noting that R in figure 25 must equal the sum of the vertical forces on the reinforced fill, this condition yields:

$$e = \frac{F_T\,(\cos\beta)\; h/3 - F_T\,(\sin\beta)\; L/2 - V_2\;(L/6)}{V_1 + V_2 + F_T\,\sin\beta} \tag{19}$$

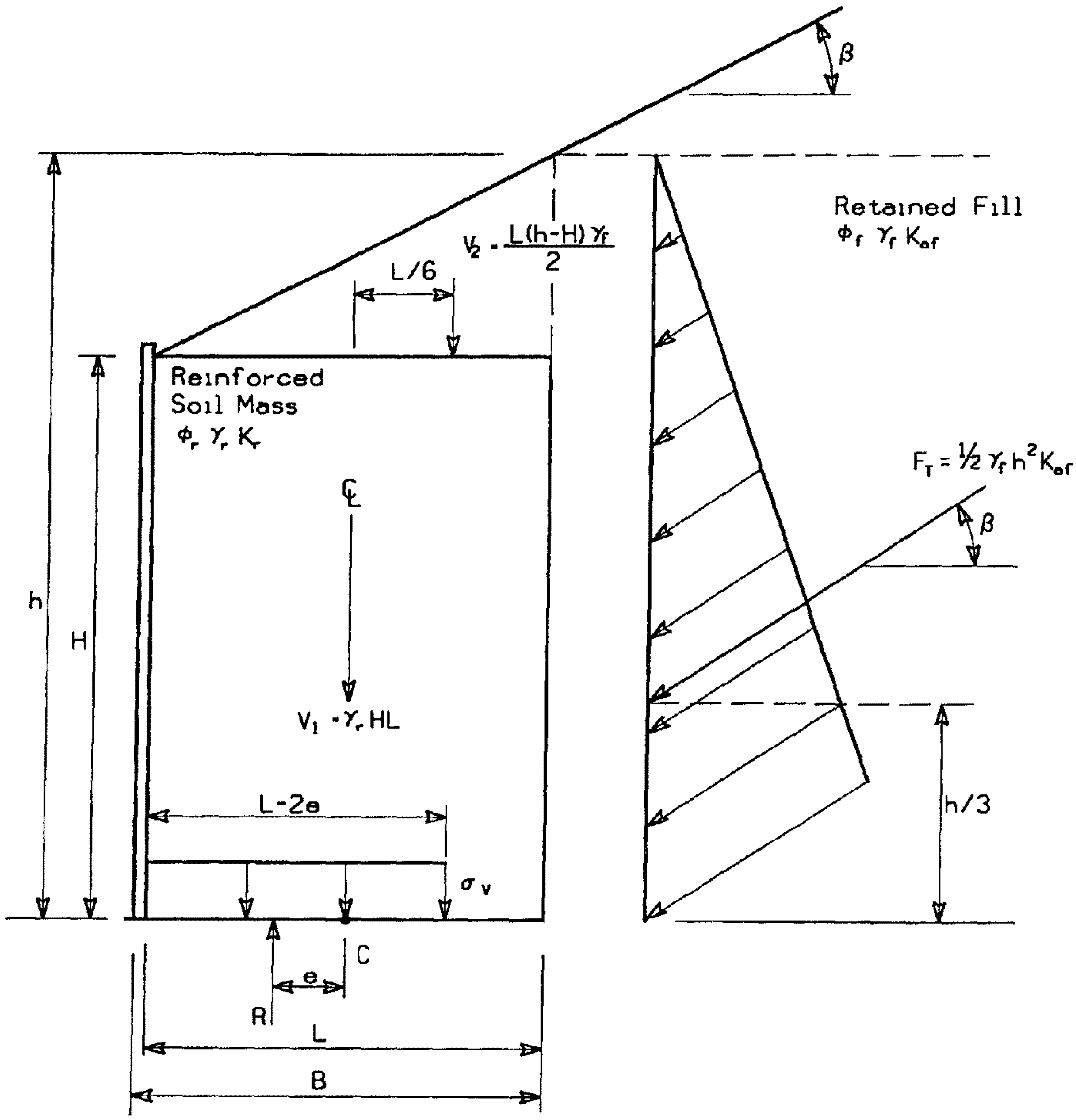
(3) e must be less than L/6 in soil or L/4 in rock. If e is greater, than a longer length of reinforcement is required.

(4) Calculate the equivalent uniform vertical stress on the base, σ_v:

$$\sigma_v = \frac{V_1 + V_2 + F_T \sin\beta}{L - 2e} \tag{20}$$

This approach, proposed originally by Meyerhof, assumes that eccentric loading results in a uniform redistribution of pressure over a reduced area at the base of the wall. This area is defined by a width equal to the wall width less twice the eccentricity as shown in figure 25.

(5) Add the influence of surcharge and concentrated loads to σ_v, where applicable.



R = Resultant of vertical forces

Note: For relatively thick facing elements (e.g., segmental concrete facing blocks) it may be desirable to include the facing dimensions and weight in bearing capacity calculations (i.e., use "B" in lieu of "L").

Figure 25. Calculation of vertical stress σ_v at the foundation level.

e. **Sliding Stability**

Check the preliminary sizing with respect to sliding at the base layer, which is the most critical depth as follows:

$$FS_{sliding} = \frac{\sum \textit{horizontal resisting forces}}{\sum \textit{horizontal driving forces}} = \frac{\sum P_R}{\sum P_d} \geq 1.5 \qquad (21)$$

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 22 through 24).

Note that any passive resistance at the toe due to embedment is ignored due to the potential for the soil to be removed though natural or manmade processes during its service life (e.g. erosion, utility installation, etc.). The shear strength of the facing system is also conservatively neglected.

Additional surcharge loads may include live and dead load surcharges.

The calculation steps for an MSE wall with a sloping surcharge are:

(1) Calculate thrust $F_T = K_{af\,(\phi,\,\beta)}\ \frac{1}{2}\ \gamma_f\ h^2$ (22)

where, $h = H + L \tan \beta$ (23)

(2) Calculate the driving force:

$$P_d = F_H = F_T \cos \beta. \qquad (24)$$

(3) Determine the most critical frictional properties at the base. Choose the minimum ϕ for three possibilities:

- Sliding along the foundation soil, if its shear strength (c_f, ϕ_f) is smaller than that of the backfill material.

- Sliding along the reinforced backfill (ϕ_r).

- For sheet type reinforcement, sliding along the weaker of the upper and lower soil-reinforcement interfaces (ρ). The soil-reinforcement friction angle ρ, should preferably be measured by means of interface direct shear tests. Alternatively, it may be taken as $\frac{2}{3}$ tan ϕ.

(4) Calculate the resisting force per unit length of wall:

$$P_R = (V_1 + V_2 + F_T \sin \beta) \cdot \mu \tag{25}$$

where

$$\mu = \min [\tan \phi_f, \tan \phi_r, \text{ or (for continuous reinforcement) } \tan\rho]$$

The effect of external loadings on the MSE mass, which increases sliding resistance, should only be included if the loadings are permanent. For example, live load traffic surcharges should be excluded.

(5) Calculate the factor of safety with respect to sliding and check if it is greater than the required value.

(6) If Not:

- Increase the reinforcement length, L and repeat the calculations.

f. Bearing Capacity Failure

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

- General Shear

  To prevent bearing capacity failure, it is required that the vertical stress at the base calculated with the Meyerhof distribution does not exceed the allowable bearing capacity of the foundation soil determined, considering a safety factor of 2.5 with respect to Group I loading applied to the ultimate bearing capacity:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS} \tag{26}$$

A lesser FS of 2.0 could be used if justified by a geotechnical analysis.

Calculation steps for an MSE wall with a *sloping surcharge* are as follows:

(1) Obtain the eccentricity e of the resulting force at the base of the wall. Remember that under preliminary sizing if the eccentricity exceeded L/6, the reinforcement length at the base was increased.

(2) Calculate the vertical stress σ_v at the base assuming Meyerhof distribution:

$$\sigma_v = \frac{V_1+V_2+F_T \sin\beta}{L-2e} \tag{27}$$

(3) Determine the ultimate bearing capacity q_{ult} using classical soil mechanics methods, e.g.:

$$q_{ult} = c_f N_c + 0.5 (L)\gamma_f N_\gamma \tag{28}$$

where c_f is the cohesion, γ_f the unit weight and N_c and N_γ are dimensionless bearing capacity coefficients and can be obtained from 4.7.1A of 1992 AASHTO and by considering that q_{ult} is reduced when the ground at the base of the wall slopes away from the structure in accordance with 4.7.1.14B of AASHTO. Again, the beneficial effect of wall embedment is neglected. For convenience, the dimensionless bearing capacity factors are shown in table 10.

(4) Check that:

$$\sigma_v \leq q_a = q_{ult} / FS \tag{26}$$

(5) As indicated in step (2) and step (3), σ_v can be decreased and q_{ult} increased by lengthening the reinforcements. If adequate support conditions cannot be achieved or lengthening reinforcements significantly increases costs, improvement of the foundation soil is needed (dynamic compaction, soil replacement, stone columns, precompression) etc.

Table 10. Bearing Capacity Factors

ϕ	N_c	N_q	N_γ	ϕ	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.90
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	37.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88	-	-	-	-

- Local Shear

To prevent large horizontal movements of the structure on weak cohesive soils:

$$\gamma H \leq 3c \qquad (29)$$

If adequate support conditions cannot be achieved, ground improvement of the foundation soils is indicated.

g. Overall Stability

Overall stability is determined using rotational or wedge analyses, as appropriate, which can be performed using a classical slope stability analysis method. Computer programs are available for these analyses (see chapter 6). The reinforced soil wall is considered as a rigid body and only failure surfaces completely outside a reinforced mass are considered. 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, sloping faced structures, or stacked structures, compound failures must be considered.

If the minimum safety factor is less than the usually recommended minimum FS of 1.3, increase the reinforcement length or improve the foundation soil.

h. Seismic Loading

During an earthquake, the retained fill exerts a dynamic horizontal thrust, P_{AE}, on the MSE wall in addition to the static thrust. Moreover, the reinforced soil mass is subjected to a horizontal inertia force $P_{IR} = M\ A_m$, where M is the mass of the active portion of the reinforced wall section assumed at a base width of 0.5H, and A_m is the maximum horizontal acceleration in the reinforced soil wall.

Force P_{AE} can be evaluated by the pseudo-static Mononobe-Okabe analysis as shown in figure 26 and added to the static forces acting on the wall (weight, surcharge, and static thrust). The dynamic stability with respect to external stability is then evaluated. Allowable minimum dynamic safety factors are assumed as 75 percent of the static safety

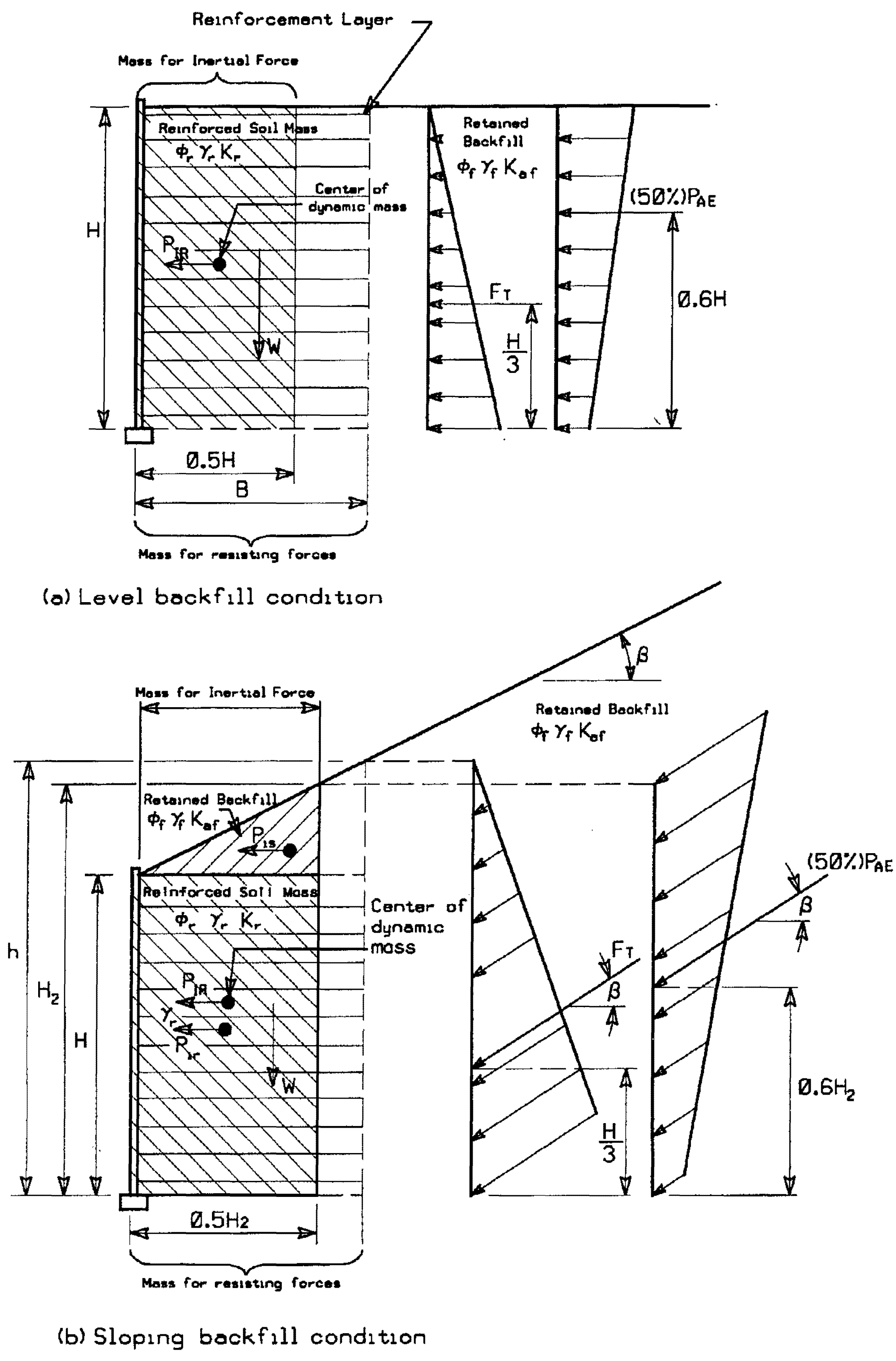


Figure 26. Seismic external stability of a MSE wall.

factors. The equation for P_{AE} (equation 32) was developed assuming a horizontal backfill, a friction angle of 30 degrees and may be adjusted for other soil friction angles using the Mononobe-Okabe method with the horizontal acceleration equal to A_m and vertical acceleration equal to zero.

The seismic external stability evaluation is performed as follows:

- Select a peak horizontal ground acceleration based on the design earthquake. The ground acceleration coefficient may be obtained from Division 1A of current AASHTO where it is given as A, Acceleration Coefficient.

- Calculate the maximum acceleration A_m developed in the wall:

$$A_m = (1.45 - A)\,A \tag{30}$$

where: A = max. ground acceleration coefficient, AASHTO, Division 1A.

A_m = max. wall acceleration coefficient at the centroid of the wall mass.

- Calculate the horizontal inertia force P_{IR} and seismic thrust P_{AE}:

$$P_{IR} = 0.5\, A_m \gamma_r H^2 \tag{31}$$

$$P_{AE} = 0.375\, A_m\, \gamma_f H^2 \quad \text{(Horizontal backslope)} \tag{32}$$

- Add to the static forces F_1 and F_2 (see figure 22) acting on the structure, 50 percent of the seismic thrust P_{AE} and the full inertial force P_{IR}. The reduced P_{AE} is used because these two forces are unlikely to peak simultaneously.

- For structures with sloping backfills, the inertial force (P_{IR}) and the dynamic horizontal thrust (P_{AE}) shall be based on a height H_2 near the back of the wall determined as follows:

$$H_2 = H + \frac{\tan\beta \cdot 0.5H}{(1 - 0.5\tan\beta)} \tag{33}$$

P_{AE} may be adjusted for sloping backfills using Mononobe-Okabe method, with the horizontal acceleration K_h equal to A_m and K_v equal to zero. A height of H_2 should be used to calculate P_{AE} in this case. P_{IR} for sloping backfills should be calculated as follows:

$$P_{IR} = P_{ir} + P_{is} \tag{34}$$

$$P_{ir} = 0.5\, A_m\, \gamma_f\, H_2 H \tag{35}$$

$$P_{is} = 0.125\, A_m\, \gamma_f\, (H_2)^2 \tan\beta \tag{36}$$

and

$$P_{AE} = 0.5\, \gamma_f\, (H_2)^2\, \Delta K_{ac} \quad \text{(sloping backfill)} \tag{32b}$$

where P_{ir} is the inertial force caused by acceleration of the reinforced backfill and P_{is} is the inertial force caused by acceleration of the sloping soil surcharge above the reinforced backfill, with the width of mass contributing to P_{IR} equal to $0.5H_2$. P_{ir} acts at the combined centroid of P_{ir} and P_{is} as shown on figure 26. ΔK_{ae} should be computed in accordance with equation C6-4 of Division 1A of AASHTO. To complete design:

- Evaluate sliding and overturning stability as detailed in the previous sections.

- Check that the computed safety factors are equal to or greater than 75 percent of the minimum static safety factors.

Relatively large earthquake shaking (i.e. $A \geq 0.29$) could result in significant permanent lateral and vertical wall deformations even if limit equilibrium criteria are met. In seismically active areas where such strong shaking could exist, a specialist should be retained to evaluate the anticipated deformation response of the structure.

Note that seismic loads may be reduced, as result of lateral wall movement due to sliding, from what is calculated based on Division 1A using the Mononobe-Okabe method, if both of the following conditions are met.

- the wall system and any structures supported by the wall can tolerate lateral movement resulting from sliding of the structure,

- the wall base is unrestrained regarding its ability to slide, other than soil friction along its base and minimal soil passive resistance.

Procedures for accomplishing this reduction in seismic load are provided in the Article 6 commentary, Division 1A, in particular Equation C6-10, of the 1994 AASHTO Bridge

Specifications. In general, this only applies to gravity and semi-gravity walls. Though the specifications in Division 1A regarding this issue are directed at structural gravity and semi-gravity walls, these specifications may also be applicable to other types of gravity walls, provided the two conditions listed above are met.

i. 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, *Soils and Foundations Manual*). Significant total settlements at the end of construction, indicate that the planned top of wall elevations need to be adjusted. This can be accomplished by increasing the top of wall elevations during design, but more practically, 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 (less 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 or the use of lightweight fill.

4.3 SIZING FOR INTERNAL STABILITY

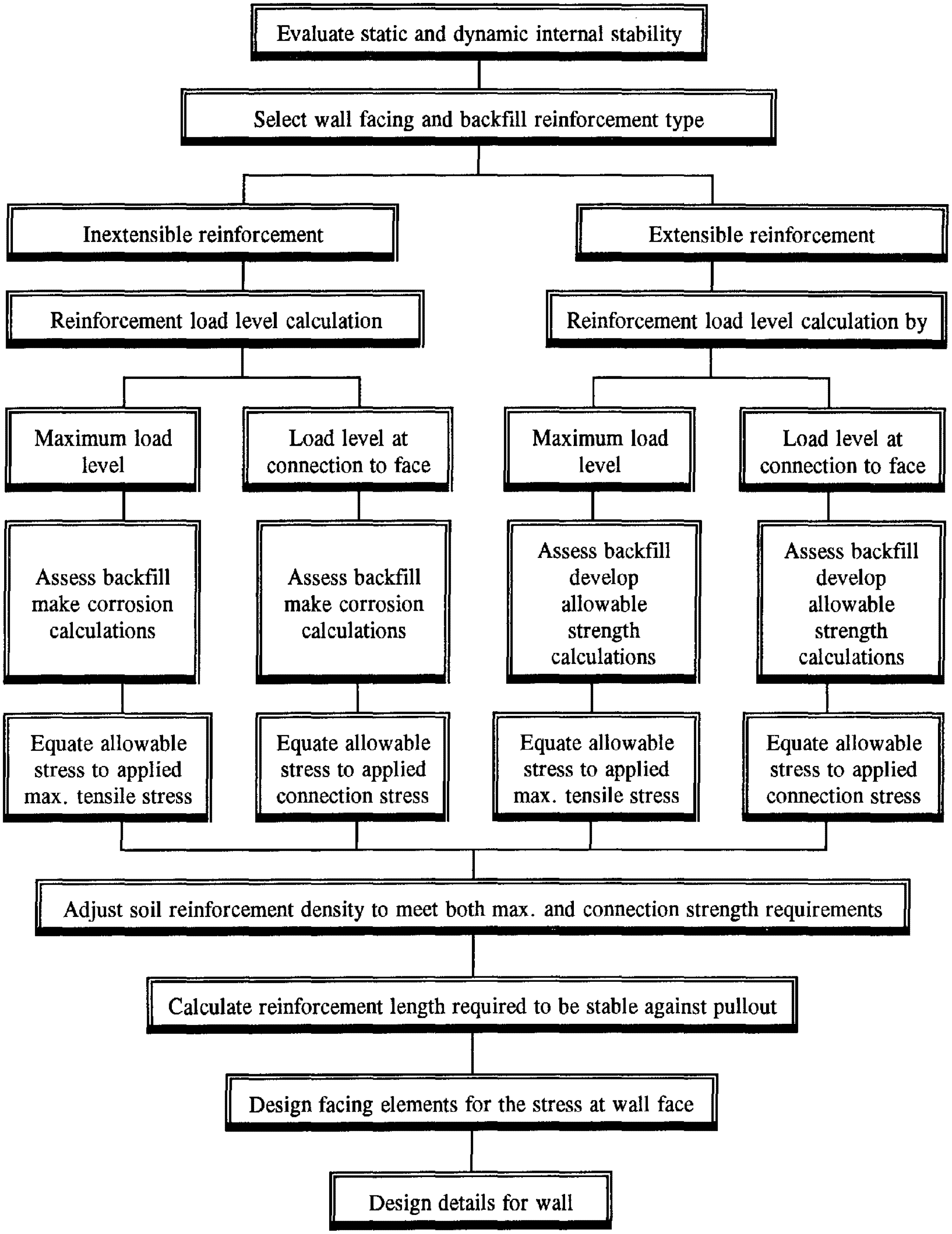
Internal failure of a MSE wall can occur in two different ways:

- 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 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, i.e., the force required to pull the reinforcement out of the soil mass. This, in turn, increases the shear stresses in the surrounding soil, leading to large movements and 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.

Schematically, the design process can be illustrated as follows:



The step by step internal design process is as follows:

- Select a reinforcement type (inextensible or extensible).
- Select the location of the critical failure surface.
- Select a reinforcement spacing compatible with the facing.
- Calculate the maximum tensile force at each reinforcement level, static and dynamic.
- Calculate the maximum tensile force at the connection to the facing.
- Calculate the pullout capacity at each reinforcement level.

a. Critical Slip Surfaces

The most critical slip surface in a simple reinforced soil wall is assumed to coincide with the maximum tensile forces line (i.e., the locus of the maximum tensile force, T_{max}, in each reinforcement layer). The shape and location of this line is assumed to be known for simple structures from a large number of previous experiments and theoretical studies.

This maximum tensile forces surface has been assumed to be approximately bilinear in the case of inextensible reinforcements (figure 27a), approximately linear in the case of extensible reinforcements (figure 27b), 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. Where the wall front batter is greater than 10 degrees the Coulomb earth pressure relationship shown on figure 27b may be used to define the failure surface.

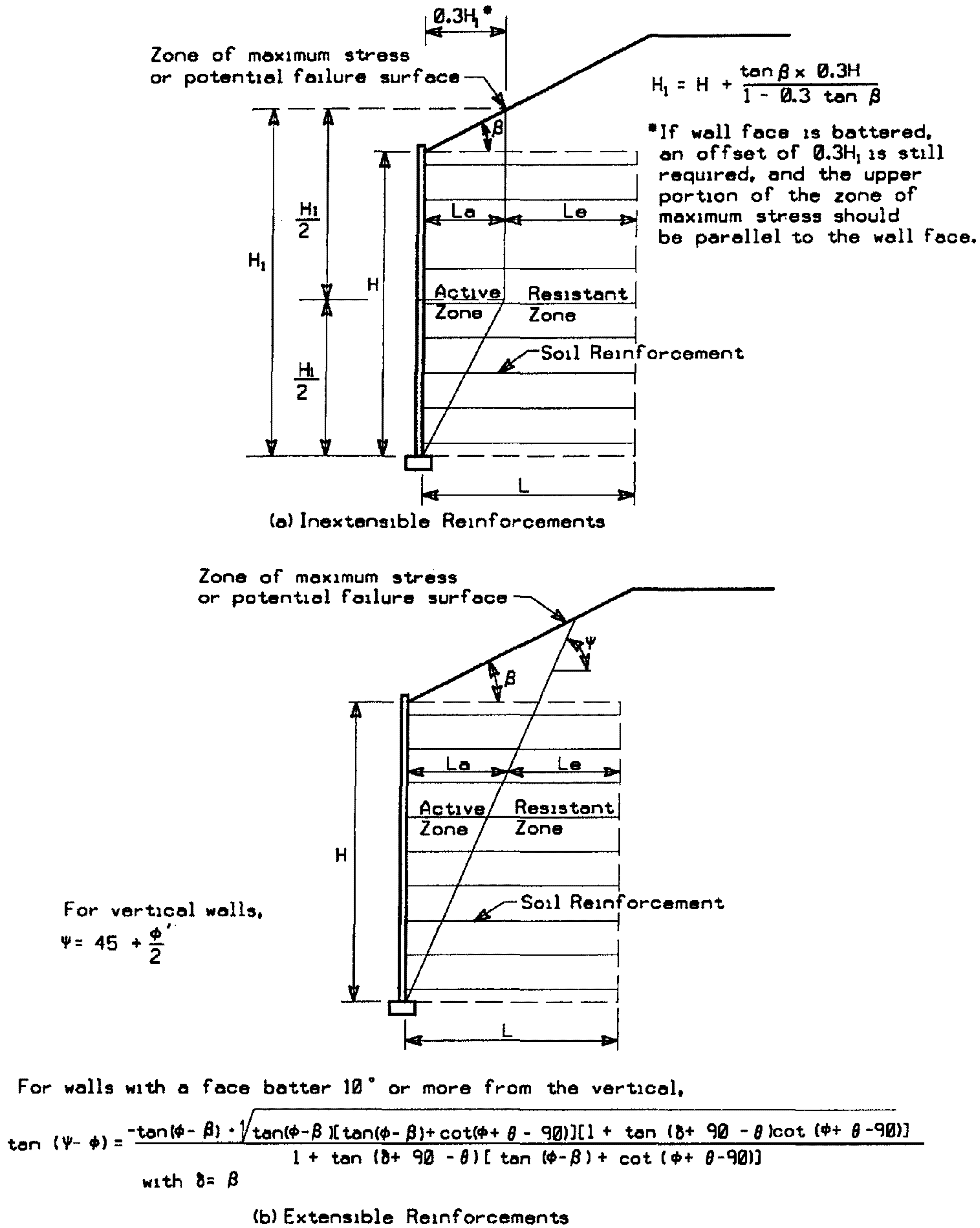


Figure 27. Location of potential failure surface for internal stability design of MSE walls.

b. Calculation of Maximum Tensile Forces in the Reinforcement Layers

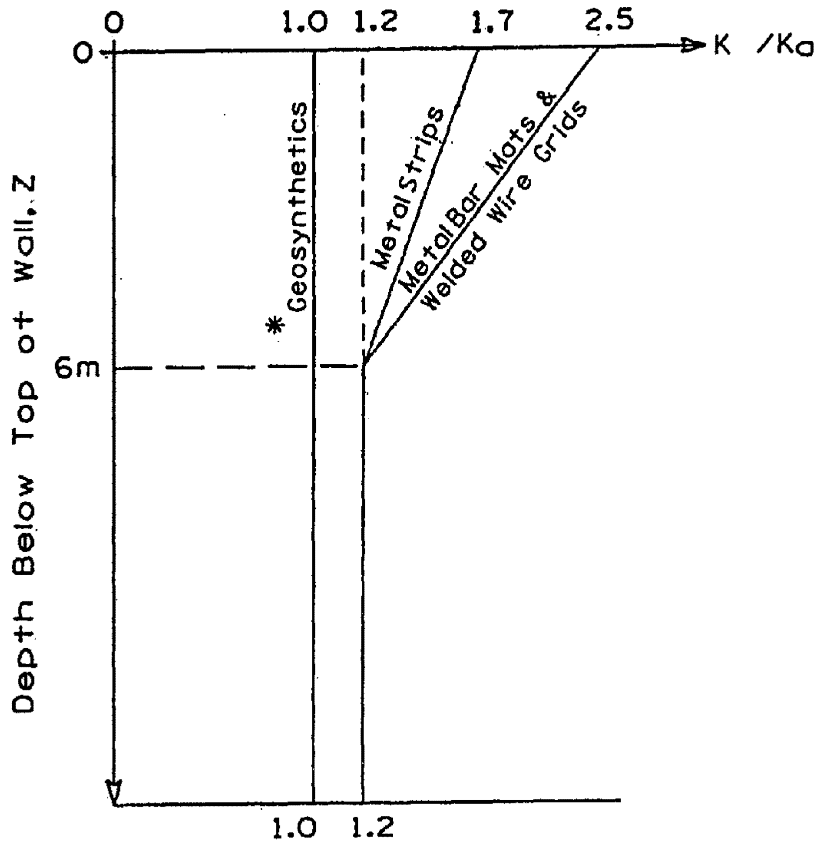
Recent research studies have indicated that the maximum tensile force is primarily related to the type of reinforcement in the MSE mass, which, in turn, is a function of the modulus, extensibility and density of reinforcement. Based on this research, a relationship between the type of the reinforcement and the overburden stress has been developed, and shown in figure 28. The resulting K/Ka for inextensible reinforcements ratio decreases from the top of wall to a constant value below 6 m.

The simplified approach used herein was developed so that iterative design procedures are avoided and by practical considerations of some of the complex refinements of the available methods i.e., the coherent gravity method (AASHTO, 1994 Interims) and the structure stiffness method (FHWA RD 89-043). The *simplified coherent gravity* method is based on these two methods.

This graphical figure was prepared by back analysis of the lateral stress ratio K from available field data where stresses in the reinforcements have been measured and normalized as a function of an active earth pressure coefficient, Ka. The ratios shown on figure 28 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 28, including relationships for newly developed reinforcement types, effect of full height panels, etc.

The lateral earth pressure coefficient K 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. For a vertical wall the earth pressure therefore reduces to the Rankine equation:

$$K_a = Tan^2\ (45 - \phi'/2) \tag{15}$$



*Does not include polymer strip reinforcement

Figure 28. Variation of stress ratio with depth in a MSE wall.

For wall face batters in excess of 10 degrees, the following simplified form of the Coulomb equation can be used:

$$K_a = \frac{\sin^2(\theta+\phi')}{\sin^3\theta\left[1 + \frac{\sin\phi'}{\sin\theta}\right]^2} \tag{37}$$

where θ is the inclination of the face from the horizontal.

The vertical stress (γH) is the result of gravity forces from soil self weight within and immediately above the reinforced wall backfill, and any surcharge loads present. Vertical stress for maximum reinforcement load calculations are shown on figure 29.

Calculations steps are as follows:

(1) Calculate at each reinforcement level the horizontal stresses σ_H along the potential failure line from the weight of the retained fill $\gamma_r Z$ plus, if present, uniform surcharge loads q concentrated surcharge loads $\Delta\sigma_v$ and $\Delta\sigma_h$.

$$\sigma_H = K\,\sigma_v + \Delta\sigma_h \tag{38}$$

where

$$\sigma_v = \gamma_r Z + q + \Delta\sigma_v$$

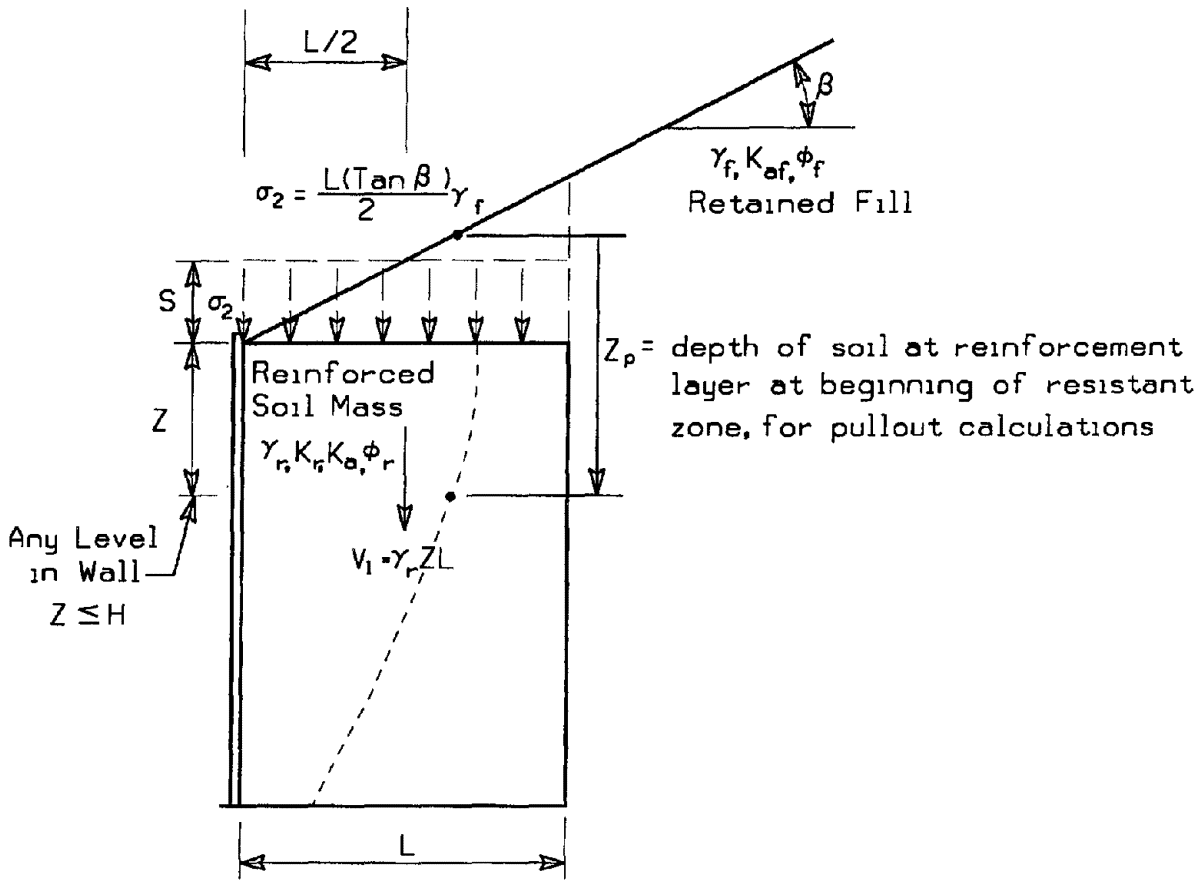
where: K = K(z) is shown in figure 28 and Z is the depth referenced below the top of wall, excluding any copings and appurtenances.

$\Delta\sigma_v$ is the increment of vertical stress due to concentrated vertical loads using a 2V:1H pyramidal distribution as shown in figure 30.

$\Delta\sigma_H$ is the increment of horizontal stress due to horizontal concentrated surcharges, if any, and calculated as shown in figure 31. Static equivalent loads for traffic barriers should be included based on current AASHTO, Section 5.8.

(2) Calculate the maximum tension T_{max} in each reinforcement layer per unit width of wall based on the vertical spacing S_v from:

$$T_{max} = \sigma_H \cdot S_v \tag{39}$$



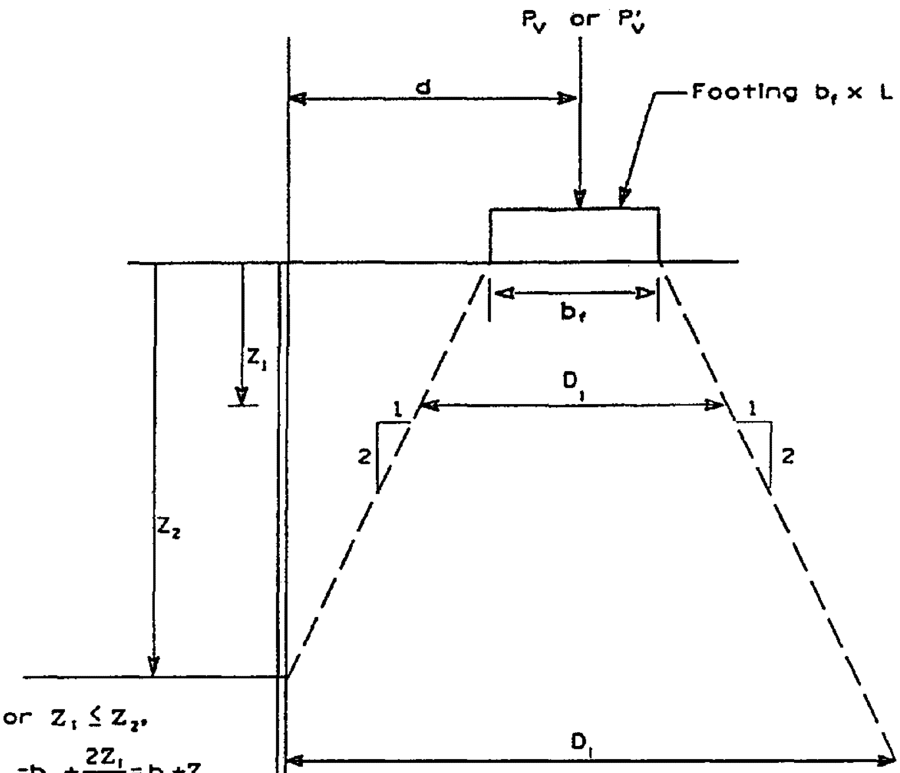
Max Stress: $S = \frac{1}{2} L \tan\beta$

$\sigma_v = \gamma_r Z + \frac{1}{2} L (\tan\beta)\gamma_f$

with K_a determined using a slope angle of 0°.

Note: H is the total height of the wall at the face.

Figure 29. Calculation of vertical stress for sloping backslope condition.



For $Z_1 \leq Z_2$,

$$D_1 = b_f + \frac{2Z_1}{2} = b_f + Z_1$$

For $Z_1 > Z_2$,

$$D_1 = \frac{b_f + Z_1}{2} + d$$

For strip load:

$$\Delta \sigma_v = \frac{P_v}{D_1}$$

For isolated footing load:

$$\Delta \sigma_v = \frac{P'_v}{D_1(L + Z_1)}$$

For point load:

$$\Delta \sigma_v = \frac{P'_v}{D_1^2} \text{ with } b_f = 0$$

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 = Length of footing

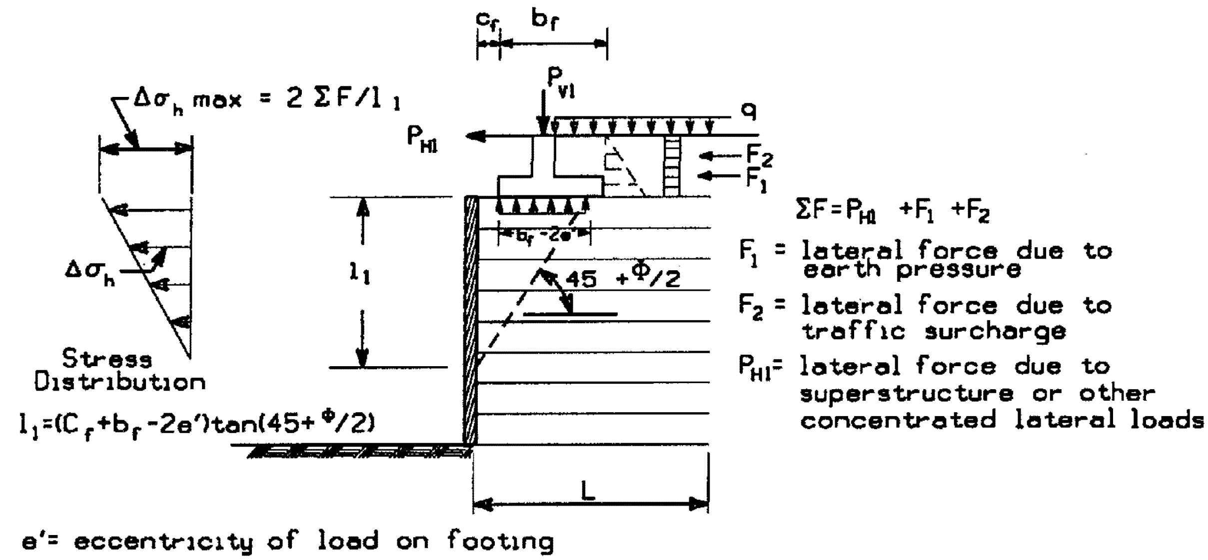
P_v = Load per linear meter (foot) of strip footing

P'_v = Load on isolated rectangular footing or point load

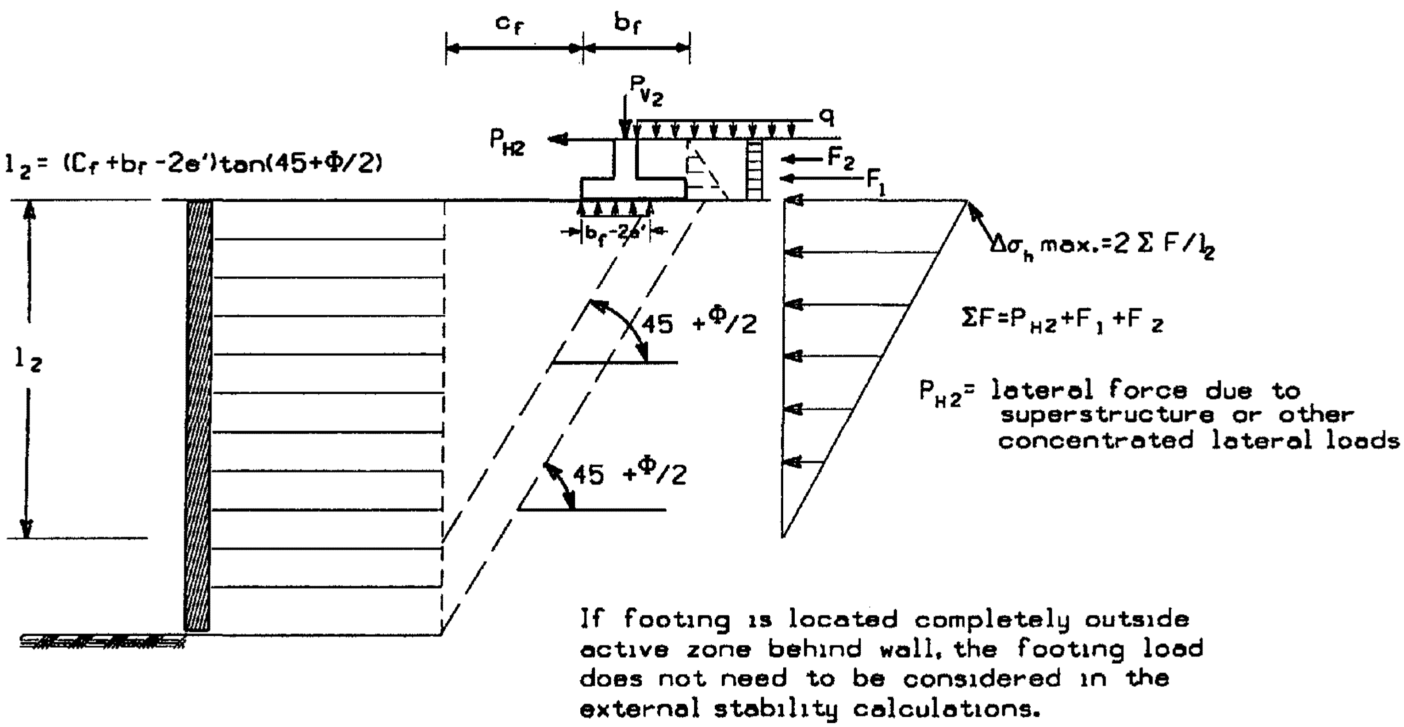
z_2 = depth where effective width intersects back of wall face = 2d - b

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 th active zone behind the wall.

Figure 30. Distribution of stress from concentrated vertical load P_v for internal and external stability calculations.



a. Distribution of Stress for Internal Stability Calculations.



b. Distribution of Stress for External Stability Calculations.

Figure 31. Distribution of stress from concentrated horizontal loads.

T_{max} may be also be calculated at each level for discrete reinforcements (metal strips, bar mats, geogrids, etc.) per a defined unit length of wall face.

(3) Calculate internal stability with respect to breakage of the reinforcement. Stability with respect to breakage of the reinforcements requires that:

$$T_a \geq \frac{T_{max}}{R_c} \tag{40}$$

where R_c is the coverage ratio b/S_H, with b the gross width of the reinforcing element, and S_H is the center-to-center horizontal spacing between reinforcements (e.g., $R_c = 1$ for full coverage reinforcement). T_a is the allowable tension force per unit width of the reinforcement.

The connection of the reinforcements with the facing, must also be checked to ensure that tensile force at the connection T_o, determined as indicated in figure 32, is not greater than the allowable tensile strength of the connection. For walls with full height panels the connection tensile force T_o shall not be reduced and would therefore equal T_{max}. No reduction at the face shall be made where concentrated horizontal and vertical surcharge loads are applied near the wall face.

c. Internal Stability with Respect to Pullout Failure

Stability with respect to pullout of the reinforcements requires that the following criteria be satisfied:

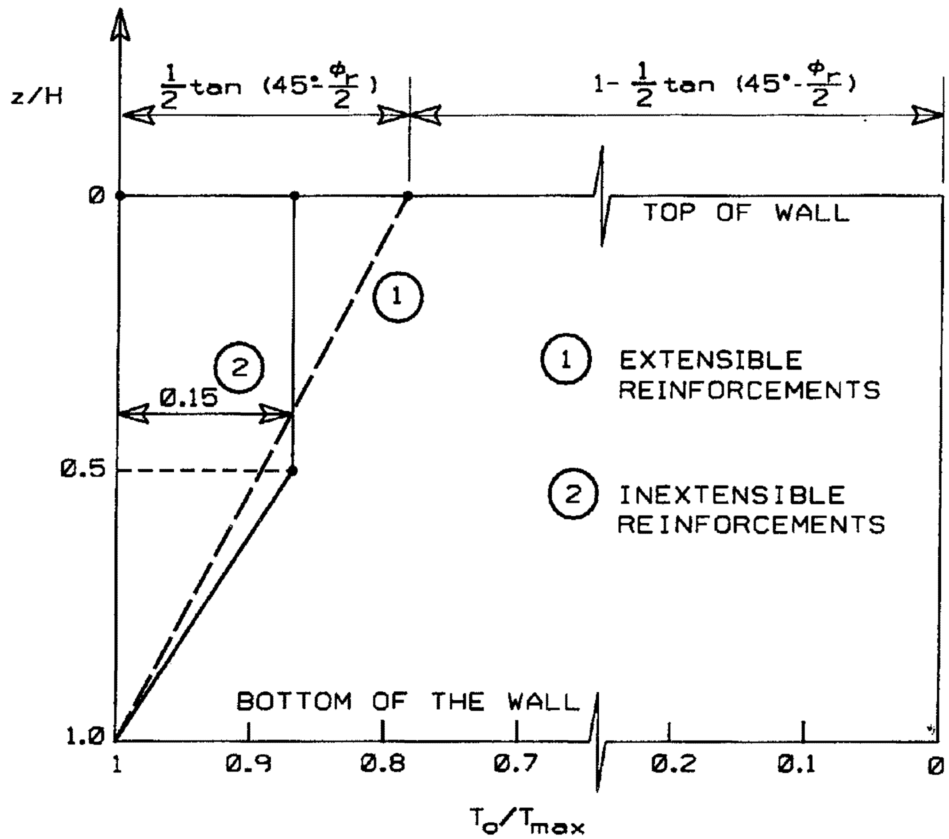
$$T_{max} \leq \frac{1}{FS_{PO}} F^* \gamma Z_p L_e C R_c \alpha \tag{41}$$

where: FS_{PO} = Safety factor against pullout $\geq$ 1.5.

T_{max} = Maximum reinforcement tension.

C = 2 for strip, grid, and sheet type reinforcement.

α = Scale correction factor.



Note: This does not apply to full height precast concrete panels, or cases in which concentrated horizontal and vertical surcharge loads are applied near the wall face.

Figure 32. Determination of the tensile force T_o in the reinforcement at the connection with the facing.

F^* = Pullout resistance factor (see chapter 3).

R_c = Coverage ratio.

γZ_p = The overburden pressure, including distributed dead load surcharges, neglecting traffic loads. (See figure 29)

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.

Therefore, the required embedment length in the resistance zone (i.e., beyond the potential failure surface) can be determined from:

$$L_e \geq \frac{1.5\ T_{max}}{C\ F^*\ \gamma\ Z_p R_c \alpha} \geq 1\ m \tag{42}$$

Note that traffic loads are not included in T_{max} in pullout calculations as indicated on figure 22.

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.

The total length of reinforcement, L, required for internal stability is then determined from:

$$L = L_a + L_e \tag{43}$$

where: L_a is obtained from figure 27 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.

$$L_a = (H - Z) \tan (45 - \phi'/2) \tag{44}$$

where: Z is the depth to the reinforcement level.

For walls with **inextensible** reinforcement from the base up to H/2:

$$L_a = 0.6\ (H\text{-}Z) \tag{45}$$

For the upper half of a wall with inextensible reinforcements:

$$L_a = 0.3H \tag{46}$$

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 5, section 5.3 for additional guidance.

d. Seismic Loading

Seismic loads produce an inertial force P_I acting horizontally, in addition to the existing static forces.

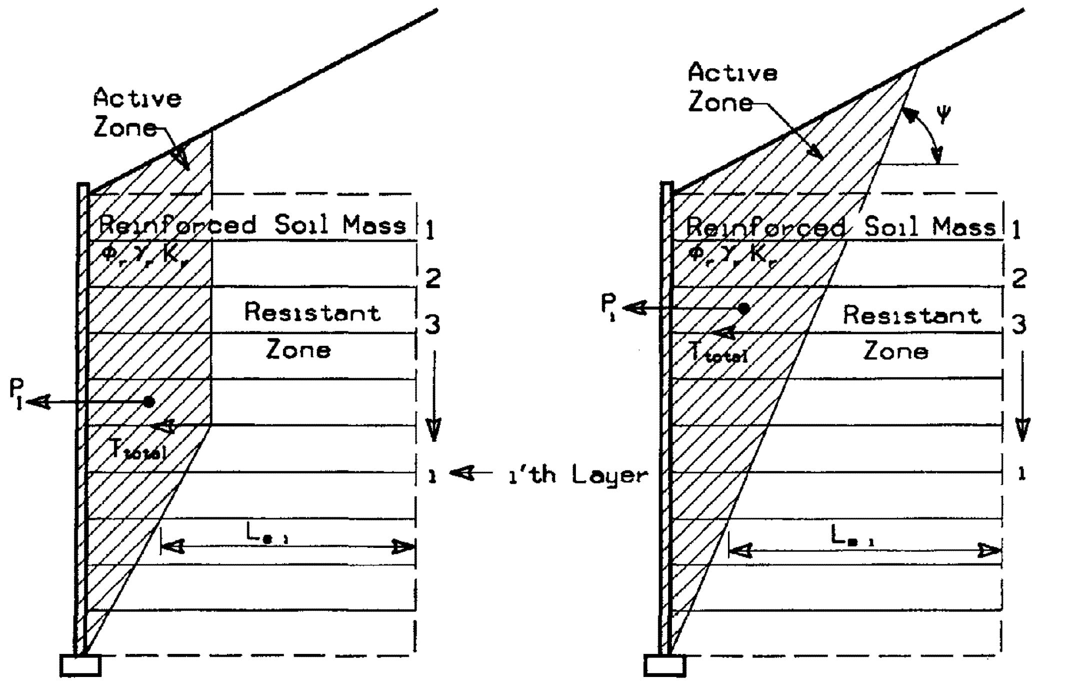
This force will lead to incremental dynamic increases in the maximum tensile forces in the reinforcements. It is assumed that the location and slope of the maximum tensile force line does not change during seismic loading (this assumption is conservative relative to pullout resistance). Calculation steps for internal stability analyses with respect to seismic loading are as follows (see figure 33).

(1) Calculate the maximum acceleration in the wall and the force P_I per unit width acting above the base:

$$P_I = A_m\ W_A \tag{47}$$

$$A_m = (1.45 - A)\ A \tag{48}$$

where: W_A is the weight of the active zone (shaded area on figure 33) and A is the AASHTO site acceleration coefficient.



Inextensible Reinforcements Extensible Reinforcements

P_I = Internal inertial force due to the weight of the backfill within the active zone.

L_{ei} = The length of reinforcement in the resistant zone of the i'th layer.

T_{max} = The load per unit wall width applied to each reinforcement due to static forces.

T_{md} = The load per unit wall width applied to each reinforcement due to dynamic forces.

The total load per unit wall width applied to each layer, $T_{total} = T_{max} + T_{md}$

Figure 33. Seismic internal stability of a MSE wall.

(2) Calculate the total maximum static load applied to the reinforcements horizontal T_{max} as follows:

Calculate horizontal stress σ_H using K coefficient (previously developed)

$$\sigma_H = K\sigma_v + \Delta\sigma_h = K\gamma Z + qK + \Delta\sigma_v K + \Delta\sigma_H \qquad (38)$$

Calculate the maximum tensile load component T_{max} per unit width:

$$T_{max} = S_v \, \sigma_H \qquad (39)$$

(3) Calculate the dynamic increment T_{md} directly induced by the inertia force P_I in the reinforcements by distributing P_I in the different reinforcements proportionally to their "resistant area" (L_e) on a load per unit wall width basis. This leads to:

$$T_{md} = P_I \frac{L_{ei}}{\sum_{i=1}^{n} (L_{ei})} \qquad (49)$$

which is the resistant length of the reinforcement at level i divided by the sum of the resistant length for all reinforcement levels.

(4) The maximum tensile force is:

$$T_{total} = T_{max} + T_{md} \qquad (50)$$

Check stability with respect to breakage and pullout of the reinforcement, with seismic safety factors of 75 percent of the minimum allowable static safety factor. For geosynthetic reinforcements the allowable strength T_a need not be reduced for creep and the friction coefficients (F^*) should be reduced to 80 percent of the static values for all reinforcements. This leads to:

For breakage failure: $$T_{total} < \frac{T_a}{0.75} \qquad (51)$$

For pullout failure: $$T_{total} \leq \frac{P_r\, R_c}{0.75\ FS_{PO}} \qquad (52)$$

$$T_{total} \leq \frac{C \cdot (0.8F^*)}{0.75 \cdot 1.5} \cdot \gamma Z' \cdot L_e\, R_c \qquad (53)$$

The recommended design method with respect to seismic loading was developed for inextensible reinforcements but it is also applicable to extensible reinforcements. The extensibility of the reinforcements affects the overall stiffness of the reinforced soil mass. As extensible reinforcement reduces the overall stiffness it is expected to have an influence on the design diagram of the lateral earth pressure induced by the seismic loading. As the overall stiffness decreases, damping should increase and amplification may also increase. Thus, the resulting inertia force may not be much different than for inextensible reinforcement.

e. Connection Strength

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 Section 8.31 of 1992 AASHTO for each geometry used. The design load is equal to T_o, as developed in this chapter.

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 *botkin* joint, as illustrated in figure 34. A slat of polyethylene may be used for the botkin, though rigid PVC pipes have also been used. Extreme care should be exercised to eliminate slack from the 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.

MSE walls constructed with MBW units are connected either by a structural connection subject to verification under AASHTO Article 8.31 or by friction between the units and the reinforcement, including the friction developed from the aggregate contained within the core of the units or by a combination of friction and shear from connection devices. This strength will vary with each unit depending on its geometry, unit batter, normal pressure and depth of unit. The connection strength is therefore specific to each unit/reinforcement combination and must be developed uniquely by test for each combination. Recommended test procedures are included in appendix A.

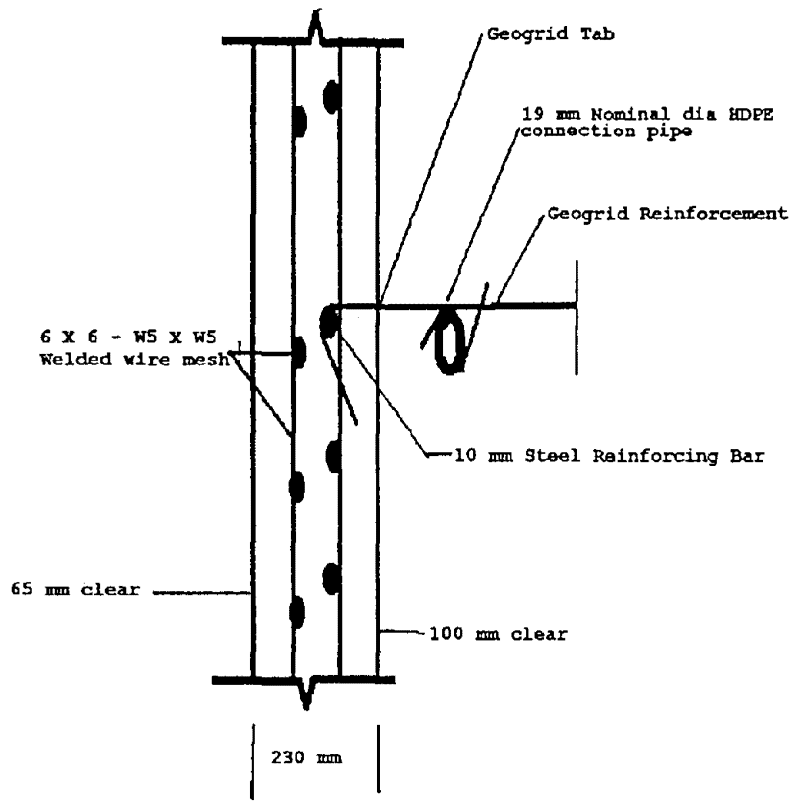


Figure 34. Botkin connection detail.

The recommended procedure for developing allowable connection strength T_{ac} requires that this strength is the lesser of:

- The design allowable strength of the reinforcement (T_{al}) as developed in Chapter 3, *Establishment of Structural Design Properties*.

- The reduced ultimate connection strength based on connection/seam strength (e.g., botkin, seams, block/reinforcement) CR_u as determined from ASTM D-4884 for seams and appendix A.3 for partial or full friction connections.

- The connection strength based on pullout as developed by testing CR_s as outlined in appendix A.3.

- The maximum connection strength as developed by testing reduced for long term environmental aging, creep and divided by a factor of safety of at least 1.5 for permanent structures, as follows:

$$T_{ac} = \frac{T_{ult} \; x \; CR_u}{RF_D \cdot RF_{CR} \cdot 1.5} \leq T_{ult} \; x \; CR_s \tag{54}$$

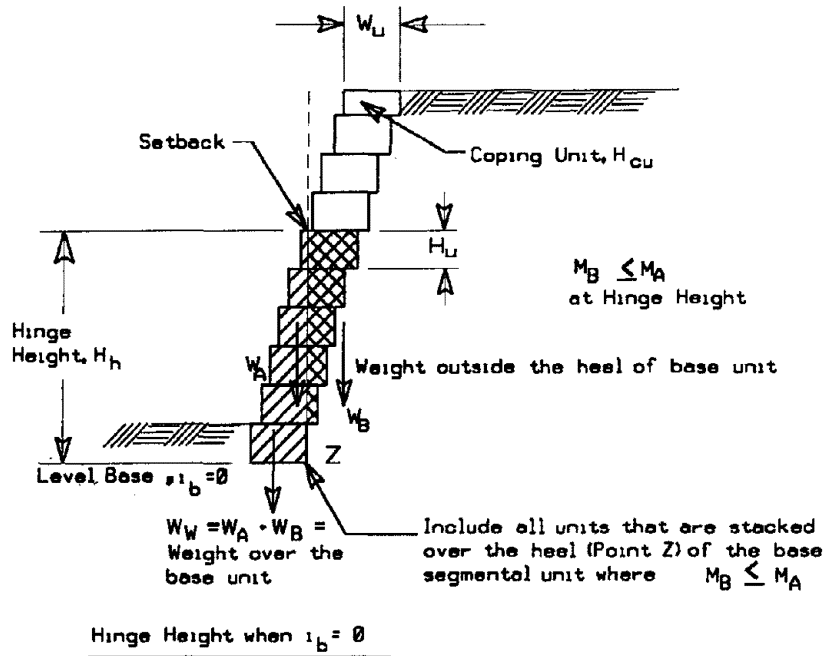
Note that the environment at the connection may not be the same as the environment within the MSE mass. Therefore, the long-term environmental aging factor (RF_D) may be significantly different than that used in computing the allowable reinforcement strength T_a.

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 35 which develops a hinge height concept.[2] Hence, for walls with a batter, 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_u and CR_s.

For connection strength under seismic loading, T_{ac} may be determined by reducing the creep reduction factor RF_{CR} and using a factor of safety of 1.1, unless a default RF is used. Where T_{ac} is partially or fully dependent on friction, T_{ac} shall be reduced to 80 percent of its static value.

Note further that because the connection strength is developed by the weight of units above, its performance under seismic load is uncertain.

Therefore, it is presently recommended that fully frictional connections not be used in locations where the combined seismic performance category is C or higher.



Hinge Height, H_h. The full weight of all segmental facing block units within H_h will be considered to act at the base of the lowermost segmental facing block.

$$H_h = 2\,[(W_u - G_u - 0.5H_u \tan i_b)\cos i_b]/\tan(\omega + i_b)$$

where:

H_u = Segmental facing block unit height (m)
W_u = Segmental facing block unit width, front to back (m)
G_u = distance to the center of gravity of a horizontal segmental facing block unit, including aggregate fill, measured from the front of the unit (m)
ω = wall batter due to setback per course (deg)
H = total height of wall (m)
H_h' = hinge height (m)

Figure 35. Determination of hinge height for segmental concrete block faced MSE Walls.

f. Reinforcement Spacing

Use of a constant reinforcement section and spacing for the full height of the wall usually gives more reinforcement near the top 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 mass, vertical spacing of primary reinforcement should not exceed 800 mm.

There are generally two practical ways to accomplish this for MSE walls with segmental precast concrete facings:

- For reinforcements consisting of strips, grids, or mats, 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 horizontal spacing of 50 mm x 4 mm strips is usually 0.75 m, although the horizontal reinforcement spacing can be decreased by adding reinforcement locations.

- For continuous sheet reinforcements, made of geotextiles or geogrids, a common way of varying the reinforcement density T_a/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 spacings 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_a/S_v can also be varied by changing the strength (T_a) especially if wrapped facing techniques requiring a constant wrap height are used.

  Low-to medium-height walls (e.g., <5 m) are usually constructed with one strength geosynthetic. Taller walls use multiple strength geosynthetics. For example the 12.6 m high Seattle preload wall used four strengths of geotextiles. A maximum spacing of 500 mm is typical for wrapped faced geosynthetic walls, although a smaller spacing is desirable to minimize bulging.

- For walls constructed with modular blocks the small height of each unit is such that it would be uneconomical to place reinforcement at each level. Therefore, the maximum stable unreinforced height provides an upper limit to vertical spacing and the block height provides a minimum.

  The maximum vertical spacing of 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 at one-half the vertical spacing.

4.4 DESIGN OF FACING ELEMENTS

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 average loading conditions at each depth in accordance with structural design requirements in Section 8, 10 and 13 of AASHTO for concrete, steel and timber facings, respectively. Allowable stresses for seismic design may be increased by 50 percent for steel, 33 percent for concrete and 50 percent for timber. The embedment of the soil reinforcement to panel connector must be developed by test, to ensure that it can resist the design T_o forces.

As a minimum, temperature and shrinkage steel must be provided. Epoxy protection of panel reinforcement where salt spray is anticipated is recommended.

b. Design of Flexible Wall Facings

Welded wire, expanded metal, or similar facing panels shall be designed in a manner which prevents the occurrence of excessive bulging as backfill behind the facing elements compresses due to compaction stresses or self weight of the backfill. This may be accomplished by limiting the size of individual panels vertically and the vertical spacing of the soil reinforcement layers, and by requiring the facing panels to have an adequate amount of vertical slip between adjacent horizontal panels. Furthermore, the top of the flexible facing panel at the top of the wall shall be attached to a soil reinforcement layer to provide stability to the top facing panel.

For segmental concrete facing blocks (MBW), the inter-unit shear capacity must be calculated, and the maximum spacing between reinforcement layers shall be limited to twice the width, W_u (see figure 35), of the proposed segmental concrete facing unit or 0.8 m whichever is less. The maximum facing height above the uppermost reinforcement layer and the maximum depth of facing below the bottom reinforcement layer should be limited to the width, W_u (see figure 35), of the proposed segmental concrete facing unit.

For seismic performance categories "C" or higher (AASHTO Division 1A), facing connections in segmental block faced walls (MBW) shall not be fully dependent on frictional resistance between the backfill reinforcement and facing blocks. Shear resisting devices between the facing blocks and soil reinforcement such as shear keys, pins, etc. shall be used. For connections partially or fully dependent on friction between the facing blocks and the soil reinforcement, the long-term connection strength T_{ac}, can be reduced to 80 percent of its static value. Further, the blocks above the uppermost layer soil reinforcement layer must be secured against toppling under all seismic events.

Geosynthetic facing elements shall 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 shall be stabilized to be resistant to ultraviolet radiation. Furthermore, product specific test data shall 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 shall be constructed in addition (e.g., concrete, shotcrete, etc.).

4.5 DESIGN DETAILS

The successful implementation of MSE wall projects often depends on certain design details not directly connected with internal or external stability considerations. Common details requiring consideration and analysis, with provided guidance, include:

- **Traffic Barriers**

  The impact traffic load on barriers constructed over the front face of MSE walls, must be designed and to resist the overturning moment by their own mass in accordance with Article 5.8 of current AASHTO.

The impact force is generally taken as 45 kN applied at a height of 850 mm above the base. This impact force, adds an additional horizontal force of 29 kN per linear meter to the upper 2 rows of reinforcement, *which the reinforcements can resist over their full length.*

For geosynthetic reinforcements, the geosynthetic strength used to structurally size the reinforcements to resist the impact load may be increased by decreasing the reduction factor for creep.

For the currently specified impact loads, the detail shown in figure 36 has been successfully used. Typically, the base slab length is 6 m, jointed to adjacent slabs with shear dowels. Parapet reinforcement shall be designed in accordance with AASHTO Article 2.7. The anchoring slab shall be strong enough to resist the ultimate strength of the standard parapet.

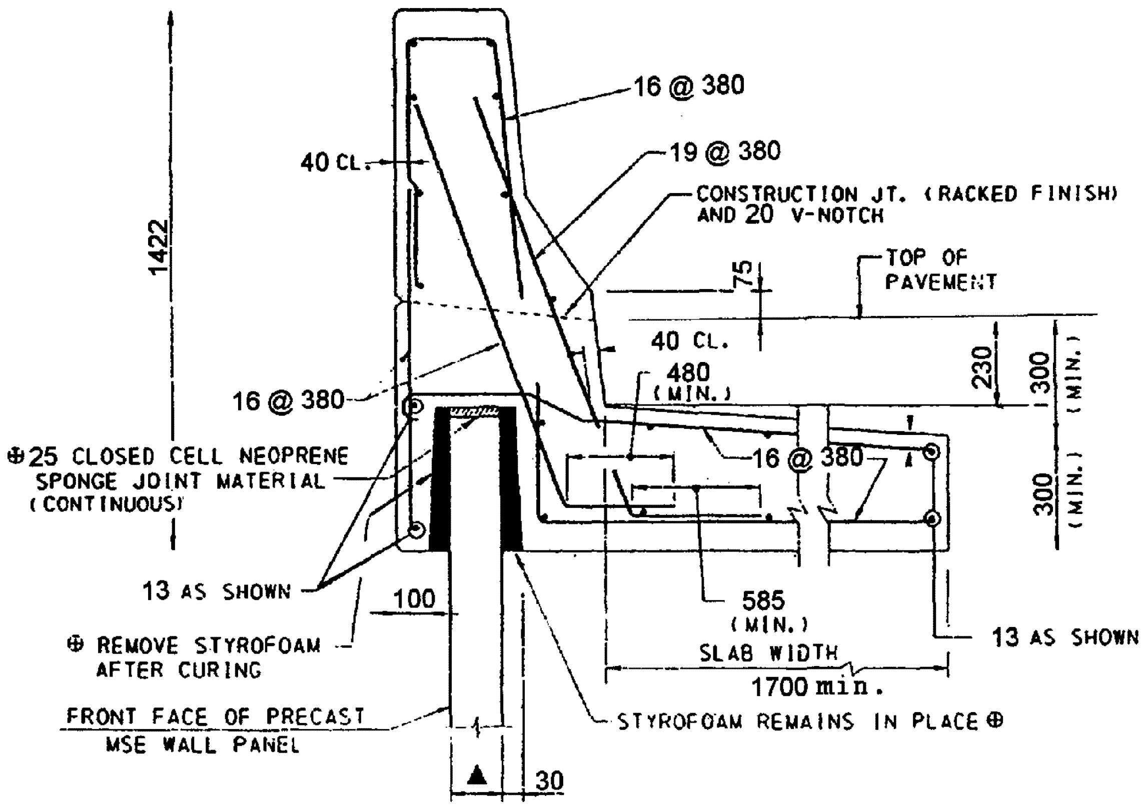
Flexible post and beam barriers, when used, shall be placed at a minimum distance of 1.0 m from the wall face, driven 1.5 m below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction. The upper two rows of reinforcement shall be designed for an additional horizontal load of 4,400 N per linear meter of wall.

- **Drainage Systems**

For side hill construction, drainage blankets are highly recommended to collect and divert groundwater from the reinforced soil mass. A common detail is shown on figure 37.

Where significant use of de-icing salts is anticipated, impervious barriers beneath the pavement structure and just above the reinforced fill zone have been used. A common detail is shown in figure 38.

Where utilities must be placed parallel to the face of the wall, interference with the reinforcement generally occurs. To be effective, the reinforcement can only be skewed vertically for the limited heights as shown in figure 39.



BARRIER SECTION (WITHOUT SHOULDER)
(C. I. P.)

Note: All dimensions in mm.

Figure 36. Impact load barrier.

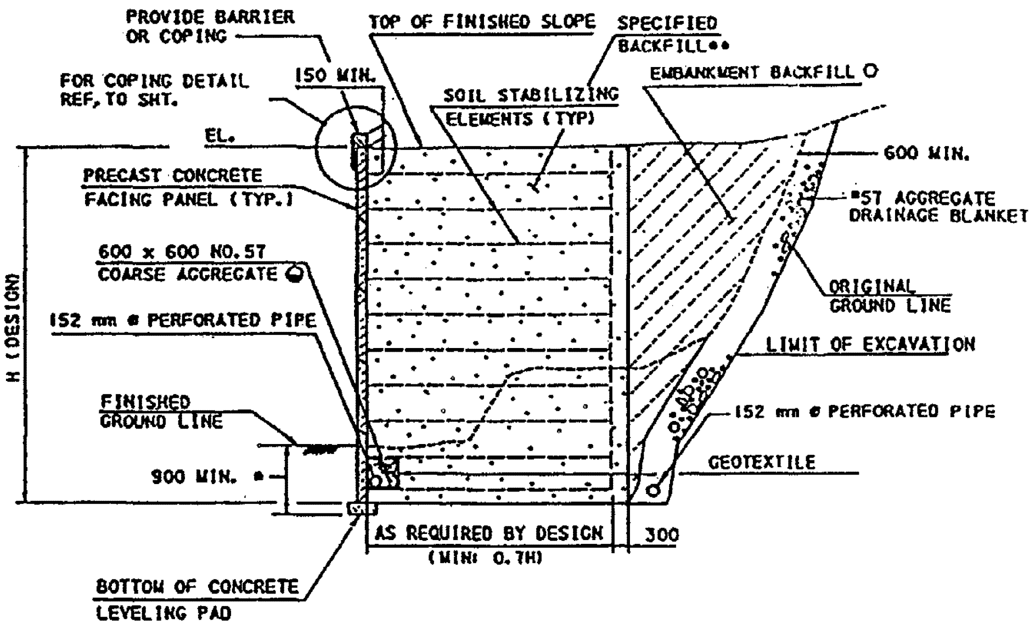


Figure 37. Drainage blanket detail.

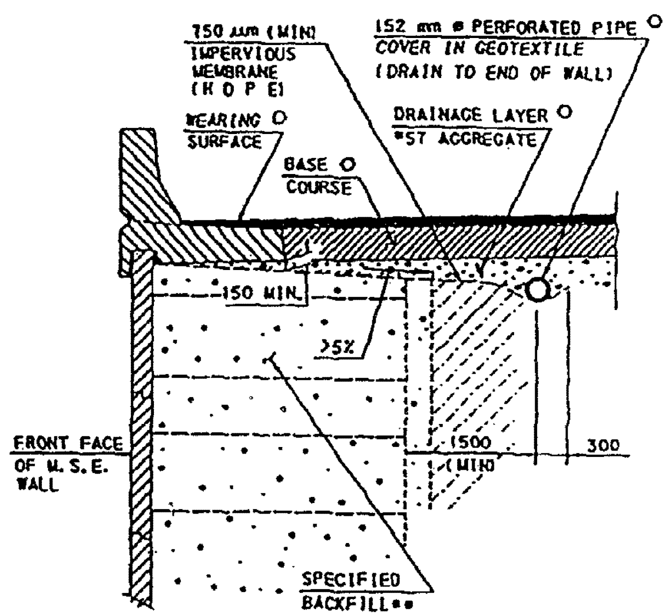


Figure 38. Impervious membrane details.

- **Termination to Cast-in-place Structures**

  The juncture of MSE walls and cast-in-place structures must be protected from loss of fines and must allow for differential settlement between the two types of construction. A common detail is shown in figure 40.

- **Hydrostatic Pressures**

  For structures along rivers and canals, a minimum differential hydrostatic pressure equal to 1.0 m of wall shall be applied at the high-water level. 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.

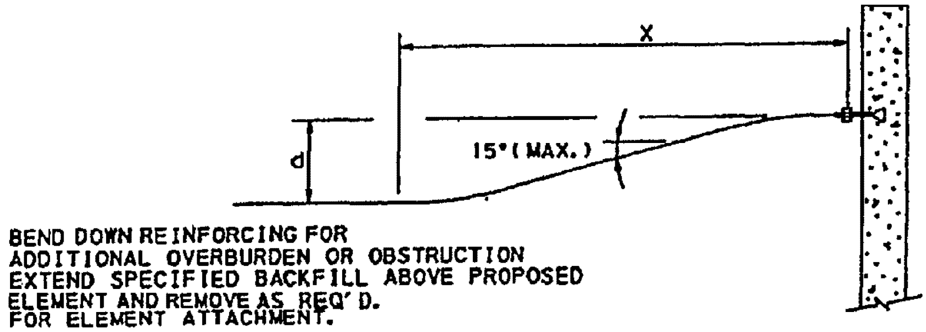
  Situations where the wall is influenced by tide or river fluctuations may require that the wall be designed for rapid drawdown conditions, which could result in differential hydrostatic pressure considerably greater than 1.0 m or alternatively rapidly draining backfill material such as shot rock or open graded coarse gravel be used as backfill. Backfill material meeting the gradation requirements in chapter 8, section 8.8 is not considered to be rapid draining.

- **Obstructions in Reinforced Soil Zone**

  If the placement of an obstruction in the wall soil reinforcement zone such as a catch basin, grate inlet, signal or sign foundation, guardrail post, or culvert cannot be avoided, the design of the wall near the obstruction shall be modified using one of the following alternatives:

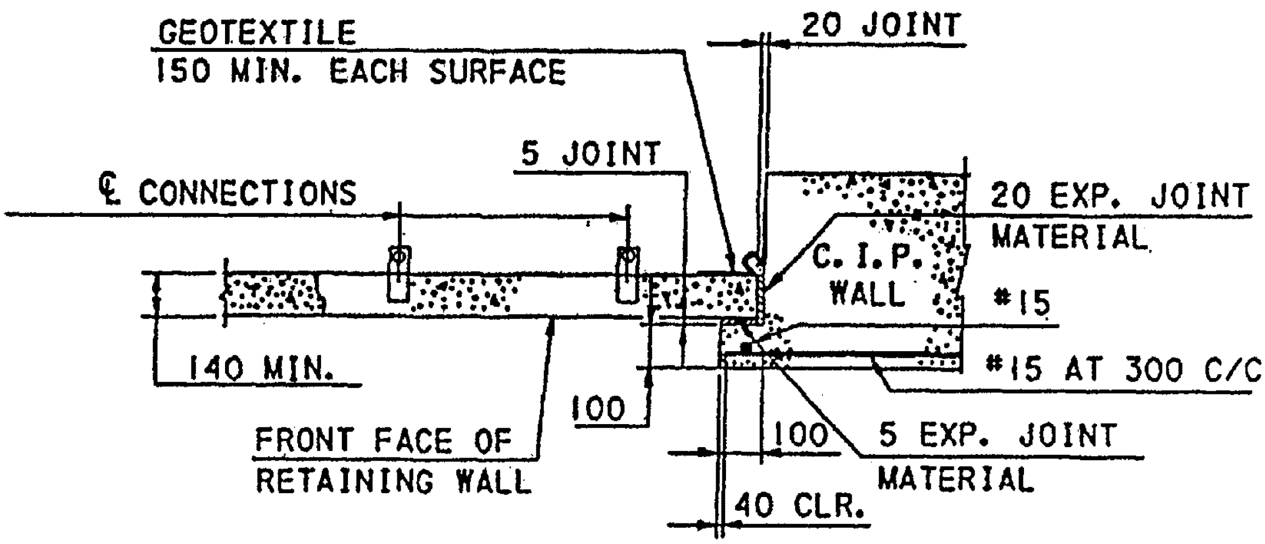
  - Assuming reinforcement layers must be partially or fully severed in the location of the obstruction, design the surrounding reinforcement layers to carry the additional load which would have been carried by the severed reinforcements.

  - Place a structural frame around the obstruction which is capable of carrying the load from the reinforcements in front of the obstruction to reinforcement connected to the structural frame behind the obstruction. This is illustrated in figure 41.



ADDITIONAL DEPTH (d) OF REQUIRED	REQUIRED MINIMUM DISTANCE (X) TO ACHIEVE SMOOTH BEND
75 mm	675 mm
150	975
225	1200
300	1500
375	1800

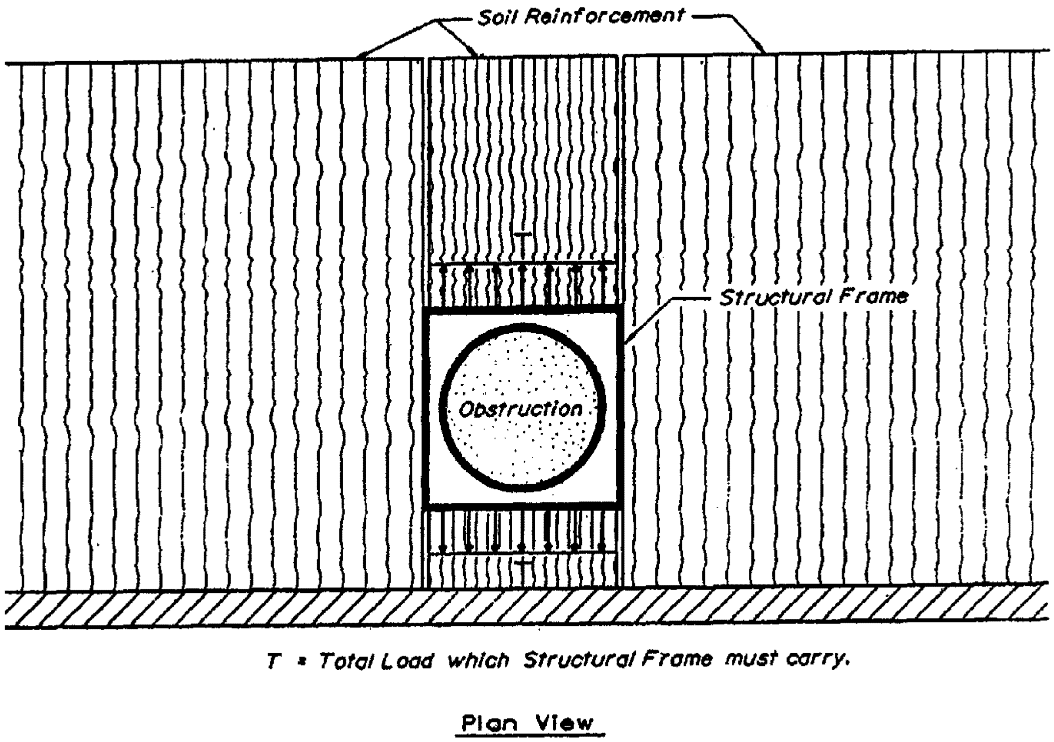
Figure 39. Reinforcing strip or mesh bend detail.



Note: All dimensions in mm.

Figure 40. Connection detail of junctures of MSE walls CIP structure.

a)



b)

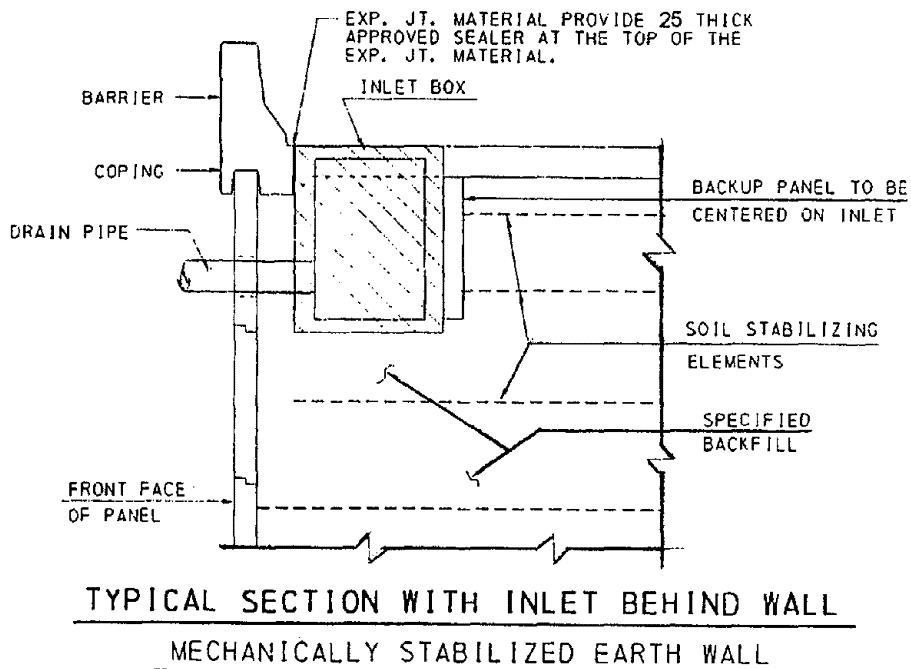


Figure 41. Obstruction details; a) conceptual b) at inlet.

- If the soil reinforcements consist of discrete strips or bar mats rather than continuous sheets, depending on the size and location of the obstruction, it may be possible to splay the reinforcements around the obstruction.

For the first alternative, the portion of the wall facing in front of the obstruction shall be made stable against a toppling (overturning) or sliding failure. If this cannot be accomplished, the soil reinforcements between the obstruction and the wall face can be structurally connected to the obstruction such that the wall face does not topple, or the facing elements can be structurally connected to adjacent facing elements to prevent this type of failure.

For the second alternative, the frame and connections shall be designed in accordance with AASHTO Article 10.32 for steel frames. Note that it may be feasible to connect the soil reinforcement directly to the obstruction depending on the reinforcement type and the nature of the obstruction.

For the third alternative, the splay angle, measured from a line perpendicular to the wall face, shall be small enough that the splaying does not generate moment in the reinforcement or the connection of the reinforcement to the wall face. The tensile capacity of the splayed reinforcement shall be reduced by the cosine of the splay angle.

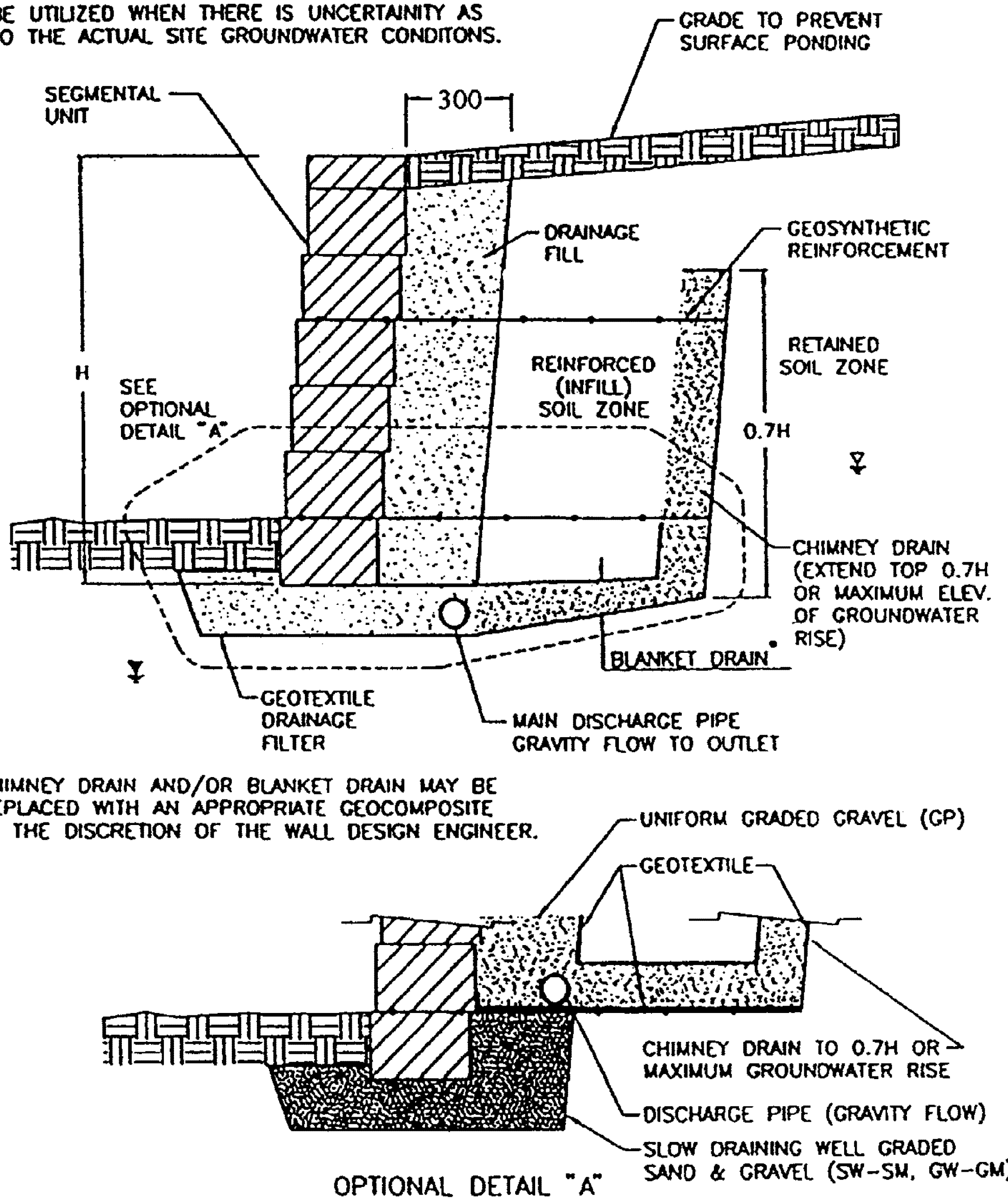
If the obstruction must penetrate through the face of the wall, the wall facing elements shall be designed to fit around the obstruction such that the facing elements are stable (i.e., point loads should be avoided) and such that wall backfill soil cannot spill through the wall face where it joins the obstruction. To this end a collar next to the wall face around the obstruction may be needed.

- **Internal Details**

Placement of well graded gravel immediately adjacent to modular blocks is recommended for several reasons. Gravel has a high permeability that will not impede water flow out of the reinforced mass and through the dry stacked modular blocks. Well graded gravel is not prone to piping through joints between modular blocks. Gravel is also easily placed and compacted, especially adjacent to elements such as modular blocks.

It is recommended that a minimum width of 0.3 m of well graded gravel be specified immediately behind *solid* modular block units, as illustrated in figure 42. A minimum

1. GROUNDWATER TABLE NEAR BOTTOM OF WALL (▽) OR POSSIBLE LATERAL (HORIZONTAL) FLOW INTO REINFORCED (INFILL) SOIL AND RETAINED SOIL ON A SEASONAL BASIS (▽).
2. LATERAL (HORIZONTAL) GROUNDWATER FLOW INTO REINFORCED SOIL WILL OCCUR.
3. THIS COMPLETE DRAINAGE SYSTEM PROVIDES MAXIMUM PROTECTION FOR SRWs AND SHOULD BE UTILIZED WHEN THERE IS UNCERTAINITY AS TO THE ACTUAL SITE GROUNDWATER CONDITONS.



NOTE: CHIMNEY DRAIN AND/OR BLANKET DRAIN MAY BE REPLACED WITH AN APPROPRIATE GEOCOMPOSITE AT THE DISCRETION OF THE WALL DESIGN ENGINEER.

Figure 42. MBW drainage detail.

volume of 0.3 m^3 per m^2 of wall face is recommended for modular units with cores, such as the unit illustrated in figure 43.

Drainage design should be sized to be compatible with the MSE fill soil. Alternately, a geotextile filter may be used to meet filtration requirements, as illustrated in figure 43, if the gravel does not meet filtration criteria. Filtration design of geotextiles is addressed in the FHWA *Geosynthetics Design and Construction Guidelines* along with a review of soil filter criteria.

A drain pipe is normally placed at the bottom of the column of well graded gravel, as illustrated in figure 42, detail A. If a granular soil leveling pad is used in construction, the drain pipe is placed to drain this zone as well.

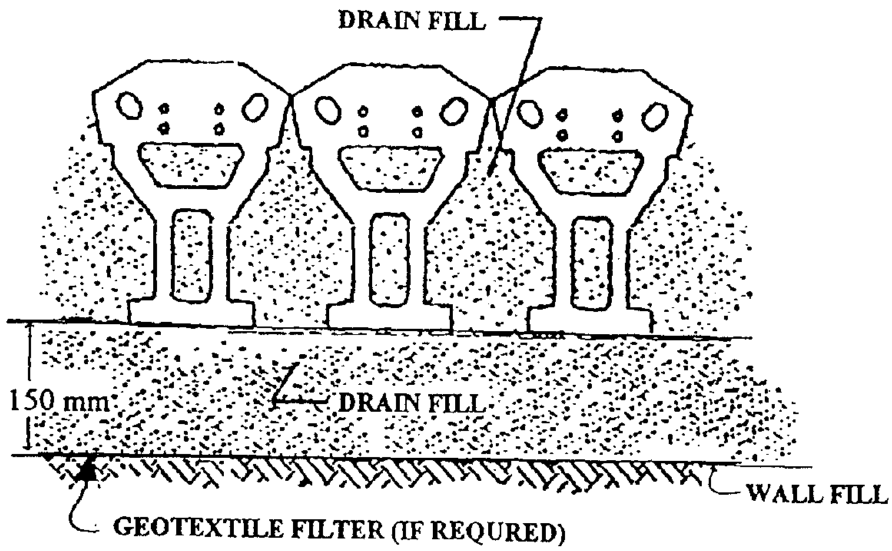


Figure 43. Drain fill placement for MBW with cores or tails.

4.6 DESIGN EXAMPLE

A typical urban highway retaining wall design with inextensible steel linear reinforcements will be illustrated using the sequential design procedure previously outlined.

Step 1: Establish design height, external loads.

- Total design height H = 7.8 m, to gutter grade.
- Required panel height = 7.5 m. vertical.
- Traffic surcharge and barrier required.
  Barrier will be cast integrally to the concrete pavement.
- Traffic surcharge = 9.4 kN/m^2.
- Seismic coefficient = 0.05 g, therefore no seismic design required.

Step 2: Establish engineering properties of foundation soils.

- ϕ' = 30°. (clayey sand, dense)
- Allowable bearing capacity - 300 kPa.
- Differential settlements on the order of 1/300 are estimated.

Step 3: Establish engineering properties for retained and reinforced backfill.

- ϕ = 30°, γ_T = 18.8 kN/m^3 for retained fill.
- ϕ = 34°, γ_T = 18.8 kN/m^3 for reinforced backfill meeting the specifications in chapter 8, section 8.8.
- F^* = 2.0 based on $C_u > 7$.

Step 4: Establish design factors of safety.

- External Stability FS.
  - Sliding = 1.5.
  - Maximum foundation pressure ≤ allowable bearing capacity.
  - Eccentricity $\leq \frac{L}{6}$.
  - Global stability ≥ 1.3.

- Internal Stability FS.
  - Pullout ≥ 1.5.
  - Allowable stress - 0.55 Fy.
  - Design life = 75 years.

Step 5: Choose facing type, reinforcement spacing and type.

- Based on the urban location a precast concrete facing with an architectural finish is required. For aesthetic reasons a maximum panel dimensions of 1.5 x 1.5 m are required with joints no greater than 19 mm. Since the estimated differential settlements along the wall are 1/300, panel joints of 19 mm are acceptable.

- Because of numerous surface drainage obstructions, linear galvanized ribbed strip reinforcements are preferable and used in the design.

- Given the panel size, the most efficient vertical spacing is 0.75 m, allowing for 2 rows of reinforcements per panel. The first row is located 375 mm from the topmost panel plus 300 mm of barrier to pavement grade.

Step 6: Establish preliminary length for reinforcing strips.

- For horizontal backfill slopes, L = 0.7 H is reasonable; therefore:

  L = 0.7 H = 0.7 (7.8) = 5.5 m.

Step 7: **Check external stability for L = 5.5 m.**

- Compute K_a for retained the fill, with a $\phi = 30$ degrees

$$K_a = \tan^2 (45 - \phi/2) = 0.33$$

- Compute sliding FS at base:

$$FS = \frac{V_1 \cdot \tan \phi}{\Sigma F_H}$$

$$V_1 = HL\gamma = 7.8 \cdot 5.5 \cdot 18.8 = 806.5\ kN/m$$

$$V_2 = qH = 9.4 \cdot 5.5 = 51.7\ kN/m$$

$$F_1 = \frac{\gamma H^2 K_a}{2} = 18.8 \cdot \frac{(7.8)^2}{2} \cdot 0.33 = 188.7\ kN/m$$

$$F_2 = qHK_a = 9.4 \cdot 7.8 \cdot 0.33 = 24.2\ kN/m$$

$$FS = \frac{806.5 \tan 30}{212.9} = 2.19 > 1.5$$

- Compute eccentricity at base:

$$e = \frac{L}{2} - \left(\frac{\Sigma M_R - \Sigma M_O}{\Sigma V}\right)$$

with:

$$M_R = V_1 \cdot L/2 + V_2 \cdot L/2 = 2360\ kN/m$$

$$M_O = F_1 \cdot H/3 + F_2 \cdot H/2 = 591\ kN/m$$

$$e = \frac{5.5}{2} - \left(\frac{2360 - 591}{858}\right)$$

$$e = 0.69\ m < \frac{L}{6} = 0.92m$$

- Compute bearing pressure at base

$$\sigma_v = \frac{\Sigma V}{L-2e}$$

$$\sigma_v = \frac{858}{5.5 - (2 \cdot 0.69)} = 208\ kPa < 300\ kPa$$

Step 8: Determine internal stability at each reinforcement level and required horizontal spacing.

- Compute K at each level e.g. at Z = 2.925 m from surface

- $\mathbf{K_a = tan^2 (45 - \phi/2)}$ = 0.28 for reinforced fill

from figure 28, and K at 2.925 m

K = 0.412

- Compute σ_H at this level per unit width

$\boldsymbol{\sigma_v = Z \cdot \gamma + q = 2.925 \cdot 18.8 + 9.4}$

$\boldsymbol{\sigma_v = 64.4\ kPa}$

$\boldsymbol{\sigma_H = \sigma_V \cdot K = 64.4 \cdot 0.412 = 26.5\ kPa}$

The impact barrier will not transfer stress to reinforced volume because it is cast to the concrete pavement structure for the full width of the roadway.

- The horizontal spacing is determined from pullout considerations by using for convenience a length over 2 panel widths centered by the reinforcements at each level rather than a unit length. Therefore, the calculations can be made using a convenient tributary area A_t:

$A_t = S_v$ x 2 panel width $= 0.75 \cdot 2\ (1.5) = 2.25\ m^2$

The maximum force on this defined length or tributary area is:

$$T = \sigma_H \cdot A_t = 26.5 \cdot 2.25 = 59.6 \text{ kN}$$

if pullout FS $\geq$ 1.5 then the resistance P_R is:

$$P_R \geq \sigma_H \cdot A_t \cdot FS = 89.4 \text{ kN}$$

- The number of reinforcing strips, N, required to satisfy the minimum resistance can be calculated from:

$$N = \frac{P_R}{2b \cdot F^* L_e \cdot \sigma_v'}$$

where b = 50 mm
L_e = 3.16 m (see figure 27)
$\sigma'_v = \gamma \cdot z$ (Neglect live load surcharge for pullout)
F^* = 1.35 (Obtained by interpolation from 2.0 at Z = 0 to tan ϕ at 6 m)

$$N = \frac{89.4}{2\ (0.05)\ 1.35\ (3.16)\ 18.8\ (2.925)} = 3.8$$

N = 4 strips per tributary area for FS > 1.5 placed in a row over 2 panels.

- Check stress in reinforcement based on a section loss of E_s subtracted from the nominal thickness of 4 mm. The basis for the thickness losses per year are as follows:

zinc loss = 15 μm (first 2 years)
= 4 μm (thereafter)
steel loss = 12 μm

Service life of zinc coating (86 μm) is:

$$\text{Life} = \frac{86 - 2\ (15)}{4} = 14 \text{ years} + 2 \text{ years} = 16 \text{ years}$$

The base carbon steel will lose section for:

75 years - 16 years = 61 years at a rate of 12 μm/year/side. Therefore, the anticipated loss is:

E_R = 12(59)2	=	1.416 mm and
E_c = 4.000 - 1.416	=	2.584 mm
and the section area	=	129.2 mm^2

If 60 grade steel is used fy = 413.7 MPa

and f_{all} = 0.55 (fy) = 227.5 MPa

The tensile stress in each strip can be calculated from:

$$fs = \frac{T}{N \cdot E_c} = \frac{59.6}{4\ (0.0001292)\ 1000}$$

$$= 115.4 \text{ MPa} < 227.5 \text{ MPa}$$

Calculate internal stability at each layer and determine the number of reinforcing strips per tributary area.

The results for each depth of reinforcement are shown below:

Depth z(m)	Vertical Pressure kPa	K	F^*	Hor. Pressure kPa	N strips per trib. area	Tensile stress/ MPa	FS pullout
0.675	22.09	0.46	1.85	10.27	7	35.75	1.61
1.425	36.19	0.45	1.69	16.18	4	70.44	1.57
2.175	50.29	0.43	1.52	21.59	4	94.01	1.62
2.925	64.39	0.41	1.35	26.51	4	115.42	1.58
3.675	78.49	0.39	1.19	30.93	4	134.65	1.49
4.425	92.59	0.38	1.02	34.85	4	151.72	1.51
5.175	106.69	0.36	0.86	38.27	4	166.61	1.52
5.925	120.79	0.34	0.69	41.19	5	143.47	1.82
6.675	134.89	0.34	0.67	45.76	4	199.24	1.59
7.425	148.99	0.34	0.67	50.55	4	220.06	1.75

CHAPTER 5

DESIGN OF MSE WALLS WITH COMPLEX GEOMETRICS

The basic design methods outlined in chapter 4 considers 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 practical and require consideration during the selection process. They include:

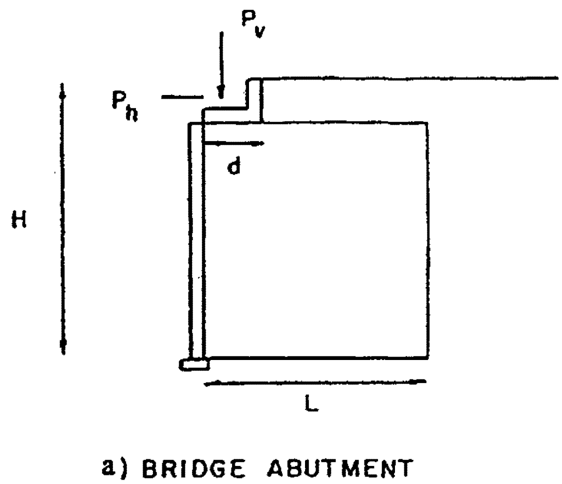
- Bridge abutments.
- Superimposed walls.
- Trapezoidal walls.
- Back-to-back walls.

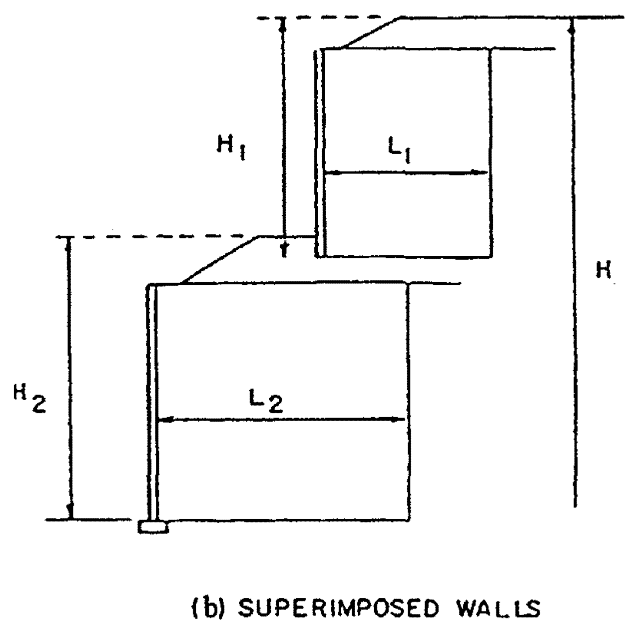
They are illustrated in figure 44.

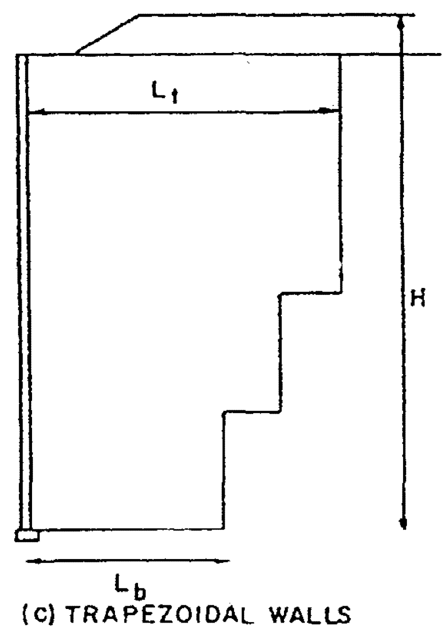
The shape and location of the maximum tensile force line are generally altered by both the geometry and the loads applied on the complex MSE wall structure. It is possible to assume an approximate maximum tensile force line for each; however, supporting experience and analysis are more limited than for rectangular reinforced soil walls.

Moreover, 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, a global stability analysis is generally required for this type of structure. A rough estimate of the global factor of safety could be made using plane failure surfaces; however, the best method is to use a reinforced soil global stability computer method. The procedures detailed in chapter 7 for evaluating RSS embankments could be used to evaluate the global stability of Mechanically Stabilized Earth walls.

The following sections give guidelines for each case.







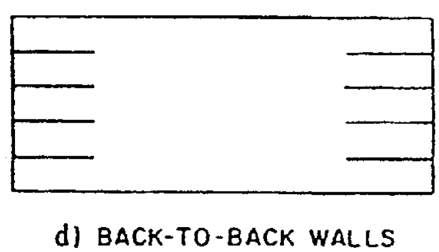


Figure 44. Types of complex MSE structures.

5.1 BRIDGE ABUTMENTS

Bridge abutments have been designed by supporting the bridge beams on a spread foundation constructed directly on the reinforced soil volume, or by supporting a smaller spread footing on deep foundations constructed thru the reinforced volume.

Abutments directly supported on the reinforced volume are more economical, 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, 1992 AASHTO suggests, that tolerable limiting angular distortions (differential settlements) between abutments or between piers and abutments be limited to **one half** of the following angular distortions:

- 0.005 for simple spans.
- 0.004 for continuous spans.

This criteria, suggests that for a 30 m span for instance, differential settlements of 60 mm for a continuous span or 75 mm for a simple span, would be acceptable, with no ensuing overstress and damage to superstructure elements.

a. 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 design procedures for taking account of the surcharge loads for internal stability analysis have been outlined in chapter 4. The same type of procedure is used for the internal stability of bridge abutment structures, calculating the horizontal stress σ_h at each level by the following formula (equation 38):

$$\sigma_H = K(\gamma Z + \Delta\sigma_v) + \Delta\sigma_h$$

where: $\Delta\sigma_v$ is the increment of vertical stress due to the concentrated vertical surcharge P_v, assuming a 2V:1H pyramidal distribution (figure 30).

$\Delta\sigma_h$ is the increment of horizontal stress due to the horizontal loads P_h and calculated as shown in figure 31a, and γZ is the vertical stress at the base of the wall or layer in question due to the overburden pressure.

For large surcharge slabs (with a support width greater than H/3) at the top of reinforced soil wall, the shape of the maximum tensile force line has to be modified to extend to the back edge of the footing, as indicated in figure 45.

Note that in MSEW bridge abutments **inextensible** reinforcements are almost always used because of displacement requirements. However, similar shifts in the maximum tension line to the back of large surcharge slabs 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 slab extends beyond H tan (45 - ϕ/2) from the wall face.

Successful experience with MSEW abutment construction has suggested that the following additional details be implemented:

- Require a minimum offset from the front of the facing to the center line of bridge bearings of 1 m.

- Require a clear distance of 150 mm between the back face of the facing panels and the front edge of footing.

- Where significant frost penetration is anticipated, place the abutment footing on a bed of compacted coarse aggregate, 1 m thick.

- Limit the bearing capacity on the reinforced volume to 200 kPa.

- Use the maximum horizontal force at each reinforcement level, for the design of connections to the panels.

- Extend the density, length and cross-section of reinforcements of the abutment to wingwalls, for a horizontal distance equal to 50 percent of the height of the abutment wall.

- 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).

The balance of the computations remain the same as for any MSE wall as outlined in chapter 4.

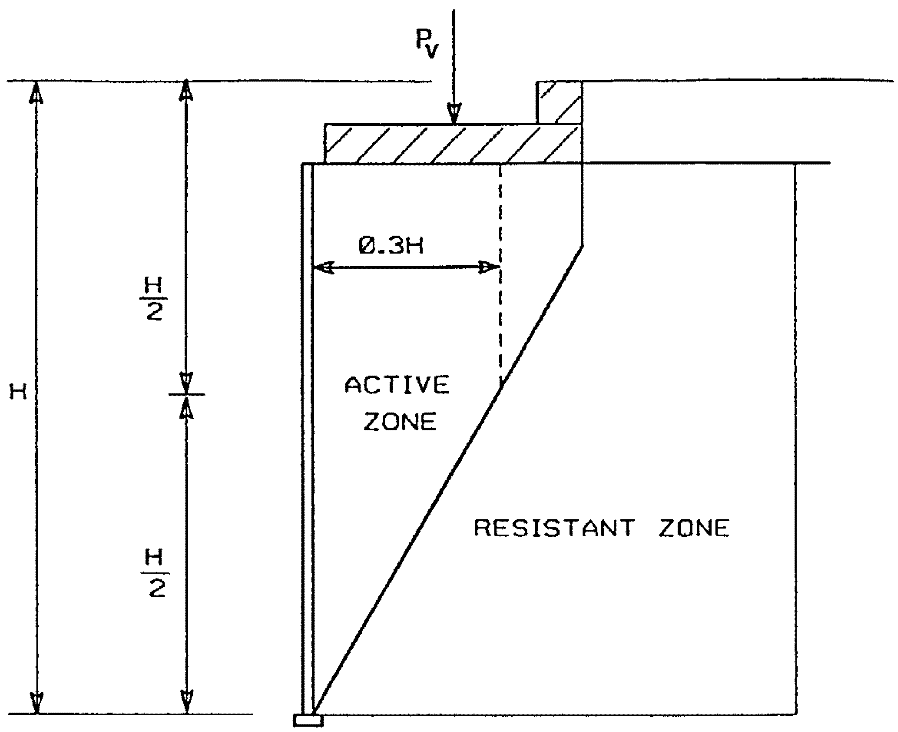


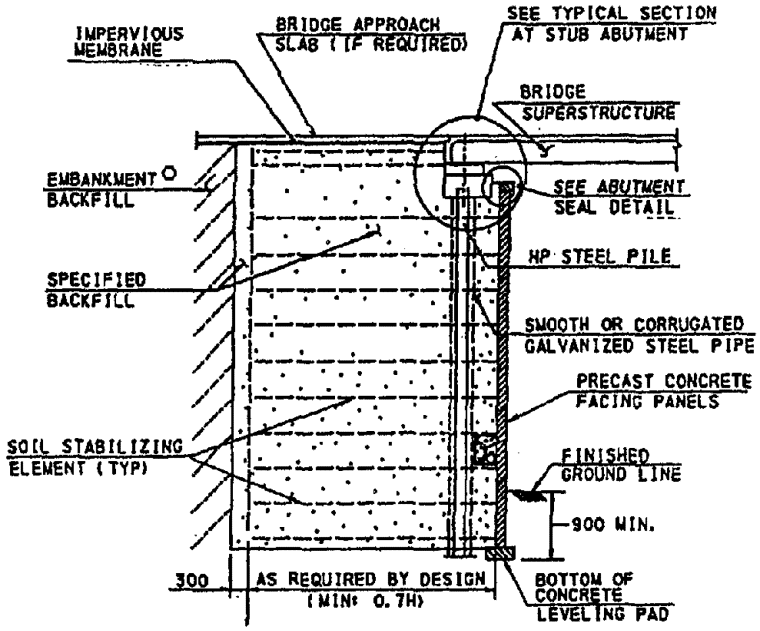
Figure 45. Location of maximum tensile force line in case of large surcharge slabs (Inextensible reinforcements).

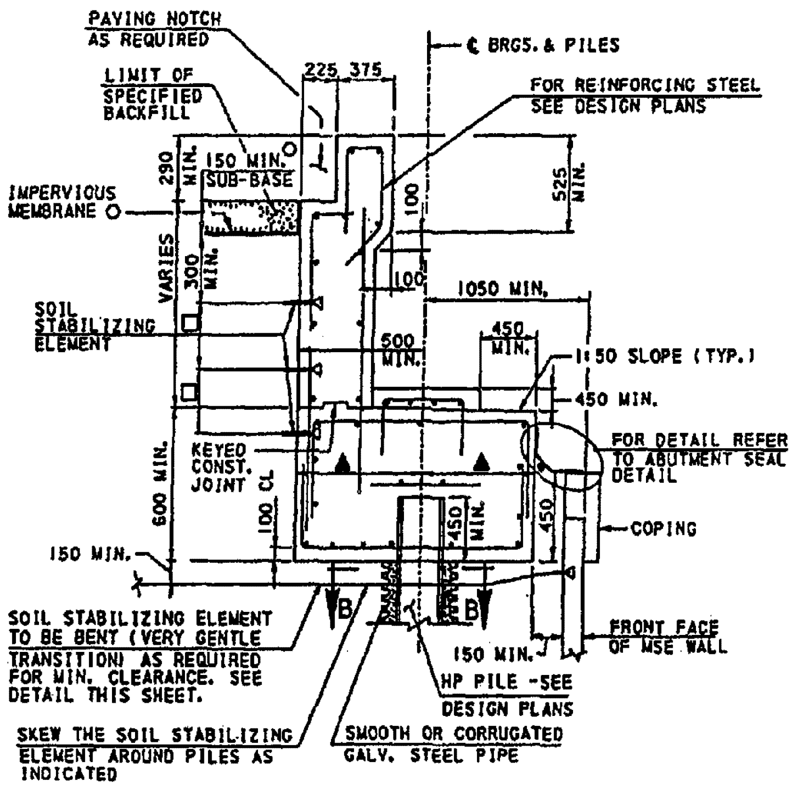
b. MSEW Abutments on Pile Foundations

Where this type of support is chosen, due to construction control, uncertainty or to limit superstructure deflection, the MSE wall is designed with no consideration to the vertical bridge loads, which are transmitted to an appropriate bearing strata by deep foundations. Typically, deep foundations have been steel piles, which are driven prior to MSE wall erection.

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 may be resisted by extending sufficient soil reinforcement (strips, grids) from the back edge of the abutment footing. The resistance is provided by soil/reinforcement interaction. A typical detail is shown on figure 46. Alternately the horizontal forces may be resisted by the pile lateral capacity or by other means.





Note: All dimensions in mm.

Figure 46. Pile supported MSE abutment.

For integral abutments, the horizontal force and its distribution with depth may be developed using pile load/deflection methods (p-y curves) and added as a supplementary horizontal force to be resisted by the wall reinforcements. This force will vary depending on the level of horizontal load, pile diameter, pile spacing and distance from the pile to the back of panels.

The following additional design details have been successfully used:

- Provide a clear horizontal distance of 0.5 m between the back of the panels and the front edge of the pile.

- Provide a casing around piles, thru the reinforced fill, where significant negative skin friction is anticipated.

Where pile locations interfere with the reinforcement, specific methods for field installation must be developed. Simple cutting of the reinforcements is not permissible.

5.2 SUPERIMPOSED WALLS

The design of superimposed MSE walls is made in two steps:

(1) A design using simplified design rules for calculating external stability and locating the internal failure plane for internal stability as shown in figure 47.

(2) A stability analysis, including both mixed and global stability using a reinforced soil global stability computer program outlined in chapter 6. This is an essential computation.

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 = total height

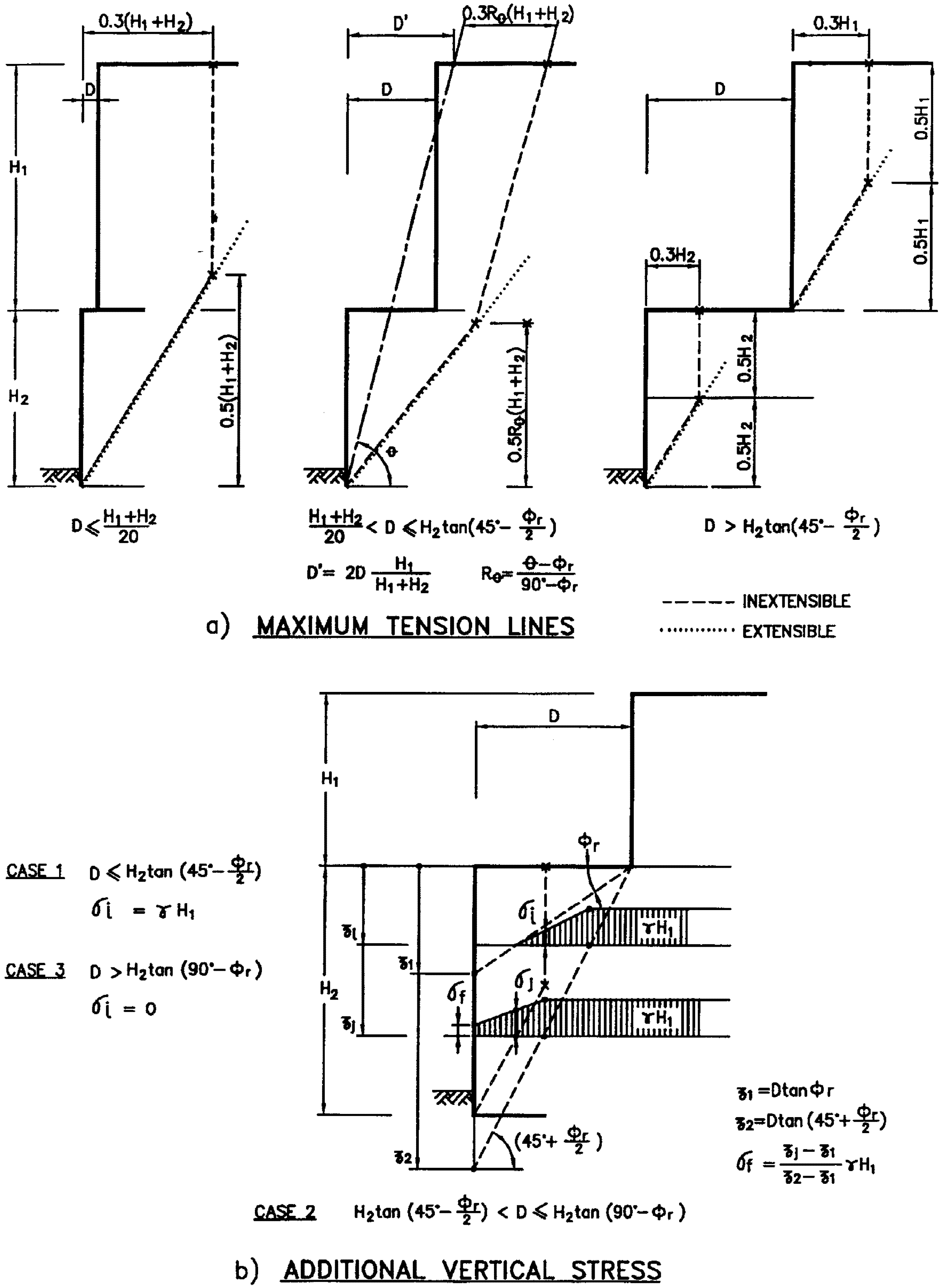


Figure 47. Design rules for superimposed walls.

Where the offset distance (D) is greater than H_2 tan $(90-\phi_r)$, walls are not considered superimposed and are independently designed.

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.

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 in computing bearing pressures. In lieu of a conventional external sliding stability computation, perform a slope stability analysis with failure circles exiting at the base. A factor of safety of 1.5 is generally warranted.

For calculating the internal stability, the maximum tensile force lines are as indicated in figure 47a. These relationships are somewhat empirical and geometrically derived.

For intermediate offset distances, see figure 47a for the location of the failure surface and consider the vertical pressures in figure 47b 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 remain identical as in chapter 4.

5.3 WALLS WITH A TRAPEZOIDAL SECTION

Use of this type of reinforcement geometry should be considered only if the base of the MSE structure is in rock or *very competent foundation soil.*

The design of trapezoidal walls requires two analyses:

(1) A design using simplified design rules for determining external stability.

(2) A global stability analysis, performed using a reinforced soil stability program as outlined in chapter 6.

Simplified design rules for these structures are as follows:

- The wall is represented by a rectangular block (L_o, H) having the same total height and the same cross-sectional area as the trapezoidal section for external stability calculations. See figure 48.

- 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) $\geq$ 0.4 H, 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.

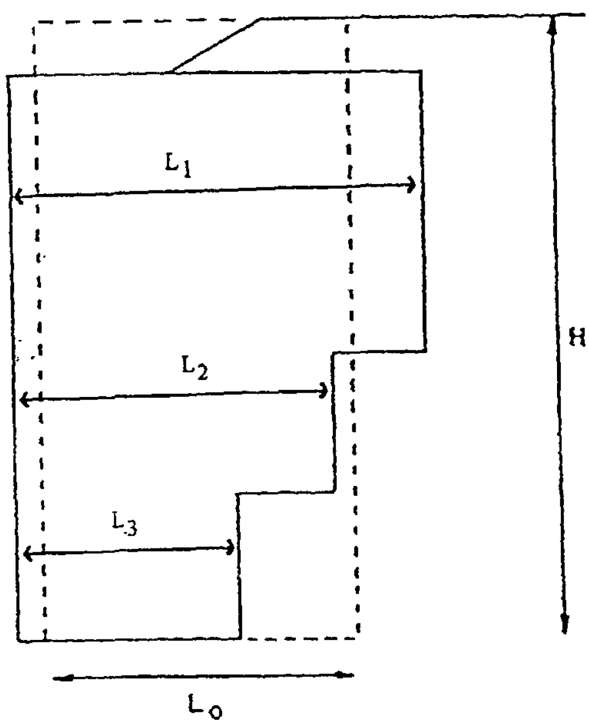


Figure 48. Dimensioning a trapezoidal MSE wall.

5.4 BACK-TO-BACK WALL DESIGN

The back-to-back design has to be considered in the case of a double-faced wall that is actually two separate walls with parallel facings. This situation can lead to a modified value of backfill thrust that influences the external stability calculations. As indicated in figure 49, two cases can be considered.

- 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:

$$D = H \tan (45^\circ - \phi/2) \qquad (55)$$

then the active wedges at the back of each wall cannot fully spread out and the active thrust is reduced. However, it is assumed that for values of:

$$D > H \tan (45^\circ - \phi/2) \qquad (56)$$

full active thrust is mobilized.

For Case II, there is an overlapping of the reinforcements so that the two walls interact. Consequently, the two walls are designed independently for internal stability with the same procedure as in chapter 4 but assuming no active thrust from the backfill.

Considering this case, some engineers might be tempted to use single reinforcements connected to both wall facings. This alternative completely changes the strain patterns in the structure and results in higher reinforcement tensions so that the design method in this manual is no longer applicable. In addition, difficulties in maintaining wall alignment could be encountered during construction, especially when the walls are not in a tangent section.

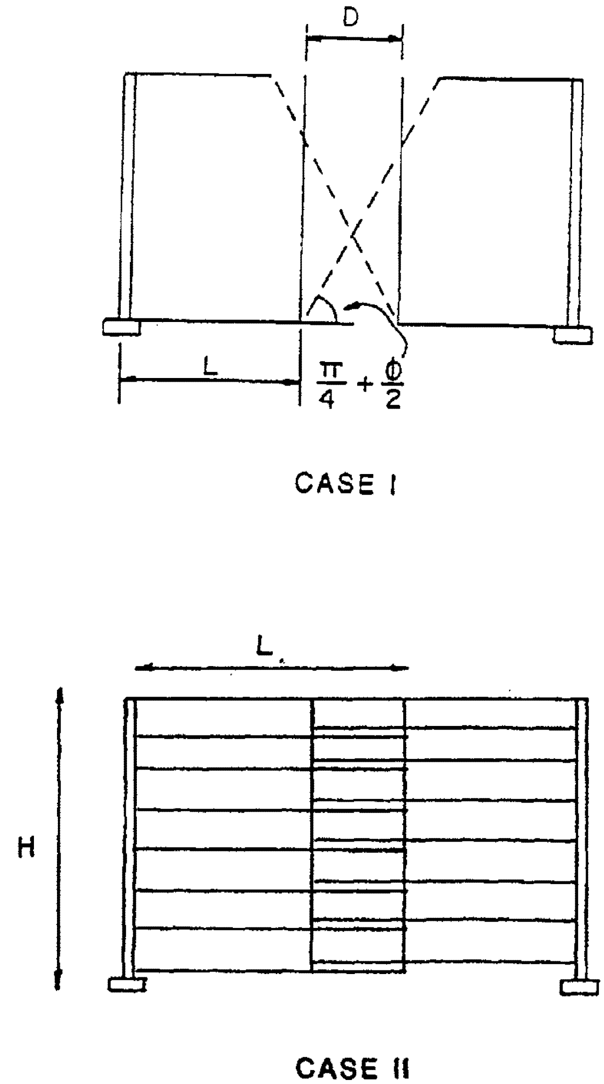


Figure 49. Back-to-back wall.

5.5 DETAILS

At abutment locations, the permeation of salt-laden runoff through the expansion joints could result in a chloride rich 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 50.

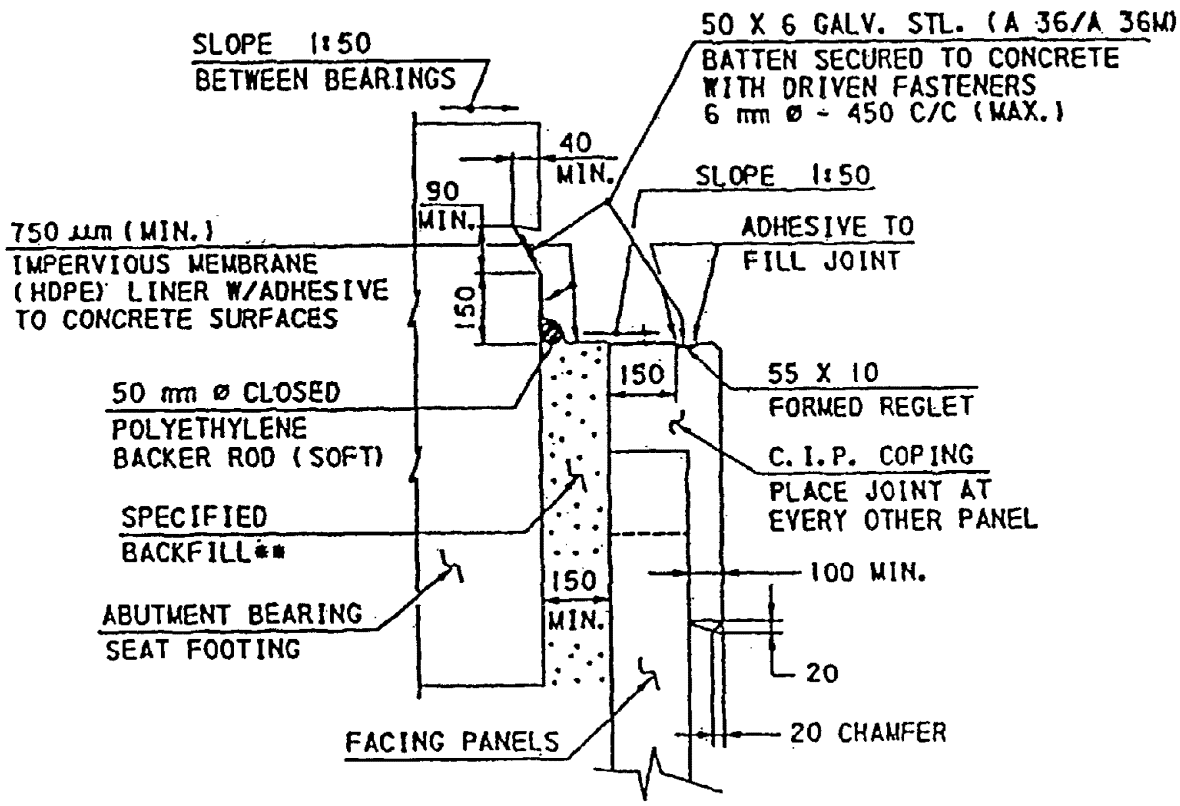


Figure 50. Abutment seat detail.

5.6 DESIGN EXAMPLE, BRIDGE ABUTMENT

A bridge abutment design as an alternate to a conventional abutment, will be illustrated using the sequential design procedures outlined in chapter 4. The bridge is at the end of the retaining wall in the example for chapter 4, and the same MSE system will be used.

Step 1: Establish design height, external loads.

- Total height, H = 9.7 m
- Facing wall height, H = 7.5 m
- Traffic surcharge, q = 9.4 kN/m^2 (0.5 m)
- Distance from front face to centerline of bearing = 1.0 m (Minimum recommended)
- Bridge vertical dead load = 45 kN/m
- Bridge vertical live load = 50 kN/m
- Bridge horizontal load = 2.25 kN/m

Step 2: Establish engineering geotechnical properties.

- Foundations : ϕ = 30°, q_a = 300 kPa (clayey sand, dense)
- Reinforced Fill : ϕ = 30°, γ = 18.8 kN/m^3
- Reinforced Fill : ϕ = 34°, γ = 18.8 kN/m^3
  q_a = 200 kPa
  F* = 2.0

Step 3: Establish design factor of safety.

- Design life = 75 years (If critical application, increase to 100 years).
- External Stability FS

- Sliding $\geq$ 1.5
- Eccentricity $\leq$ L/6
- Maximum foundation pressure $\leq$ allowable

- Internal Stability FS
  - Pullout $\geq$ 1.5
  - Allowable stress - 0.55 fy

Step 4: **Choose facing type, reinforcement spacing and type.**

- The project is at the same location as in the example for chapter 4. Therefore, precast panels 1.5 x 1.5 m and galvanized steel ribbed strips will be used at a vertical spacing of 0.75 m.

Step 5: **Establish preliminary length for reinforcing strips.**

- For abutments 0.7 (H_{TOTAL}) should be sufficient; 0.7 (9.7) = 6.8 m, use 7 m as production is in one half meter length increments.

Step 6: **Size abutment footing.**

- With a minimum distance of 1.0 m from the front face to the centerline of bearing, the sizing shown on figure 51 appear reasonable as a first try. Taking a unit weight of concrete at 23.6 kN/m^3, the following can be computed per unit length.

V_1 = 23.01 kN
V_2 = 2.83 kN
V_3 = 13.69 kN
DL = 45 kN
LL = 50 kN
F_S = 6.89 kN
F_1 = 15.17 kN
F_2 = 2.25 kN

GEOMETRY OF M.S.E. ABUTMENT AND APPLIED FORCES

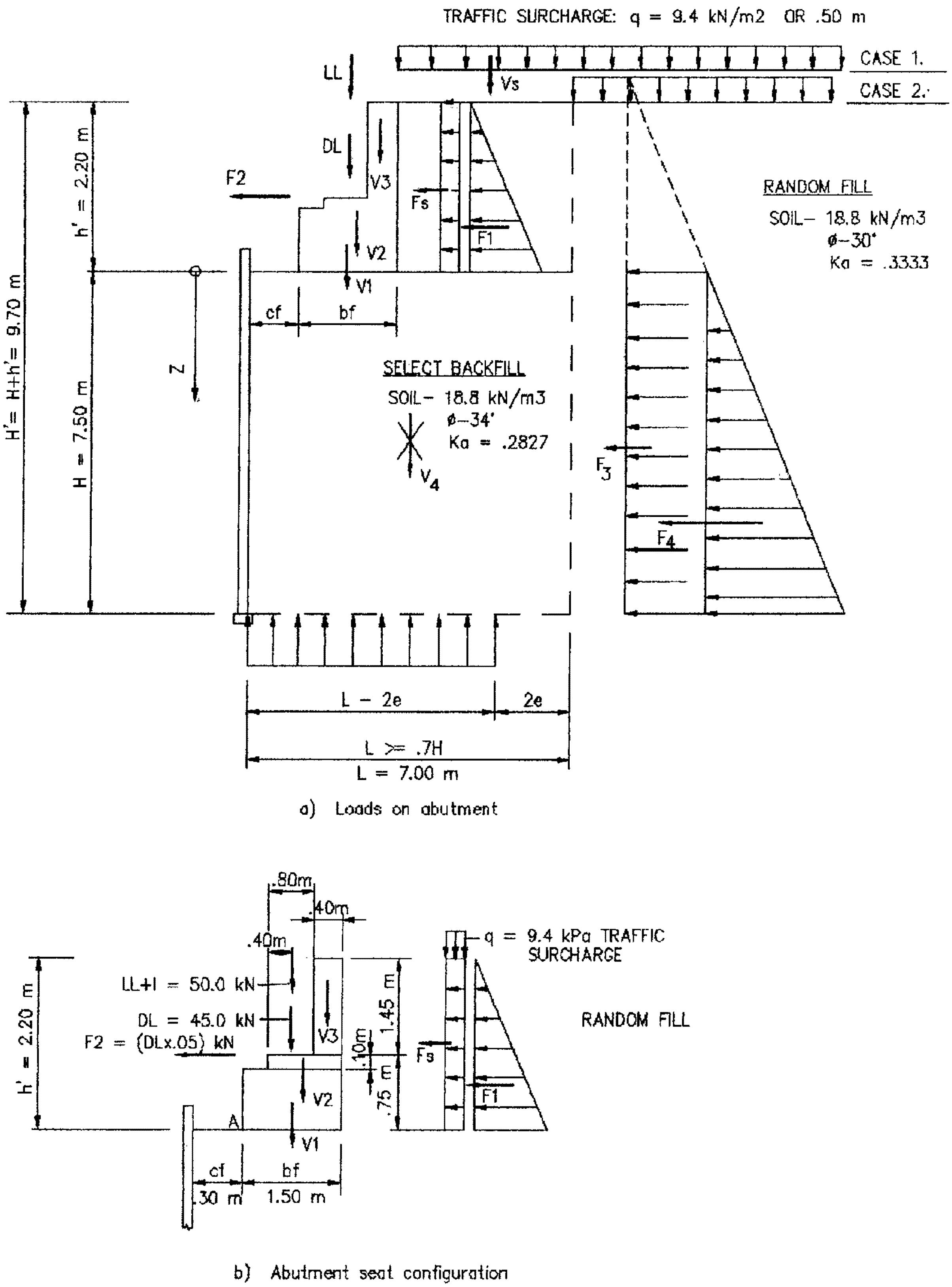


Figure 51. MSE abutment design example.

- Check sliding, eccentricity and bearing pressure for the abutment footing.

$$FS_{sliding} = \frac{(\Sigma V_A - L\ L)\ \tan 34^o}{\Sigma F}$$

$$= \frac{(134.53\ -\ 50.0)\ 0.6745}{24.31}$$

$$FS_{sliding} = 2.35 > 1.5$$ <u>ok</u>

$$e' = \frac{b_f}{2} - \frac{\Sigma M_R - \Sigma M_O}{\Sigma V}$$

$$e' = \frac{1.50}{2} - \frac{104.1\ -\ 20.39}{134.53}$$

$$e' = 0.13 < \frac{b_f}{6} = 0.25$$ <u>ok</u>

$$\sigma_v = \frac{\Sigma V}{b_f - 2e'} = \frac{134.53}{1.5\ -\ (2 \cdot 0.13)}$$

$$\sigma_v = 108.5\ \ kPa < 200\ kPa$$ <u>ok</u>

Step 7: Check external stability with a reinforcement length of 7 m.

Refer to figure 51 for loads and distances:

V_4	= 987 kN	-	Reinforced volume
V_s	= 65.8 kN	-	Traffic surcharge
V_5	= 289.52 kN	-	Wt. of soil block above reinforced volume
F_4	= 176.23 kN	-	Horizontal earth pressure force component
F_3	= 126.89 kN	-	Horizontal earth pressure force component
ΣF_a	= 24.31 kN	-	from abutment seat
ΣV_A	= 134.53 kN	-	from abutment seat including DL and LL.
ΣM_{RA}	= 104.10 kN·m	-	from abutment seat including DL and LL.
l_1	= 2.9 m	-	see figure 31a

- Compute net load P′ by removing the soil weight in at abutment footing area:

$$P' = \Sigma V_A - (h' + q) \cdot (b_f + c_f) \cdot \gamma$$
$$= 134.53 - [(2.20 + 0.05)(0.3 + 1.5)\, 18.8]$$
$$= 43.16\, kN$$

- Compute ΣM_R and ΣM_o about B as follows:

$$\Sigma M_R = \frac{L}{2}\,(V_4 + V_5 + V_s) + P' \cdot (b_f + \Sigma M_{RA} / \Sigma V_A)$$
$$= \frac{7.0}{2}\,(987.0 + 289.52 + 65.80) + 43.16\,[0.30 + \frac{104.10}{134.53}]$$
$$= 4744.47\ kN \cdot m$$

$$\Sigma M_o = F_4 \cdot \frac{H}{3} + F_3 \cdot \frac{H}{2} + \Sigma F_A\,(H - \frac{l_1}{3})$$
$$= 176.23 \cdot \frac{7.5}{3} + 126.89 \cdot \frac{7.5}{2} + 24.31\,(7.5 - \frac{2.9}{3})$$
$$= 1075.25\ kN \cdot m$$

Separating the surcharge moment:

$$M_s = V_s \cdot \frac{L}{2} = 65.8 \cdot \frac{7.0}{2} = 230.3\ kN \cdot m$$

- therefore, taking moments about B at the base level and subtracting the surcharge moment for the worst case:

$$e = \frac{L}{2} - \frac{\Sigma M_R - (\Sigma M_O - M_S)}{\Sigma V - V_S}$$

$$e = \frac{7.0}{2} - \frac{4744.47 - (1075.25 - 230.3)}{1385.48 - 65.8}$$

$$e = 0.89 < \frac{7.00}{6} = 1.17 \qquad \underline{ok}$$

- Compute bearing pressures at the foundation level:

$$\sigma_v = \frac{1385.48}{7 - (2 \cdot 0.89)}$$

$$\sigma_v = 265.42\ kPa < 300\ kPa \qquad \underline{ok}$$

- Check sliding FS:

$$FS_{sliding} = \frac{[\Sigma V - V_S\ ]\ \tan 30^o}{\Sigma F}$$

$$= \frac{(1385.48\ -\ 65.8)\ 0.577}{327.43}$$

$$FS\ = 2.33 > 1.50 \qquad \underline{ok}$$

tep 8: Determine internal stability at each reinforcement level and required horizontal spacing.

- Compute coefficient of earth pressure at each level e.g. at 4.825 m from the top of backwall or 2.625 m from the top of the MSE wall.

K = 0.367 (see figure 28)

- Compute vertical soil pressure at depth of 2.625 m from top of MSE wall.

$$\sigma_{VS} = \gamma\ (z + h' + q) = 18.8\ (2.625 + 2.20 + 0.5)$$

$$= 100.11\ kPa$$

- Compute vertical pressure from abutment footing. (See figure 30)

$$\sigma_{VA} = \frac{43.16}{[\ (1.5 - 2\ (0.13)\ ]\ +\ (\frac{2.625}{2} + 0.3)}$$

$$= 15.13\ kPa$$

- Determine supplemental horizontal pressure (see figure 31a) at level z_i = 2.625 m.

$$l_1 = (b_f + c_f - 2e') \tan (45 + \phi/2)$$
$$= (1.5 + 0.3 - 2 \cdot 0.13) \tan 62 = 2.9m$$

- at level z_i therefore:

$$\Delta \sigma_H = \frac{2 F_A (l_1 - z_i)}{l_1^2} = \frac{2 \cdot 24.31 (2.9 - 2.625)}{(2.9)^2}$$

$$\Delta \sigma_H = 1.59 \ kPa$$

- Compute horizontal pressure at the 2.625 m level.

$$\sigma_H = (\sigma_{VS} \cdot K) + (\sigma_{VA} \cdot K) + \Delta \sigma_H$$
$$= (100.11 \cdot 0.367) + (15.13 \cdot 0.367) + 1.59$$
$$\sigma_H = 43.84 \ kPa$$

Step 9: Determine required reinforcement at 2.625 m level based on a defined length of 2 panels in length and spacing S_v.

- Determine force on the tributary area:

$$T = 43.84 \cdot 2.25 = 98.64 \ kN$$

- Determine the effective length L_e:

$$L_e = L - 0.3 H' = 7 - (0.3 \cdot 9.7) = 4.9 \ m$$

- Determine number of strips required to satisfy pullout criteria:

$$N = \frac{T \cdot FS}{2b \cdot F^* \cdot L_e \cdot \sigma_v}$$
$$= \frac{98.64 \cdot 1.5}{2 \cdot 0.05 \cdot 0.934 \cdot 4.09 \cdot 18.8 (2.625 + 2.20)}$$
$$N = 4.27 \ use \ 5 \ strips$$

Place 3 strips in upper row, 2 in lower row.

- For design life of 75 years.

$$E_c = 4.00 - 1.416 = 2.584\ mm$$

- Maximum stress in each strip is:

$$fs = \frac{T}{N \cdot E_c} = \frac{98.64}{5\ (0.0001292)\ 1000}$$

$$= 152.7\ MPa < 227.5\ MPa \qquad \underline{ok}$$

Calculate internal stability at each layer and determine the number of reinforcing strips per tributary area. The tabulated results are as follows:

Depth z(m)	**Vertical Pressure kPa**	**K**	**F***	**Hor. Pressure kPa**	**N strips per trib. area**	**Tensile stress/ MPa**	**FS pullout**
0.375	82.73	0.4199	1.4311	49.32	6	143.15	1.53
1.125	92.40	0.4023	1.2655	47.42	5	165.18	1.52
1.875	103.40	0.3846	1.0998	45.70	5	159.18	1.68
2.625	115.22	0.3669	0.9341	43.89	5	152.87	1.75
3.375	127.56	0.3493	0.7684	44.55	5	155.18	1.82
4.125	140.28	0.3393	0.6745	47.59	5	165.75	1.86
4.875	153.25	0.3393	0.6745	51.99	5	181.08	2.08
5.625	166.42	0.3393	0.6745	56.46	5	196.64	2.29
6.375	179.73	0.3393	0.6745	60.98	5	212.38	2.51
7.125	193.16	0.3393	0.6745	65.53	6	190.20	3.26

CHAPTER 6

REINFORCED (STEEPENED) SOIL SLOPES PROJECT EVALUATION

6.1 INTRODUCTION

Where limited 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 later in chapter 7. Section 6.2 reviews the types of systems and the materials of construction. Section 6.3 provides a discussion of the internal stability design approach for use of reinforcement as compaction aids, steepening slopes and slope repair. Computer assisted methods for internal stability evaluation are also reviewed. The section concludes with a discussion of external stability requirements. Section 6.4 reviews the construction sequence. Section 6.5 covers treatment of the outward face of the slope to prevent erosion. Section 6.6 covers design details of appurtenant features including traffic barrier and drainage considerations. Finally, section 6.7 presents several case histories to demonstrate potential cost savings.

6.2 REINFORCED SOIL SLOPE SYSTEMS

a. Types of Systems

Reinforced soil systems consist of planar reinforcements arranged in nearly horizontal planes in the backfill 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, many of which have proprietary computer programs.

This manual does not cover reinforcing the base section of an embankment for construction over soft soils, a different type reinforcement application. The user is referred to the FHWA *Geosynthetics Design and Construction Guidelines* for that application.

b. 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 design with cohesion added by the reinforcement (which is not covered in this manual).

- **Backfill Requirements.** Backfill requirements for reinforced soil slopes are discussed in chapter 3. Because a flexible facing (e.g. wrapped facing) is normally used, minor distortion at the face that may occur due to backfill settlement, freezing and thawing, or wetting and drying can be tolerated. Thus, lower quality backfill than recommended for MSE walls can be used. The recommended backfill is limited to low-plasticity, granular material (i.e., PI $\leq$ 20 and $\leq$ 50 percent finer than 0.075 mm). However, with good drainage, careful evaluation of soil and soil-reinforcement interaction characteristics, field construction control, and performance monitoring (see chapter 9), most indigenous soil can be considered.

6.3 DESIGN APPROACH

a. Use Considerations

As reviewed in chapter 2, there are two main purposes for using reinforcement in slopes as follows:

- 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: The factor of safety must be adequate for both the short-term and long-term conditions and for all possible modes of failure.

As illustrated in figure 52, 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 mass.
- Compound, where the failure surface passes behind and through the reinforced soil mass.

In some cases, the calculated stability safety factor can be approximately equal in two or all three modes.[3]

b. 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 in a continuous plane along the edge of the slope (see figure 4b). Only narrow strips, about 1.2 to 2 m in width, at 0.3 to 0.5 m 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 7. Where reinforcement is required by analysis, the geosynthetic may be considered as secondary reinforcement used to improve compaction and stabilize the slope face between primary reinforcing layers.

c. 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 53:

- Circular or wedge-type potential failure surface is assumed.

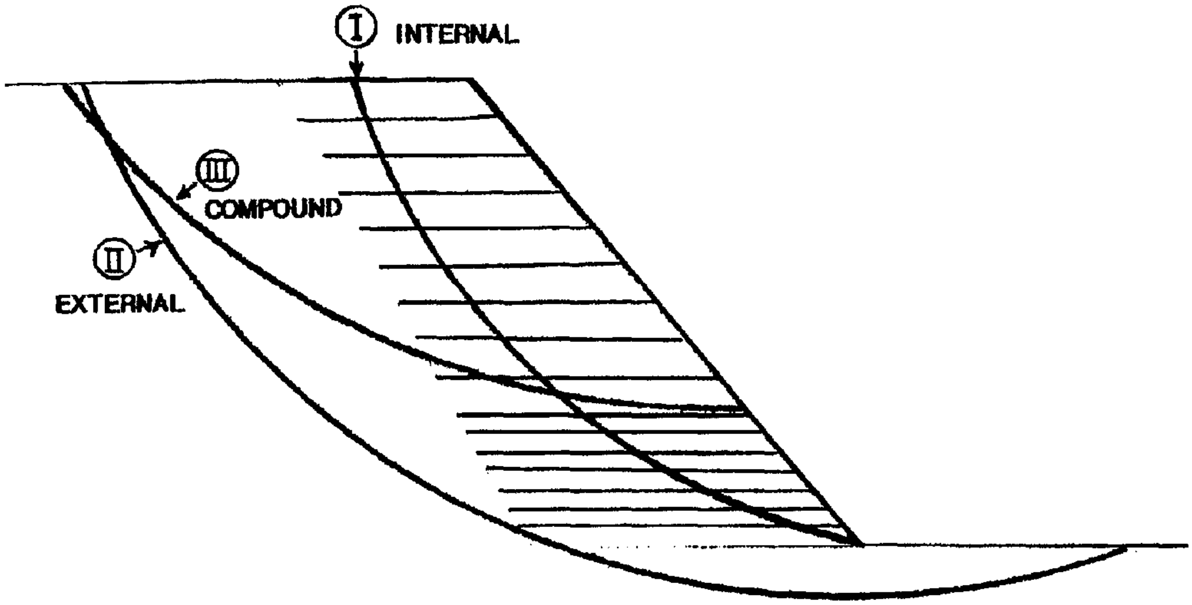


Figure 52. Failure modes for reinforced soil slopes.

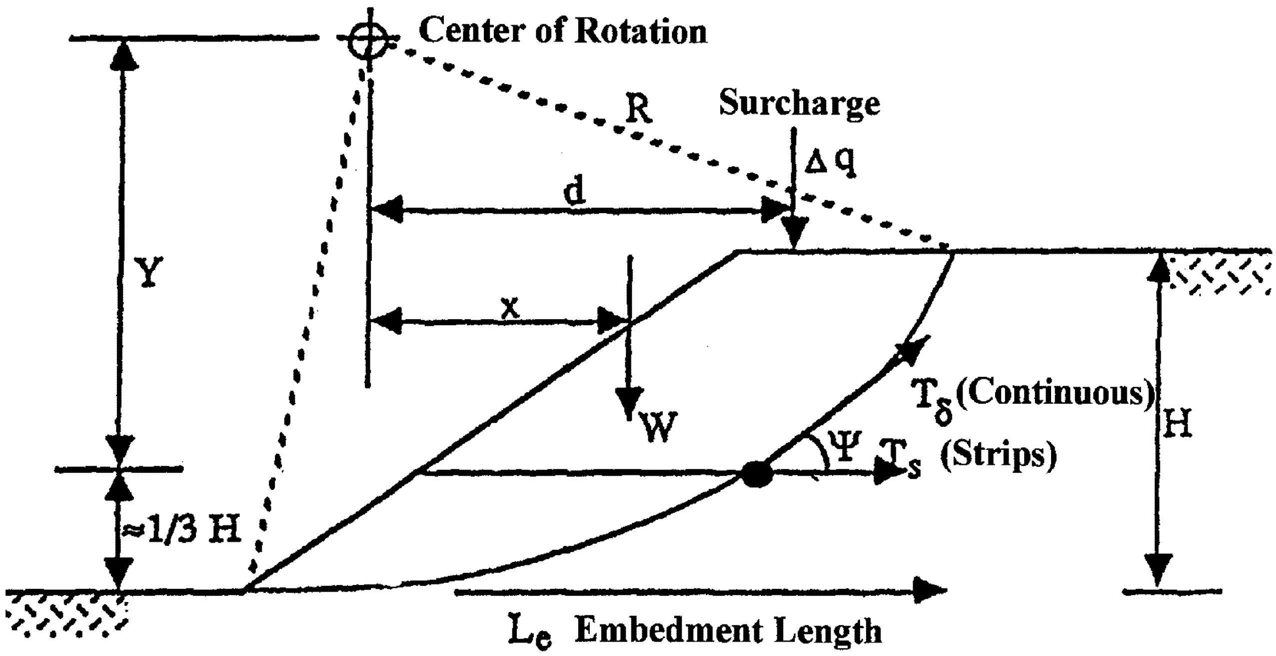


Figure 53. Modified limit equilibrium analysis for reinforced slope design.

- 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.

As shown in figure 52, a wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. The critical slope stability factor of safety is taken from the unreinforced failure surface requiring the maximum reinforcement. This is the failure surface with the largest unbalance 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 slopes is performed by determining the factor of safety with successively modified reinforcement layouts until the target factor of safety is achieved.

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 53 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., Bishop, Janbu).

The limit equilibrium approach is suitable for slopes up to 70 degrees at which point lateral earth pressure tends to control design. Thus slopes steeper than 70 degrees are defined as walls and 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. Tensile force direction is, therefore, dependent on the extensibility of the reinforcements used, and the following inclination is normally suggested:

- Inextensible Reinforcements: T parallel to the reinforcements.

- Extensible Reinforcements: T tangent to the sliding surface.

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 seen that an increase in normal stress will occur for reinforcements with an orientation other than tangential to the failure surface.[4] 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 inextensible 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.

d. 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 would also include the confinement effects of the reinforcement on the shear strength of the soil in the vicinity of the reinforcement.

Several reinforced slope programs are commercially available. These programs generally 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 or layout. Most of the programs are limited to simple soil profiles and, in some cases, the reinforcement layouts. Also, external stability evaluation is generally limited to specific soil and reinforcement conditions. In some cases, the programs are reinforcement-specific. These programs could be used to provide a preliminary evaluation or to check a detailed analysis. Examples include:

PCSTABL6	-	Purdue University
STABGM	-	Virginia Tech
XSTABL	-	University of Idaho
UTEXAS2	-	US Army COE
UTEXAS3	-	University of Texas
New Janbu	-	Tensar
Strata Slope	-	Strata Systems
Tenslo1	-	Tensar

A generic program developed by FHWA for both reinforcement design and evaluation of almost any condition will be reviewed following the design methodology presentation. The program is based on the design method presented in *Reinforced Soil Structures Volume I. Design and Construction Guidelines*, FHWA-RD-89-043 and chapter 7.

e. Evaluation of External Stability

The external stability of a reinforced soil mass depends on the ability of the mass to act as a stable block and withstand all external loads without failure. Failure possibilities as shown in figure 54 include 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 mass must be sufficiently wide at any level to resist 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 Manual*, (1993).

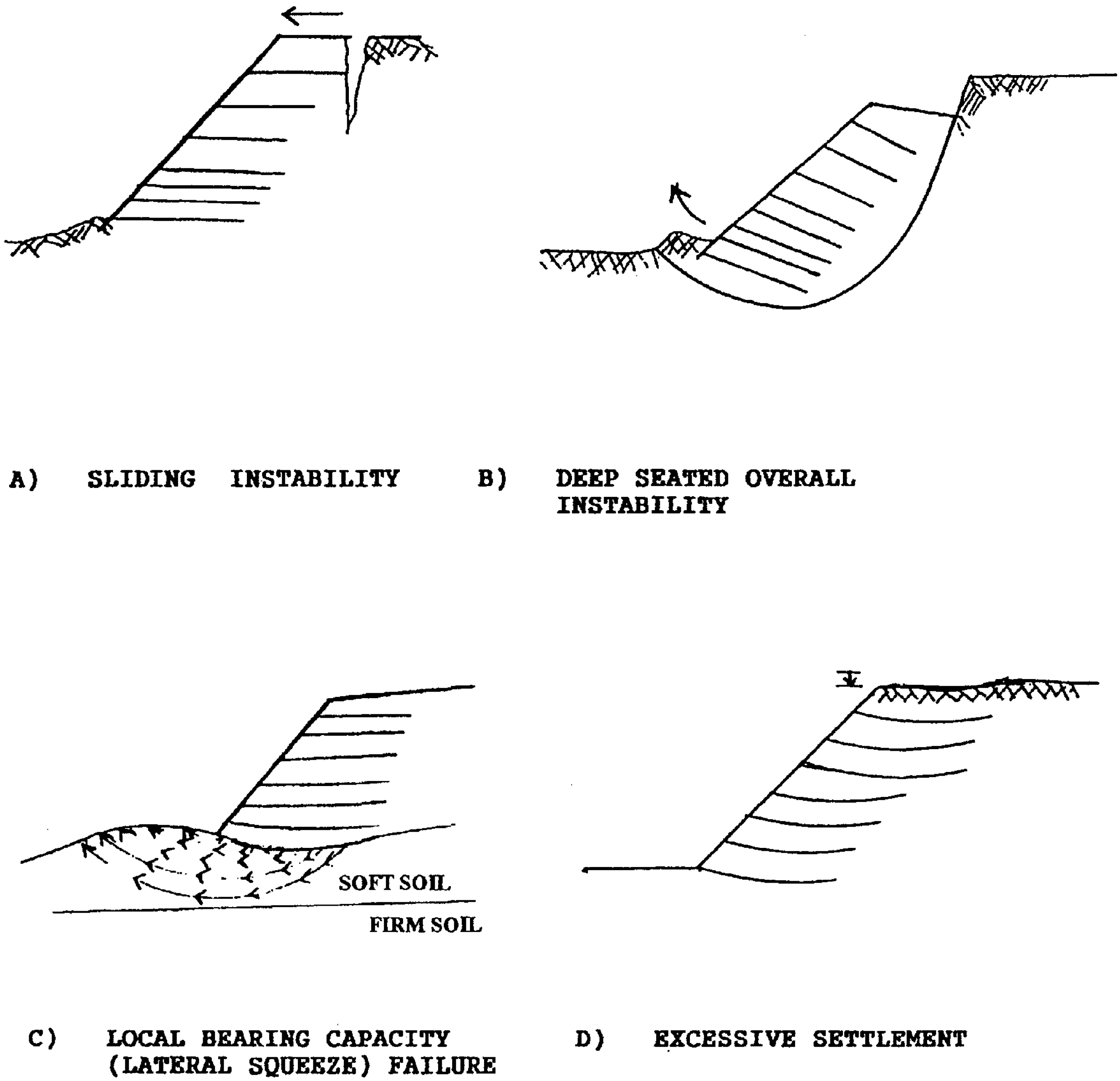


Figure 54. External failure modes for reinforced soil slopes.

Conventional soil mechanics stability methods should also be used to evaluate the global stability of the reinforced soil mass. 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 soils behind or below the reinforced soil mass. Compound failure surfaces initiating externally and passing through or between reinforcement sections should also be evaluated, especially for complex slope or soil conditions. Evaluation of potential seepage forces is especially critical for global stability analysis.

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 stresses 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, Silvestri, and Bonaparte, Giroud, and Holtz are appropriate. [5, 6, 7] The approach by Silvestri is demonstrated in example problem 2 in chapter 7.

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 effect decisions on facing elements as discussed previously in chapter 2.

In areas subject to potential seismic activity, a simple pseudo-static type analysis should be performed using a seismic coefficient obtained from the current AASHTO Specification, Division 1A, or using local practice. Reinforced slopes are flexible systems and unless used for bridge abutments they are not laterally restrained. Thus it is appropriate to use A = A/2 for seismic design in accordance with the AASHTO code.

If any of the external stability safety factors are less than the required factor of safety, the following 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 and improvement of the foundation soils.

- Embed the slope below grade (> 1 m), 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.)

6.4 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 55:

- 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.

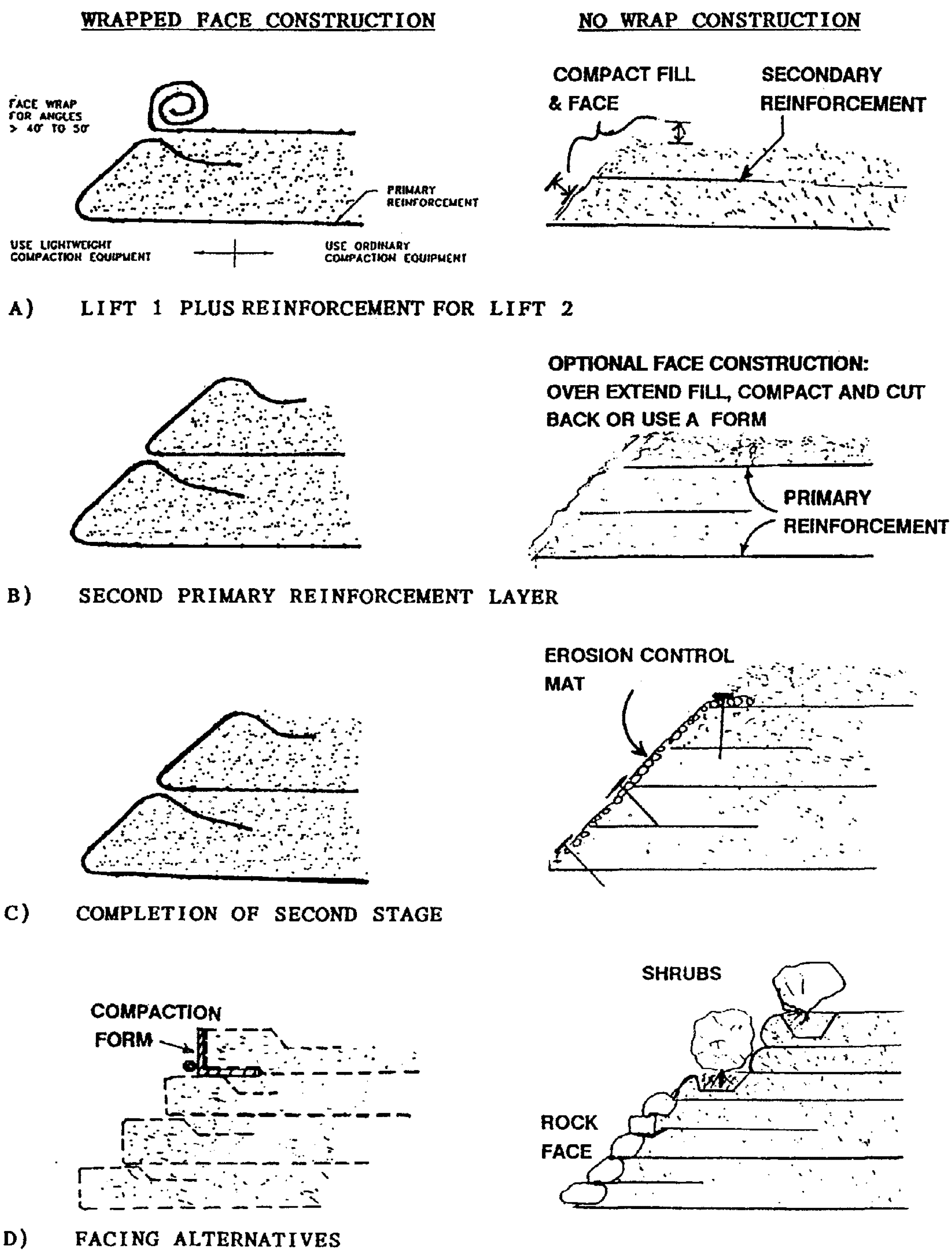


Figure 55. Construction of reinforced soil slopes.

- 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 150 mm is recommended along the edges perpendicular to the slope for wrapped face structures. Alternatively, with geogrid reinforcement, the edges may be clipped or tied together. When geosynthetics are not required for face support, no overlap is required and edges should be butted.

- Reinforcement Backfill 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 150 mm of fill between the reinforcement and the wheels or tracks of construction equipment.

    - 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 backfill material, care should be taken to avoid any deformation or movement of the reinforcement.

    - Use lightweight compaction equipment near the slope face to help maintain face alignment.

- Compaction Control

    - Provide close control on the water content and density of the backfill. It should be compacted at least 95 percent of the standard AASHTO T99 maximum density within 2 percent of optimum moisture.

- If the backfill is a coarse aggregate, then a relative density or a method type compaction specification should be used.

- Face Construction

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 is not required for slopes up to 1H:1V, if the reinforcement is maintained at close spacing (i.e., every lift or every other lift but no greater than 400 mm). In this case, the reinforcement can be simply extended to the face. For this option, a facing treatment as detailed under Treatment of Outward Face, should be applied at sufficient intervals during construction to prevent face erosion.

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 1 m into the embankment below the next reinforcement layer (see figure 55).

- For steep slopes, form work may be required to support the face during construction, especially if lift thicknesses of 450 to 600 mm or greater are used.

- For geogrids, a fine mesh screen or geotextile may be required at the face to retain backfill materials.

- Additional Reinforcing Materials and Backfill Placement

If drainage layers are required, they should be constructed directly behind or on the sides of the reinforced section.

6.5 TREATMENT OF OUTWARD FACE

a. Vegetation

Stability of a slope can be threatened by erosion due to surface water runoff. Erosion rills and gullies can lead to surface sloughing and possibly deep-seated failure surfaces.

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. Geosynthetic reinforced slopes inherently can be difficult sites to establish and maintain vegetative cover due to the steep grades that can be achieved. The steepness of the grade limits the amount of water absorbed by the soil before runoff occurs. Once vegetation is established on the face, it must be protected to ensure long-term survival. Although root penetration should not affect the reinforcement, the reinforcement will most likely restrict root growth. This can have an adverse influence on the growth of some plants. Maintenance issues, such as mowing, must be carefully considered in the selection of vegetation. Guidance should be obtained from maintenance and regional landscaping groups in the selection of the most appropriate low maintenance vegetation.

b. Armored

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. A permanent facing such as gunite or emulsified asphalt may be applied to a RSS slope face to provide long-term ultra-violet protection, if the geosynthetic UV resistance is not adequate for the life of the structure.

- Geosynthetic Erosion Control Mats

  A synthetic (permanent) erosion control mat may be used as a permanent facing. The mat must be stabilized against ultra-violet light and inert to naturally occurring soil-born chemicals and bacteria. The erosion control mat serves to: 1) protect the bare soil face against erosion until vegetation is established; 2) reduce runoff velocity for increased water absorption by the soil, thus promoting long-term survival of the vegetative cover; and 3) reinforce the root system of the vegetative cover. Maintenance of vegetation may be required, and should be considered when choosing the type of cover.

- Other permanent facing elements may include riprap, 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.

6.6 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.

a. 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. This does not significantly impair the overall strength of the geosynthetic and no adjustments in the design are required. Metallic reinforcements may require special consideration, such as installation of post during backfilling or use of cantilever type guardrail systems.

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. of 1992 AASHTO *Standard Specifications for Highway Bridges* or as addressed in the 1989 AASHTO *Roadside Design Guide*.

b. 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 mass will decrease stability against sliding failure.

- Uncontrolled seepage into the reinforced mass will increase the weight of the reinforced mass and may decrease the shear strength of the soil, and decreasing stability.

- Seepage through the mass can reduce pullout capacity of the geosynthetic at the face and increase soil weight, creating erosion and sloughing problems.

Drains are typically placed at the rear of the reinforced soil mass to control subsurface water seepage as detailed in chapter 7. Surface runoff should also be diverted at the top of the slope to prevent it from flowing over the face.

c. Obstructions

If encountered in a design, guidance provided in chapter 4 should be considered.

6.7 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 reference which, in most cases, contains additional details.

a. The Dickey Lake Roadway Grade Improvement Project [8]

Dickey Lake is located in northern Montana approximately 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. MDOT 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 embankments(s) were developed by MDOT, with the plans indicating the desired finished geometry. The slopes generally ranged from 9 m to 18 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 grids 4.6 to 18.3 m long and spaced 0.6 to 1.2 m vertically throughout the reinforced embankment. The ultimate strength of the primary reinforcement was on the order of 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 1.5 m with a vertical spacing of 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 7.

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 $180/m^2 of vertical face and has been periodically monitored by visual inspection and slope inclinometers. Project photos are shown in figure 56. 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.

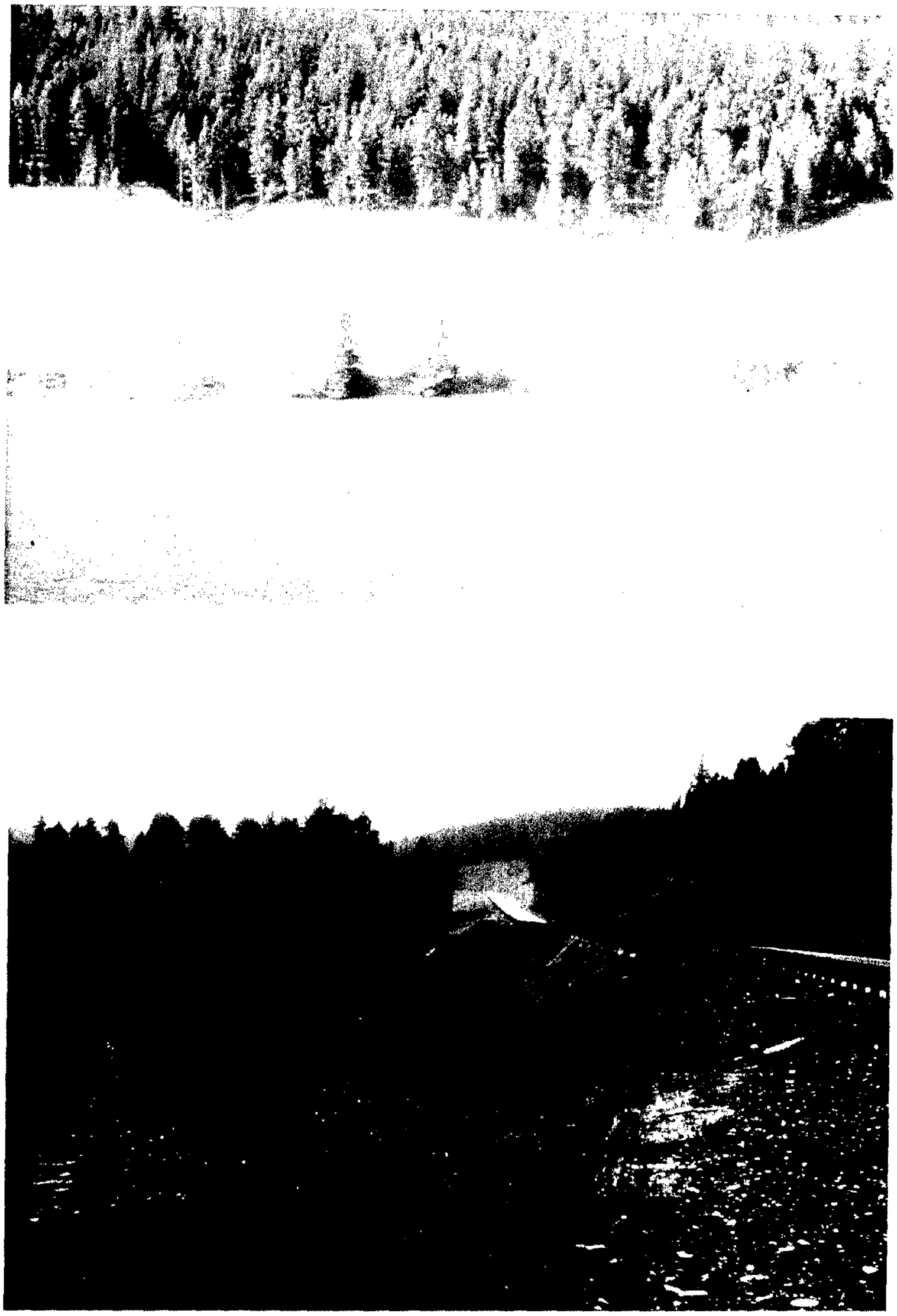


Figure 56. Dickey Lake site.

b. Salmon-Lost Trail Roadway Widening Project [9]

As part of a highway widening project in Idaho, the Federal Highway Administration designed and supervised the construction of a 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 a recent article 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 backfill. An important factor in the design was to deal with seeps or weeps coming out of the existing slope. Geotextile reinforcements with an inplane tranmissivity 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 6 and 7 of this manual. The final design consisted of two reinforced zones with a constant reinforcing spacing of 0.3 m. The reinforcement in the lower zone had an ultimate tensile strength of 100 kN/m, and the reinforcement in the upper zone had a reinforcement strength of 20 kN/m. The reinforcement strength was reduced based on partial reduction factors which were reviewed in chapter 3. Field tests were used to reduce the reduction factor for construction damage from 2.0 to 1.1 at a substantial savings to the project (40 percent reduction in reinforcement).

The construction was completed in 1993 (see figure 57 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.

Figure 57. Salman Lost Trial site.

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 $\$160/m^2$ of vertical face. MSE wall costs in other areas of the site were on the order of $\$240/m^2$ of vertical face for similar or lower heights.

c. Cannon Creek Alternate Embankment Construction Project (10)

A large embankment was planned to carry Arkansas State Highway 16 over Cannon Creek. The proposed 77,000 m^3 embankment had a maximum height of 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 soils could be used with geosynthetic reinforcement. Both options were bid as alternatives and the geosynthetic option was used in construction (see figure 58). The reinforcement used was a high-density polyethylene geogrid with a reported wide-width strength of 100 kN/m. The geogrid reinforcement option was estimated to be $200,000 less expensive than the gravelly soil fill option.

d. Pennsylvania SR 54 Roadway Repair Project (11)

During the winter of 1993 - 1994, a sink hole 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 15-m-high embankment. The native soil, a sandy clay, was deemed an unsuitable backfill soil 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 $\$11.50/m^3$. 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 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.

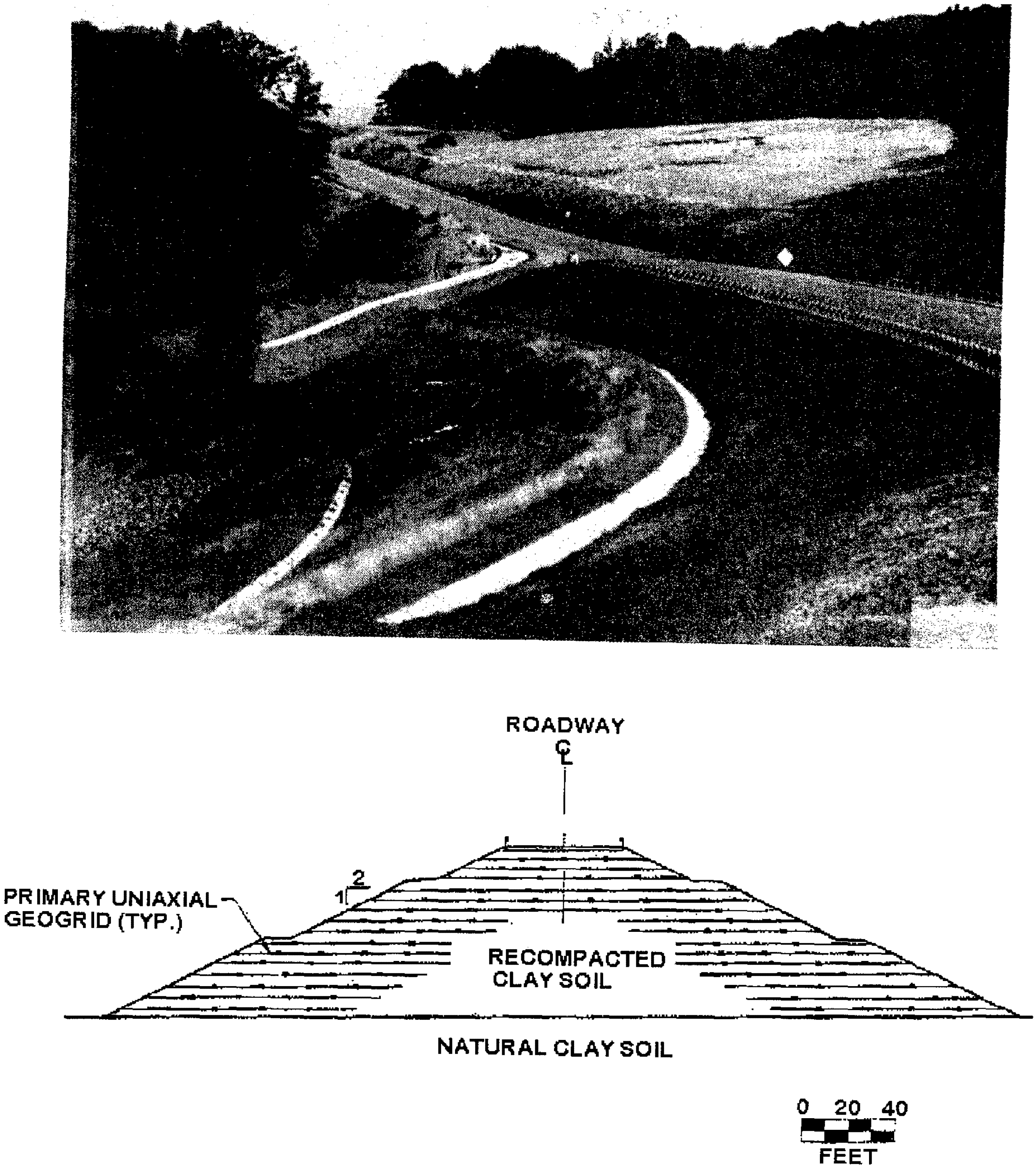


Figure 58. Cannon Creek project.

With the geotextile placed at a compacted lift spacing of 0.3 m full pore pressure dissipation was provided 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 0.3 m lift intervals, the effective drainage path was reduced from the full height of the slope (15 m) to 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 16 kN/m and placed at every lift (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 were used to confirm 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 ($\pm$ 1 percent strain). Project photos are shown in figure 59.

In-place costs of the geotextile, along with the on-site fill cost for the project, were \$70,000, resulted in a savings of \$199,000 over the select-fill alternative. Additional savings resulted from not having to remove the on-site soils from the project site.

Figure 59. Pennsylvania SR54.

CHAPTER 7

DESIGN OF REINFORCED SOIL SLOPES

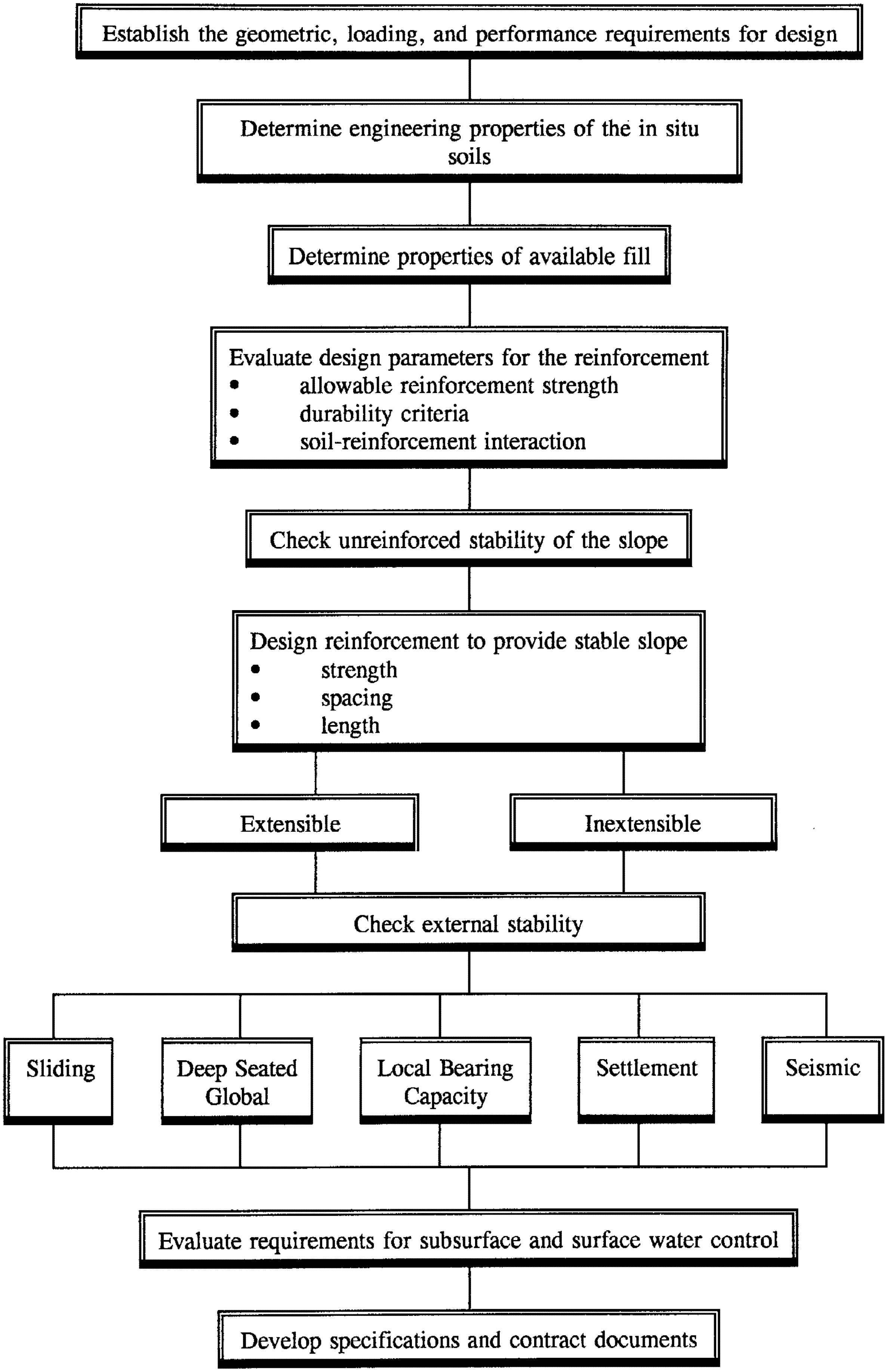
7.1 INTRODUCTION

This chapter provides step-by-step procedures for the design of reinforced soil slopes. 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 (1995)* in cases where the stability of the foundation is at issue.

As indicated in chapter 6, there are several approaches to the design of reinforced steepened slopes. The method presented in this chapter uses the classical rotational, limit equilibrium slope stability method as was shown in figure 53. 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 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 following flow chart shows the steps required for design of reinforced soil slopes.


Establish the geometric, loading, and performance requirements for design
Determine engineering properties of the in situ soils
Determine properties of available fill
Evaluate design parameters for the reinforcement
• allowable reinforcement strength
• durability criteria
• soil-reinforcement interaction
Check unreinforced stability of the slope
Design reinforcement to provide stable slope
• strength
• spacing
• length
Extensible
Inextensible
Check external stability
Sliding
Deep Seated Global
Local Bearing Capacity
Settlement
Seismic
Evaluate requirements for subsurface and surface water control
Develop specifications and contract documents

7.2 REINFORCED SLOPE DESIGN GUIDELINES

The design steps outlined in the flow chart are as follows:

Step 1. Establish the geometric, loading, and performance requirements for design.

a. Geometric and loading requirements (see figure 60).

- 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*).

- Traffic Barrier

  - See article 2.7 of 1992 AASHTO *Standard Specifications for Highway Bridges* and AASHTO 1989 *Roadside Design Guide*.

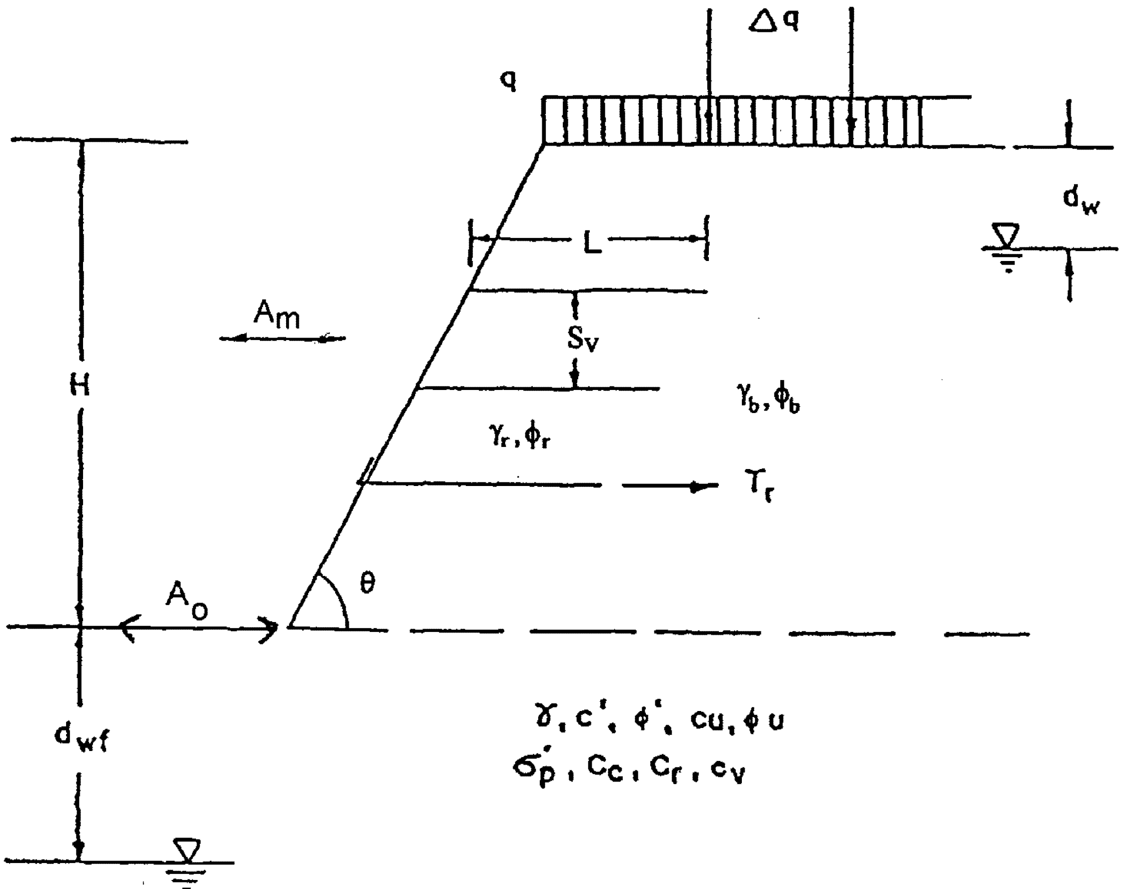
b. Performance requirements.

- 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.



Notations:

H = slope height
θ = slope angle
T_r = strength of reinforcement
L = length of reinforcement
S_v = vertical spacing of reinforcement
q = surcharge load, q
Δq = temporary live load, Δq
A_m = design seismic acceleration
d_w = depth to the ground water table in slope
d_{wf} = depth to the 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
A_O = ground acceleration coefficient
g = acceleration due to gravity

Figure 60. Requirements for design of reinforced soil slopes.

Step 2. **Determine the engineering properties of the in situ soils.** (see recommendations in chapter 3, section 3.4.)

- 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 ground water table d_w, and piezometric surfaces.
- For failure repair, identify location of previous failure surface and cause of failure.

Step 3. **Determine the properties of reinforced fill and, if different, the retained fill.** (see recommendations in chapter 3, section 3.4.)

- 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´.
- Chemical composition of soil (pH).

Step 4. **Evaluate design parameters for the reinforcement.** (see recommendations in chapter 3, section 3.4.)

- Allowable geosynthetic strength, T_a = ultimate strength (T_{Ult}) $\div$ reduction factor (RF) for creep, installation damage and durability:

For granular backfill meeting the recommended gradation in chapter 3, and electrochemical properties in chapter 3, RF = 7, may be conservatively used for preliminary design and routine, noncritical structures where the minimum test requirements outlined in table 9 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!

- Pullout Resistance: (See recommendations in chapter 3 and Appendix A.)
  - F.S. = 1.5 for granular soils.
  - Use F.S. = 2 for cohesive soils.
  - Minimum anchorage length, L_e, = 1 m.

Step 5. CHECK UNREINFORCED STABILITY.

see discussion in Chapter 6.

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 Manual*, 1993) 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., the STABL family of programs developed for the Federal Highway Administration at Purdue University including the current version, STABL4M, and the program XSTABL developed at the University of Idaho. 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 61.

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. For such cases, a more extensive foundation analysis is warranted, and foundation improvement measures should be considered as reviewed in chapter 6.

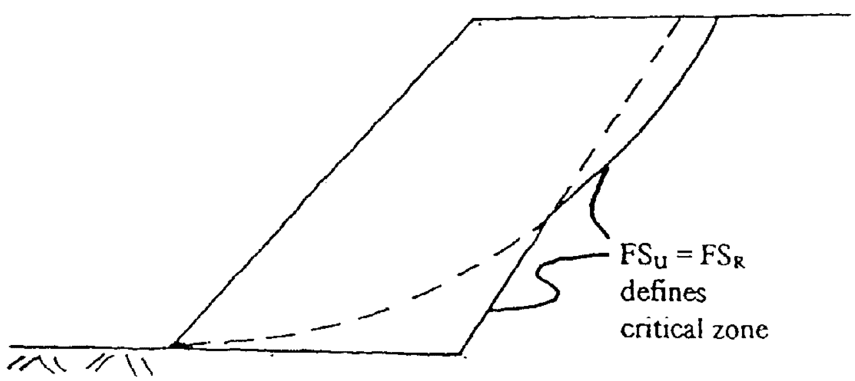


Figure 61. Critical zone defined by rotational and sliding surface that meet the required safety factor.

Step 6. **Design reinforcement to provide a stable slope.** (see figure 62, and discussion in chapter 6.)

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 equation:

$$T_S = (FS_R - FS_U) \frac{M_D}{D} \tag{57}$$

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.

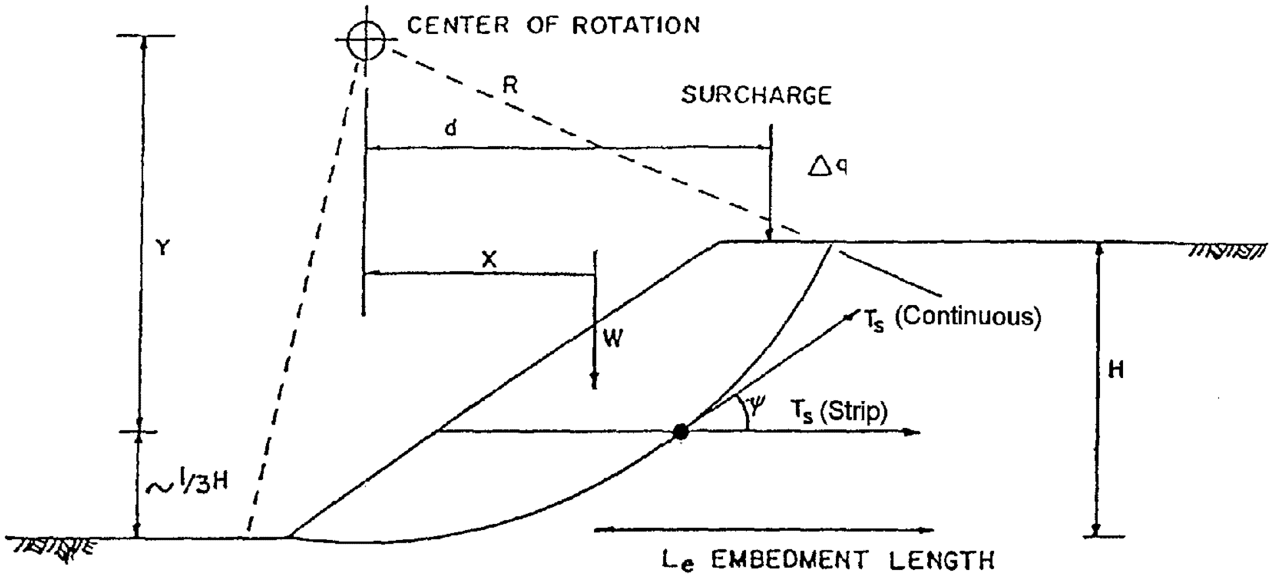
= radius of circle R for **extensible reinforcement** (i.e., assumed to act tangentially to the circle).

= radius of circle R for **continuous sheet 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 inextensible 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.

- $T_{S\text{-}MAX}$ the largest T_S calculated and establishes the total design tension.



Factor of safety of unreinforced slope:

$$F.S._u = \frac{\text{Resisting Moment } (M_R)}{\text{Driving Moment } (M_D)} = \frac{\int_0^{L_{SP}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)}$$

where: W = weight of sliding earth mass
L_{SP} = length of slip plane
Δq = surcharge
τ_f = shear strength of soil

Factor of safety of reinforced slope:

$$F.S. = F.S._u + \frac{T_s \cdot D}{M_D}$$

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 reinforcement

Figure 62. Rotational shear approach to determine required strength of reinforcement.

- **Note: the minimum safety factor usually does not control the location of $T_{S\text{-}MAX}$; the most critical surface is the surface requiring the largest magnitude of reinforcement.**

b. Determine the total design tension per unit width of slope, $T_{S\text{-}MAX}$, using the charts in figure 63 and compare with $T_{S\text{-}MAX}$ 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 50.

- Figure 63 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 63 is not intended to be a single design tool. Other design charts available from the literature could also be used.[12,13,14,15] As indicated in chapter 6, 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 ($H \leq 6$m) assume a uniform reinforcement distribution and use $T_{S\text{-}MAX}$ to determine spacing or the required tension T_{max} requirements for each reinforcement layer.

- For high slopes ($H > 6$ m), divide the slope into two (top and bottom) or three (top, middle, and bottom) reinforcement zones of equal height and use a factored $T_{S\text{-}MAX}$ in each zone for spacing or design tension requirements (see figure 64). The total required tension in each zone is found from:

For 2 zones:

$$T_{Bottom} = 3/4\ T_{S\text{-}MAX} \qquad (58)$$

$$T_{Top} = 1/4\ T_{S\text{-}MAX} \qquad (59)$$

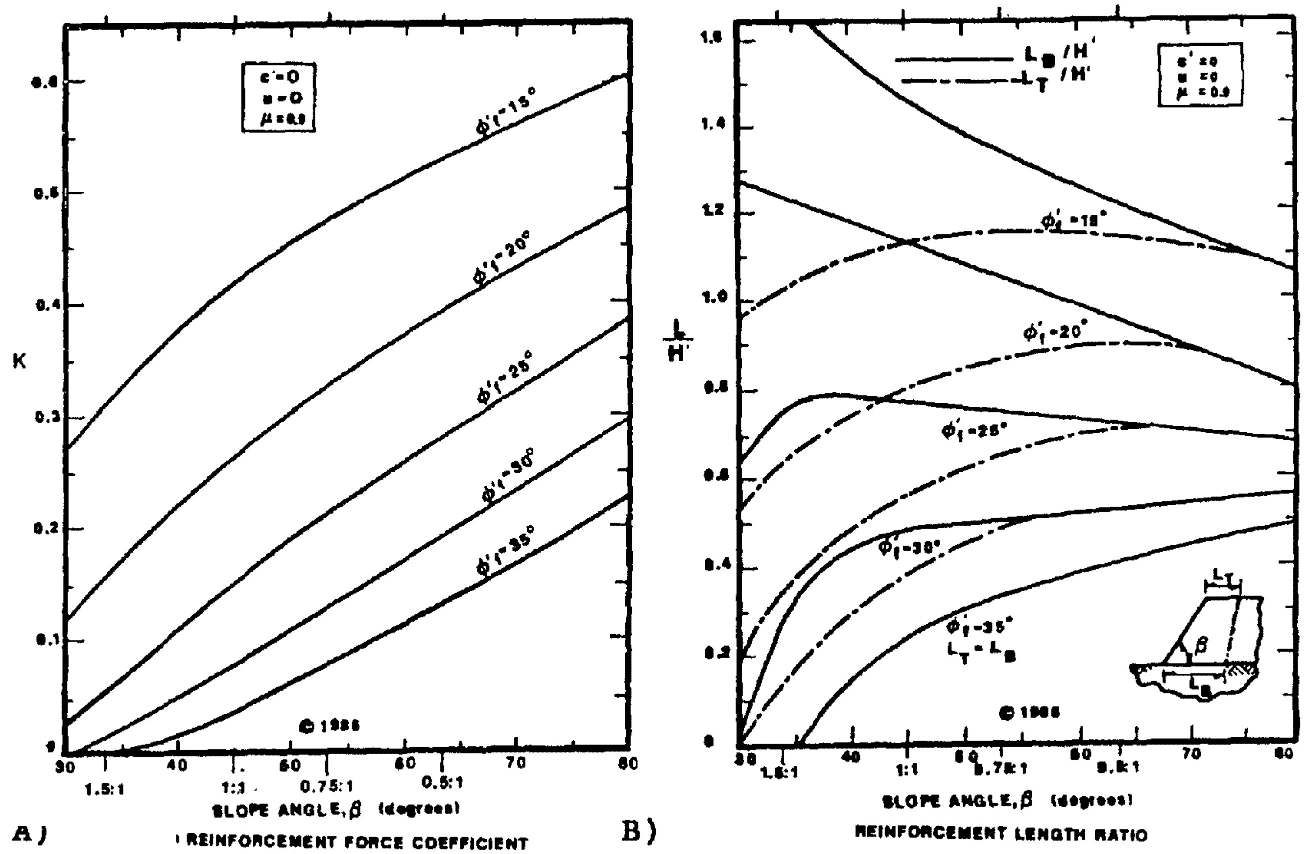


CHART PROCEDURE:

1. Determine force coefficient K from figure A above where:

$$\phi_f = \tan^{-1}\left(\frac{\tan \phi_r}{FS_R}\right)$$

where: ϕ_r = friction angle of reinforced fill

2. Determine:

$$T_{s\text{-}max} = 0.5\,K\,\gamma_r\,H'^2$$

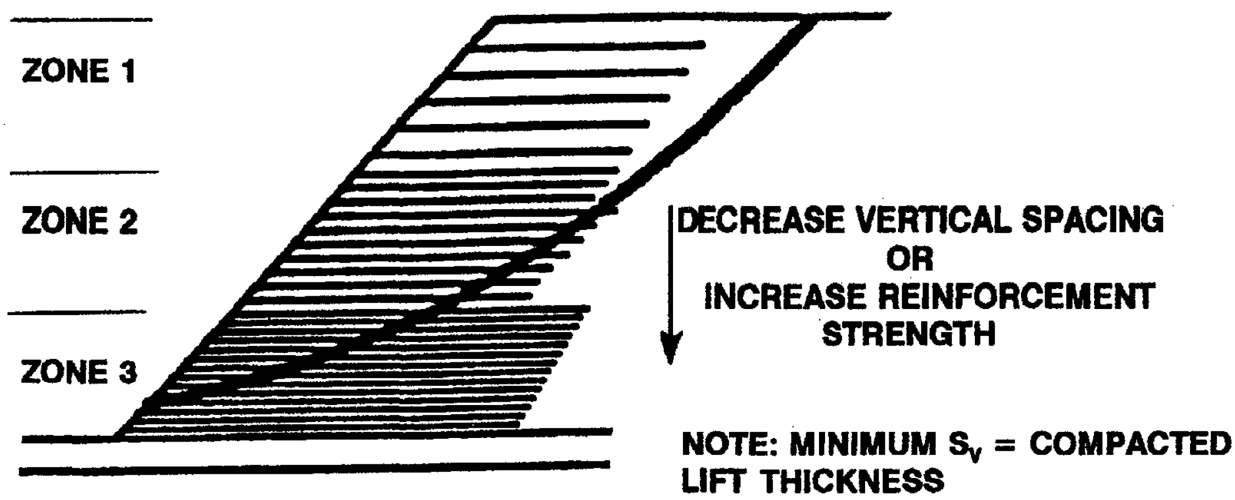
where: $H' = H + q/\gamma_r$
q = a uniform surcharge

3. Determine the required reinforcement length at the top L_T and bottom L_B of the slope from figure B.

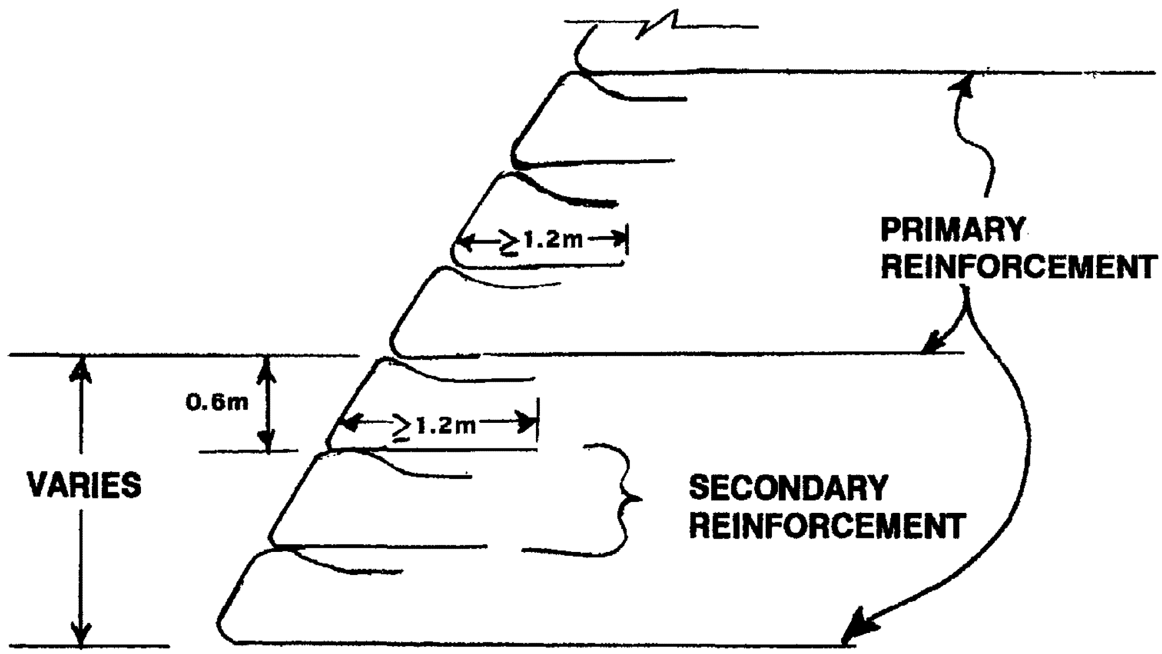
Limiting Assumptions:

- **Extensible reinforcement.**
- **Slopes constructed with uniform, cohesionless soil (c=0).**
- **No pore pressures within slope.**
- **Competent, level foundation soils.**
- **No seismic forces.**
- **Uniform surcharge no greater than $0.2\gamma_r H$.**
- **Relatively high soil/reinforcement interface friction angle $\phi'_{sr} = 0.9\ \phi_r$ (may not be appropriate for some geotextiles).**

Figure 63. Chart solution for determining the reinforcement strength requirements. (after Schmertmann, et al., 1987)



A) SPACING VERSUS REINFORCEMENT STRENGTH



B) PRIMARY AND SECONDARY REINFORCMENT APPROACH

Figure 64. Reinforcement spacing considerations for high slopes.

For 3 zones:

$$T_{Bottom} = 1/2\ T_{S\text{-}MAX} \quad (60)$$

$$T_{Middle} = 1/3\ T_{S\text{-}MAX} \quad (61)$$

$$T_{Top} = 1/6\ T_{S\text{-}MAX} \quad (62)$$

The force is assumed to be uniformly distributed over the entire zone.

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:

$$T_{max} = \frac{T_{zone}\, S_v}{H_{zone}} = \frac{T_{zone}}{N} \leq T_a R_c \quad (63)$$

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.
= $T_{S\text{-}MAX}$ for low slopes (H< 6m).

H_{zone} = height of zone.
= T_{top}, T_{middle}, and T_{Bottom} for high slopes (H > 6m).

N = number of reinforcement layers.

- Use short (1.2 to 2 m) lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 600 mm or less for face stability and compaction quality (see figure 64b).

  - For slopes flatter than 1H:1V, closer spaced reinforcements (i.e., every lift or every other lift, but no greater than 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: [19]

$$F.S. = \frac{c'H + (\gamma_g - \gamma_w)Hz\cos^2\beta\tan\phi' + F_g(\cos\beta\sin\beta + \sin^2\beta\tan\phi')}{\gamma_g Hz\cos\beta\sin\beta} \tag{64}$$

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) and provide localized tensile reinforcement to the surficial soils.

  - If the interface friction angle of the intermediate reinforcement ρ_{sr} 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 57 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_{TOTAL}) must be sufficient to provide adequate pullout resistance based on:

$$L_e = \frac{T_{max}\, FS}{F^* \cdot \alpha \cdot \sigma_v' \cdot 2} \qquad (65)$$

  where F*, α , and σ'_v are defined in chapter 3, section 3.3.

- Minimum value of L_e is 1m. 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 65).

  - 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 65. Longer reinforcements may be required to resolve deep seated failure problems (see step 7).

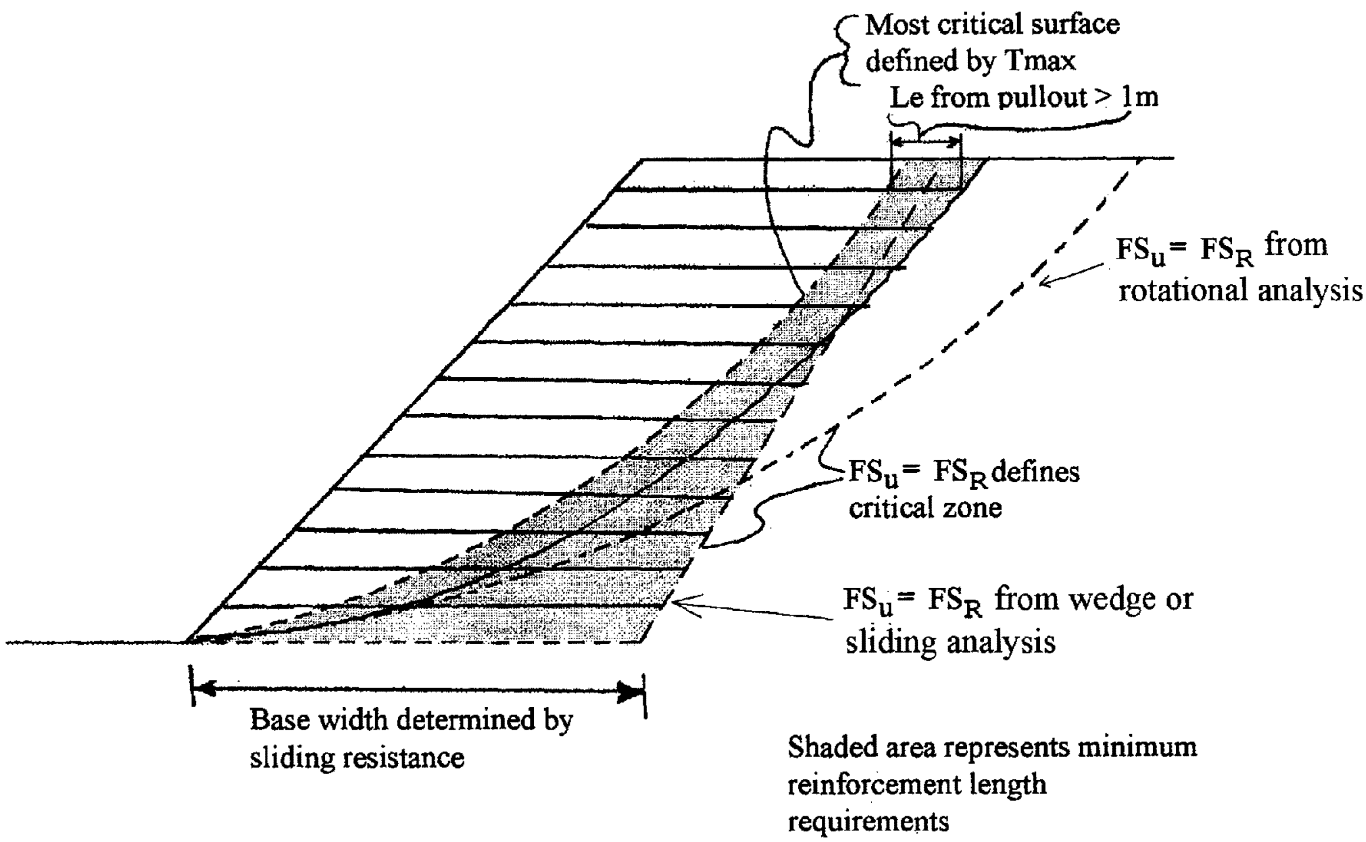


Figure 65. Developing reinforcement lengths.

- 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 65.

- 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 1m 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 63. 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.

Step 7. **Check external stability.** (see discussion in chapter 6.)

- Sliding resistance (figure 66)

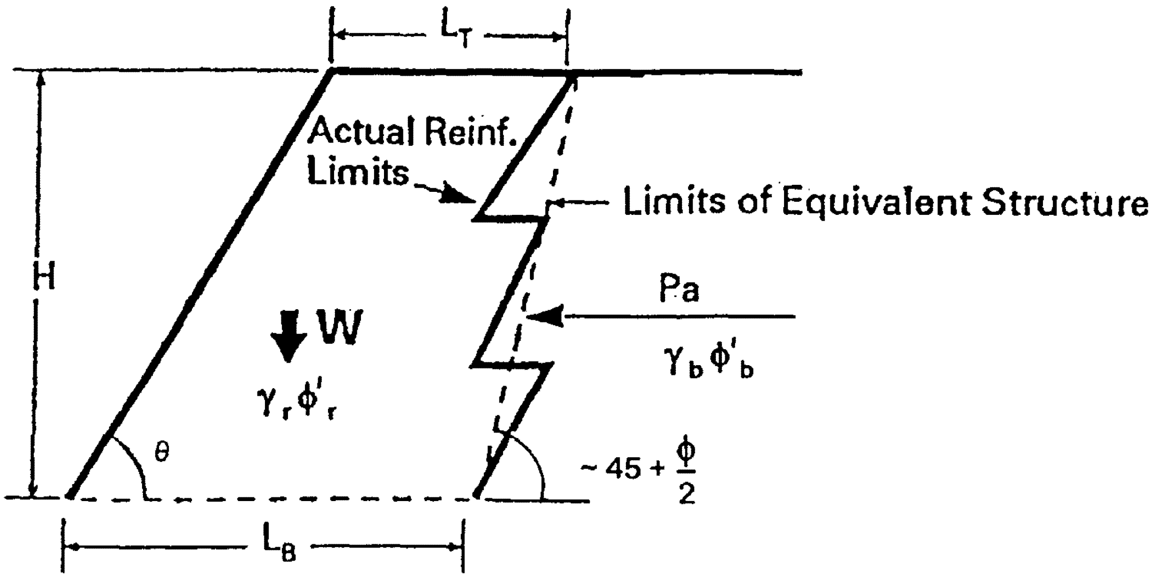


Figure 66. Sliding stability analysis.

- Evaluate the width of the reinforced soil mass **at any level** to resist sliding along the reinforcement. A 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 can be checked to make sure it is sufficient to resist sliding from the following relationships:

Resisting Force = F.S. x Sliding Force

$$(W + P_a \sin \phi_b) \tan \phi_{min.} = FS\ P_a \cos \phi_b \qquad (66)$$

$$with:\ W = 1/2\ L^2\ \gamma_r \tan \theta_r \qquad for\ L < H \qquad (67)$$

$$W = [\ LH - H^2/(2\tan\theta)\ ]\gamma_r \quad for\ L > H \qquad (68)$$

$$P_a = 1/2\ \gamma_b\ H^2\ K_a \qquad (69)$$

where:

L	=	length of bottom reinforcing layer in each level where there is a reinforcement length change.
H	=	height of slope.
FS	=	factor of safety criterion for sliding (>1.3).
P_A	=	active earth pressure.
ϕ_{min}	=	minimum angle of shearing friction either between reinforced soil and reinforcement or the friction angle of the foundation soil.
θ	=	slope angle.
γ_r & γ_b	=	unit weight of the reinforced and retained backfill respectively.
ϕ_b	=	friction angle of retained fill.

- Deep seated global stability (figure 67a).

  - Evaluate potential deep-seated failure surfaces behind the reinforced soil mass to provide:

$$F.S. = M_D/M_R \geq 1.3 \qquad (70)$$

The analysis performed in step 5 should provide this information. However, as a check, classical rotational slope stability methods such as simplified Bishop, Morgenstern and Price, Spencer, or others may be used (see FHWA *Soils and Foundations Workshop Manual*, 1993). Appropriate computer programs also may be used.

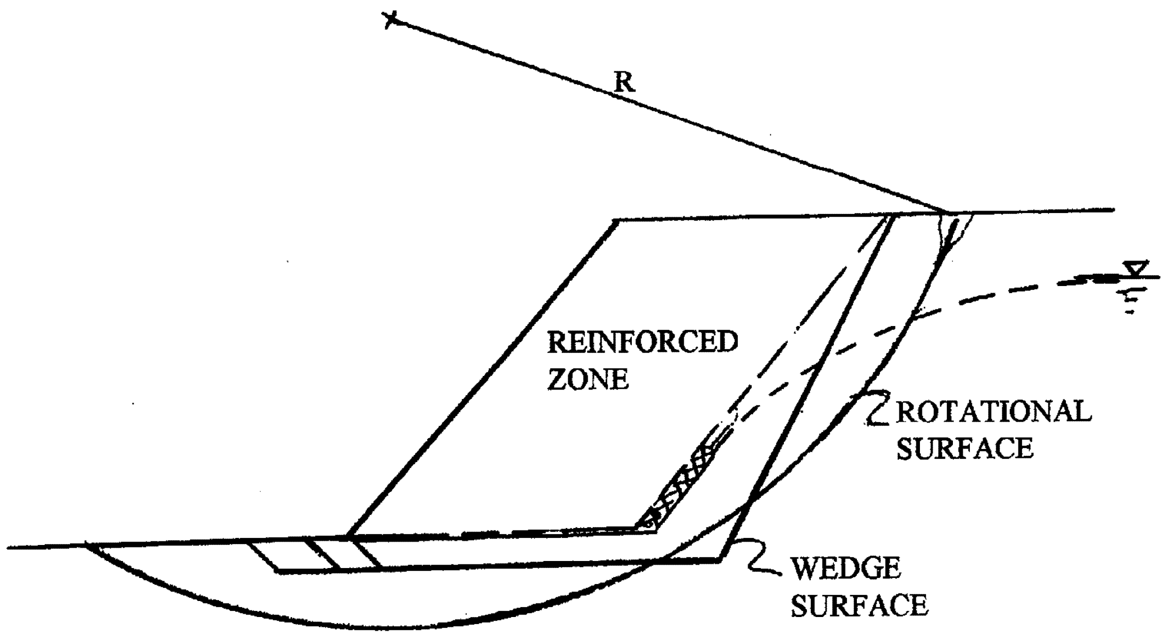
- Local bearing failure at the toe (lateral squeeze) (figure 67b).

  - 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:[16]

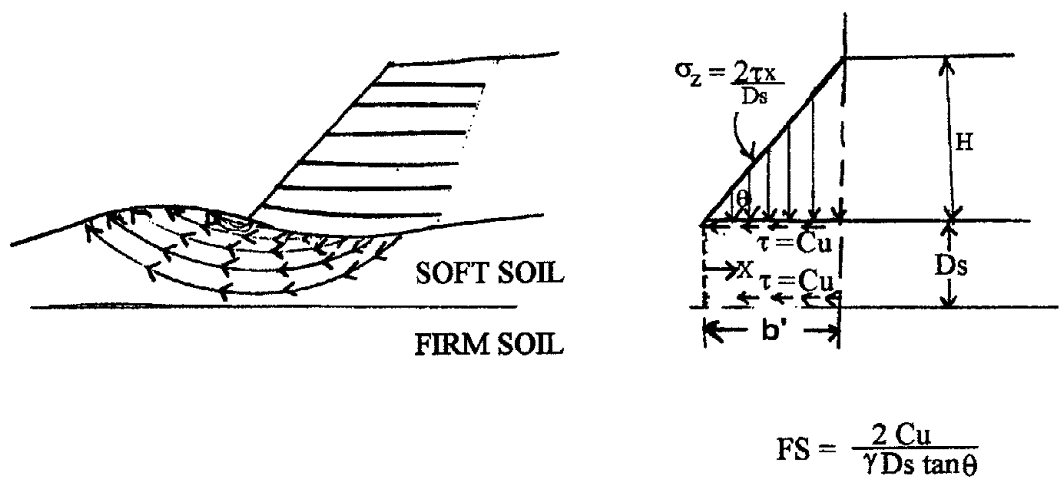
$$FS_{squeezing} = \frac{2\,c_u}{\gamma\, D_s \tan\theta} \geq 1.3 \qquad (71)$$

where:

θ = angle of slope.
γ = unit weight of soil in slope.
D_s = depth of soft soil beneath slope base of the embankment.
c_u = undrained shear strength of soft soil beneath slope.



a) Deep seated (global) stability analysis.



b) Local bearing failure (lateral squeeze)

Figure 67. Failure through the foundation.

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 base width of the slope, b', bearing capacity will govern the 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 Manual*, 1993).

- Dynamic stability (figure 68).

  - Perform a pseudo-static type analysis using a seismic ground coefficient A_o, obtained from local building code and a design seismic acceleration A_m equal to $A_o/2$.

    F.S. dynamic $\leq$ 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 68). 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.

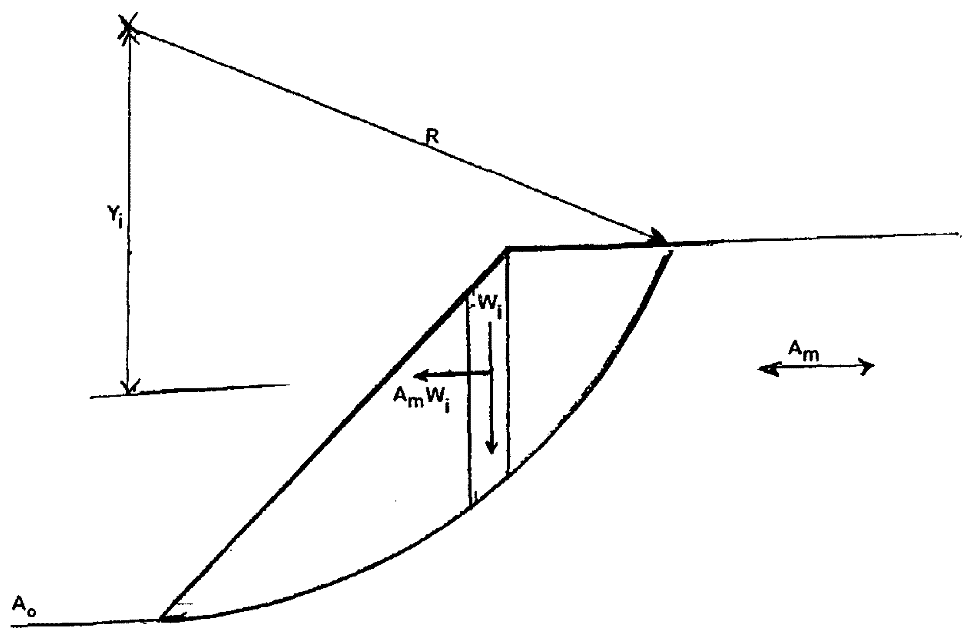


Figure 68. Seismic stability analysis.

Step 8. 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 mass as shown in figure 69. Geocomposite drainage systems or conventional granular blanket and trench drains could be used. Granular drainage systems are not addressed in this document, as it is assumed that criteria for these systems already exists within state agencies.

  - 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:

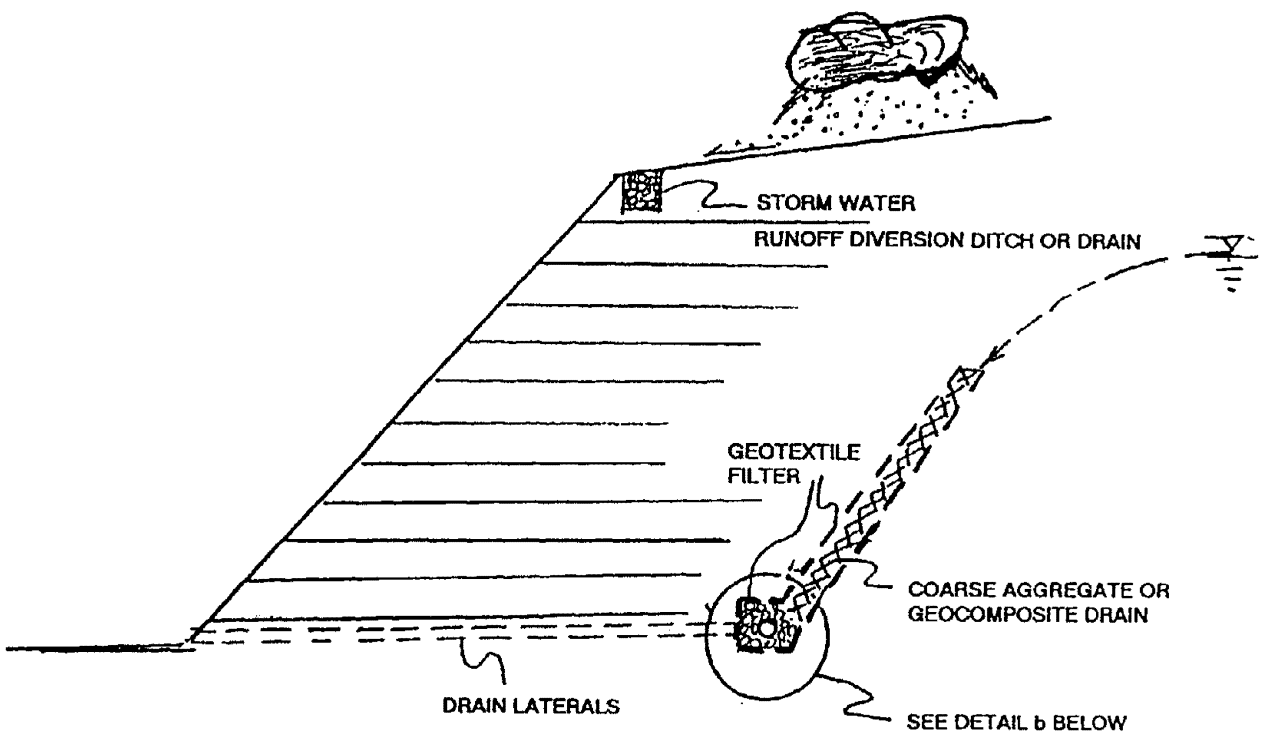
    * Geotextile filtration/clogging.

    * Long-term compressive strength of polymeric core.

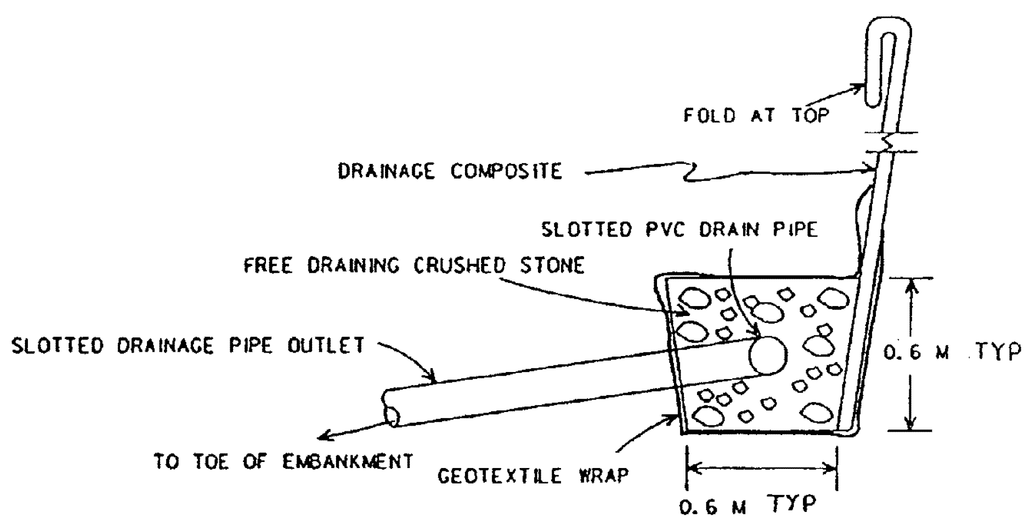
    * Reduction of flow capacity due to intrusion of geotextile into the core.

    * Long-term inflow/outflow capacity.

  Procedures for checking geotextile permeability and filtration/clogging criteria were presented in FHWA *Geosynthetic Design and Construction Guidelines* (1995). 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 by the authors for addressing core compression. The design pressure on a geocomposite core should be limited to either:



a) GROUND WATER AND SURFACE DRAINAGE



b) TYPICAL DRAIN DETAILS

Figure 69. Subsurface drainage considerations.

*	the maximum pressure sustained on the core in a test of 10,000 hour minimum duration.

*	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 D-4716 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. Need depends upon soil type, slope angle, slope height, and primary reinforcement spacing. Guidance is provided in table 11. 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 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.

  - Calculated flow-induced tractive shear stress on the face of the reinforced slope by:

$$\lambda = d \cdot \gamma_W \cdot s \qquad (72)$$

where:

λ = tractive shear stress, kPa.
d = depth of water flow, m.
γ_w = unit weight of water, kN/m^3.
s = the vertical to horizontal angle of slope face, m/m.

For $\lambda < 100$ Pa, consider vegetation with temporary or permanent erosion control mat

For $\lambda > 100$ 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 discretion of the construction contractor.** Guideline material specifications for synthetic permanent erosion control mats are provided in chapter 8.

Table 11. RSS slope facing options.[19]

Slope Face Angle and Soil Type	Type of Facing: When Geosynthetic is not Wrapped at Face		Type of Facing: When Geosynthetic is Wrapped at Face	
	Vegetated Face	Hard Facing	Vegetated Face	Hard Facing
> 50° All Soil Types	Not Recommended	Gabions	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° Clean Sands (SP) Rounded Gravel (GP)	Not Recommended	Gabions Soil-Cement	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° Silts (ML) Sandy Silts (ML)	Bioreinforcement Drainage Composites	Gabions Soil-Cement Stone Veneer	Sod Permanent Erosion Blanket w/ seed	Wire Baskets Stone Shotcrete
35° to 50° 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° 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: Unified Soil Classification
Geosynthetic or natural horizontal drainage layers to intercept and drain the saturated soil at the face of the slope

7.3 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 RSS program. Layout includes number, length, design strength, and vertical distribution of the geosynthetic reinforcement. The charts presented in figure 63 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.

External stability analysis in step 7 will then include an evaluation of local bearing capacity, foundation settlement, and dynamic stability.

7.4 DESIGN EXAMPLES

a. Example 1. Reinforced Slope Design -Road Widening

A 1 km long, 5-m high, 2.5H:1V side slope road embankment in a suburban area is to be widened by one lane. At least a 6-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 7.5-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 7.2 to more clearly follow the meaning of the design sequence.

Step 1. Slope description.

a. Geometric and load requirements

- $H = 5$ m

- $\beta = 45°$

- $q = 10$ 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.

- $\gamma_d = 19$ kN/m^3, $\omega_{opt} = 15\%$, $C = 100$ kPa, $\phi' = 28°$, and $c' = 0$

- At the time of the borings, $d_w = 2$ 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:

Sieve Size	Percent passing
100 mm	100
20 mm	99
4.75 mm	63
0.425 mm	45
0.075 mm	25

  PI (of fines) = 10
  Gravel is durable
  pH = 7.5

- γ_r = 21 kN/m^3, ω_{opt} = 15

- ϕ' = 33°, 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.

- Allowable Strength: $T_a = T_{ult}/RF$

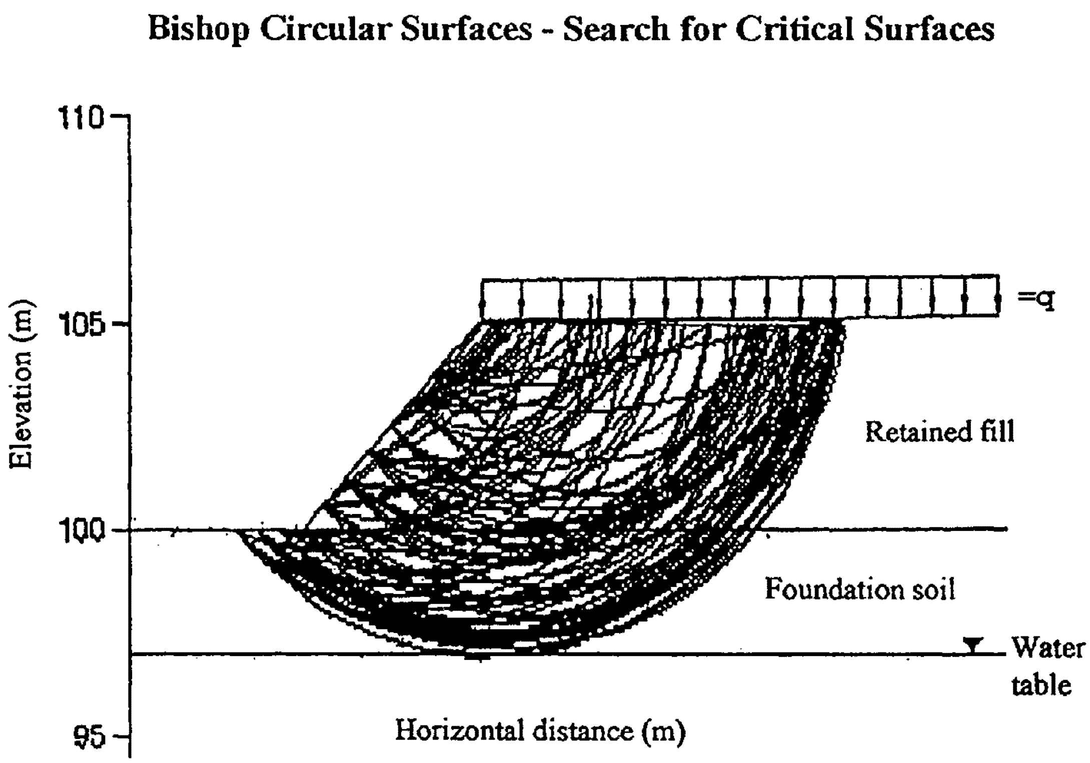
- Pullout Factor of Safety: $FS_{po} = 1.5$

Step 5. Check unreinforced stability

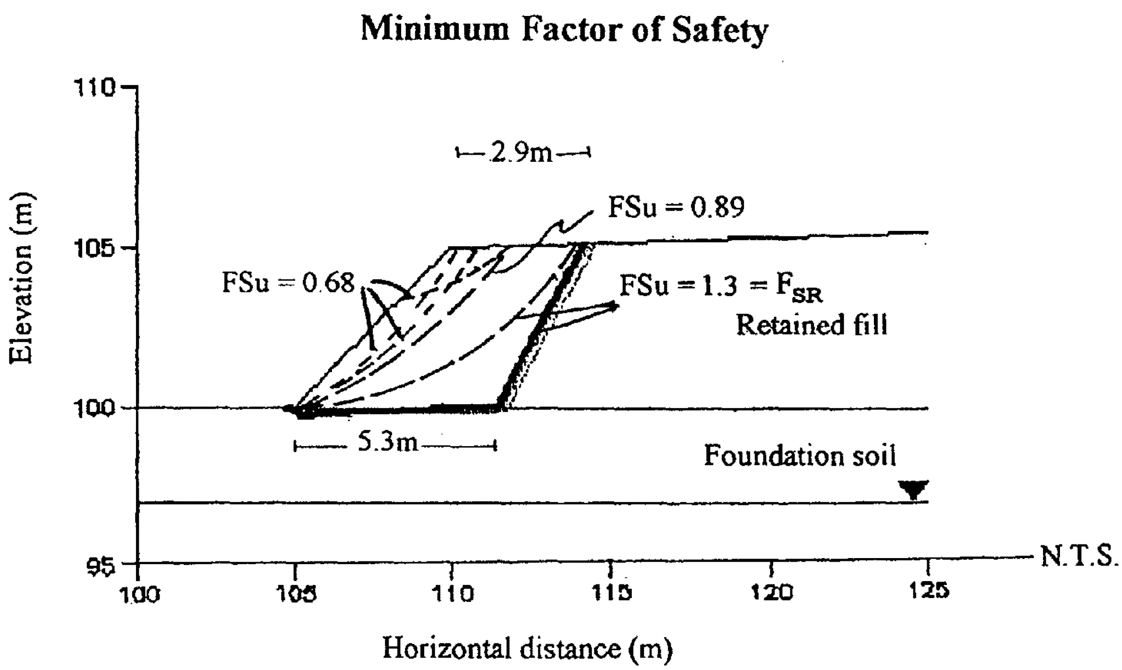
Using STABL4M, a search was made to find the minimum unreinforced safety factor and to define the critical zone. Both rotational and wedge stability evaluations were performed with figure 70a showing the rotational search. The minimum unreinforced safety factor was 0.68 with the critical zone defined by the target factor of safety FS_R as shown in figure 70b. 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 57 as follows. (Note: with minor code modification, this could easily be done as part of the computer analysis.)



A) Unreinforced stability analysis.

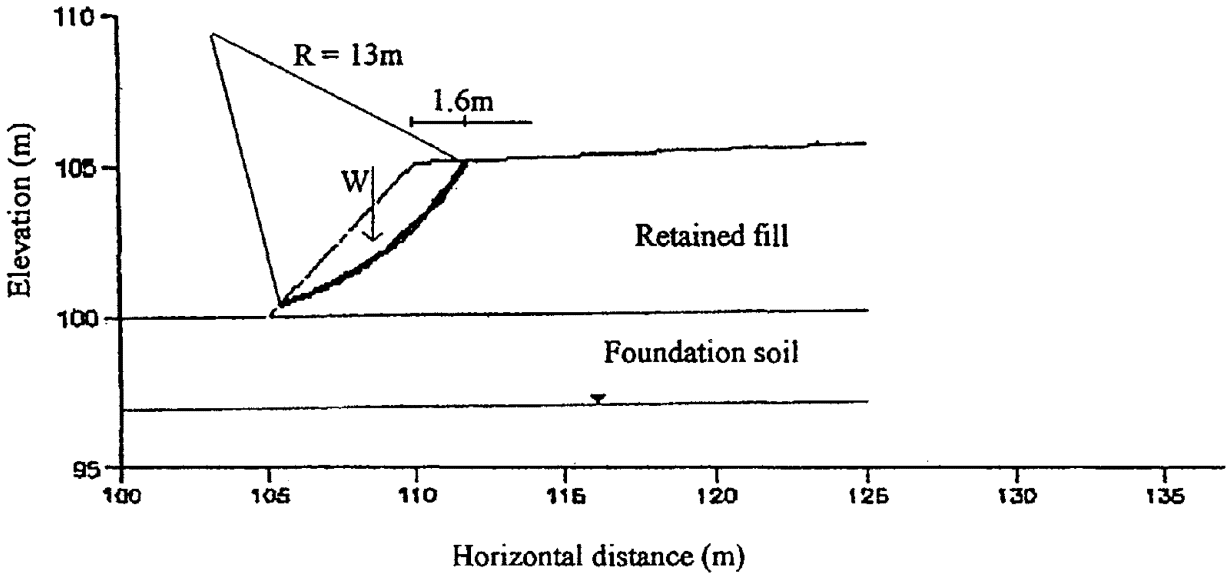


B) Results of unreinforced stability analysis.

Figure 70. Design example 1.

Strenth Design - Reinforcement for Critical Surface

Factor of safety for circle with $T_{s\text{-}max}$ 0.89
Total required reinforcement: 49.7 kN/m



for R= 13m , Md = 1575 kN/m

C) Surface requiring maximum reinforcement (i.e. most critical reinforced surface)

Figure 70. Design example 1 continued.

a. Calculate the total reinforcement tension T_S, required:

$$T_S = (1.3 - FS_U) \frac{M_D}{R} \tag{73}$$

Evaluating all of the surfaces in the critical zone indicates maximum total tension $T_{S\text{-}MAX}$ = 49.7 kN/m for FS_U = 0.89 as shown in figure 70c.

b. Checking $T_{S\text{-}MAX}$ by using the design charts in figure 63:

$$\phi_f = \tan^{-1}\left(\frac{\tan\phi_r}{FS_R}\right) = \tan^{-1}\left(\frac{\tan 33^o}{1.3}\right) = 26.5^o \tag{74}$$

From figure 63a, K ≈ 0.14

and,

$H' = H + q/\gamma_r$ + 0.1 m (for 2% road grade)
= 5 m + (10 kN/m^2 ÷ 21 kN/m^3) + 0.1 m = 5.6 m
then,

$$\begin{aligned} T_{S\text{-}MAX} &= 0.5\, K \gamma_r H'^2 \\ &= 0.5\,(0.14)\,(21\, kN/m^3)\,(5.6\, m)^2 \\ &= 46.1\, kN/m \end{aligned} \tag{75}$$

The evaluation using figure 62 appears to be in reasonably good agreement with the computer analysis for this simple problem.

c. Determine the distribution of reinforcement.

Since H < 6 m, use a uniform spacing. Due to the cohesive nature of the backfill, maximum compaction lifts of 200 mm are recommended.

d. As was discussed in the design section, to avoid wrapping the face use S_v = 400 mm reinforcement spacing; therefore, N = 5 m/0.4 m = 12.5, use 12 layers with the bottom layer placed after the first lift of embankment fill.

$$T_{max} = \frac{T_{S\text{-}MAX}}{N} = \frac{49.7\ kN/m}{12} = 4.14\ kN/m \qquad (76)$$

(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 70a) could be used to define the limits of the reinforcement. This is especially true for this problem since the sliding failure surface with FS ≥ 1.3 encompasses the rotational failure surface with FS ≥ 1.3.

From direct measurement at the bottom and top of the sliding surface in figure 70b, the required lengths of reinforcement are:

L_{bottom} = 5.3 m
L_{top} = 2.9 m

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 = 0.2 m), subtract the distance from the critical surface to the face of the slope in figure 70c from L_{top}. This gives L_e at top = 1.3 m.

Assuming the most conservative assumption for pullout factors F* and α from chapter 3, section 3.3 gives $F^* = 0.67 \tan\phi$ and $\alpha = 0.6$ Therefore,

$$FS = \frac{L_e\ F*\ \alpha\ \sigma_v\ C}{T_{max}} = \frac{1.3\,(0.67\ \tan\ 33^o)\ (0.6)\ (0.2\,m\ x\ 21\,kN/m^3 + 10\,kN/m^2)\,(2)}{4.14\ kN/m} \tag{77}$$

$$FS_{PO} = 2.3 > 1.5\ required$$

Check the length requirement using figure 63b.
For L_B

$$\phi_f' = \tan^{-1}(\frac{\tan 28^o}{1.3}) = 22.2^o \tag{78}$$

From figure 63: $L_B/H' = 0.96$
thus, $L_B = 5.6$ m (0.96) = 5.4 m

For L_T

$$\phi_f' = \tan^{-1}(\frac{\tan 33^o}{1.3}) = 26.5^o \tag{79}$$

From figure 63: $L_T/H' = 0.52$
thus, $L_T = 5.6$ m (0.52) = 2.9 m

The evaluation again, using figure 63, 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 63 or any number of proprietary computer programs could be used for a rapid evaluation of $T_{S\text{-}MAX}$ and T_{max}.

In summary, 12 layers of reinforcement are required with a design strength T_{max} of 4.14 kN/m and an average length of 4 m over the full height of embankment.

b. Example 2. Reinforced Slope Design -New Road Construction

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 19 m and the desired slope of the elevated embankment is 0.8H:1.0V. A geogrid with an ultimate tensile strength of 100 kN/m (ASTM D4595 wide width method) is desired for reinforcing the new slope. A uniform surcharge of 12.5 kN 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 12.5 kPa. The backfill 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, $H = 19$ m
- Reinforced slope angle, $\theta = \tan^{-1}(1.0/0.84) = 50°$
- Existing slope angle, $\beta = \tan^{-1}(0.61/1.0) = 31.4°$
- Surcharge load, $q = 12.5$ kN/m^2

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:

$\phi' = 34°$, $c' = 12.5$ kPa
Depth of water table, $d_w = 1.5$ m below base of embankment

Step 3. Properties of available fill.

The backfill material to be used in the reinforced section was reported to have the following properties:

$\gamma = 18.8$ kN/m^3, $\phi' = 34°$, $c' = 0$

Step 4. Reinforcement performance requirement.

Allowable tensile force per unit width of reinforcement, T_a, with respect to service life and durability requirements:

$$T_a = T_{ult}/RF \text{ and } RF = RF_{CR} \times RF_{ID} \times RF_D \times FS \qquad (80)$$

For the proposed geogrid to be used in the design of the project, the following factors are used:

FS = 1 (note: FS = 1.5 on reinforcement is included in stability equation).
RF_D = durability factor of safety = 1.25.
RF_{ID} = construction damage factor of safety = 1.2.
RF_{CR} = creep reduction factor = 3.0.

Reduction factors were determined by the owner based on evaluation of project conditions and geogrid tests and field performance data submitted by the manufacturer. If this information is not available, a global default value defined in chapter 3, could have been used.

Therefore:

$$T_a = \frac{(100 kN/m)}{(1.25)(1.2)(3)(1)} = 22\ kN/m$$

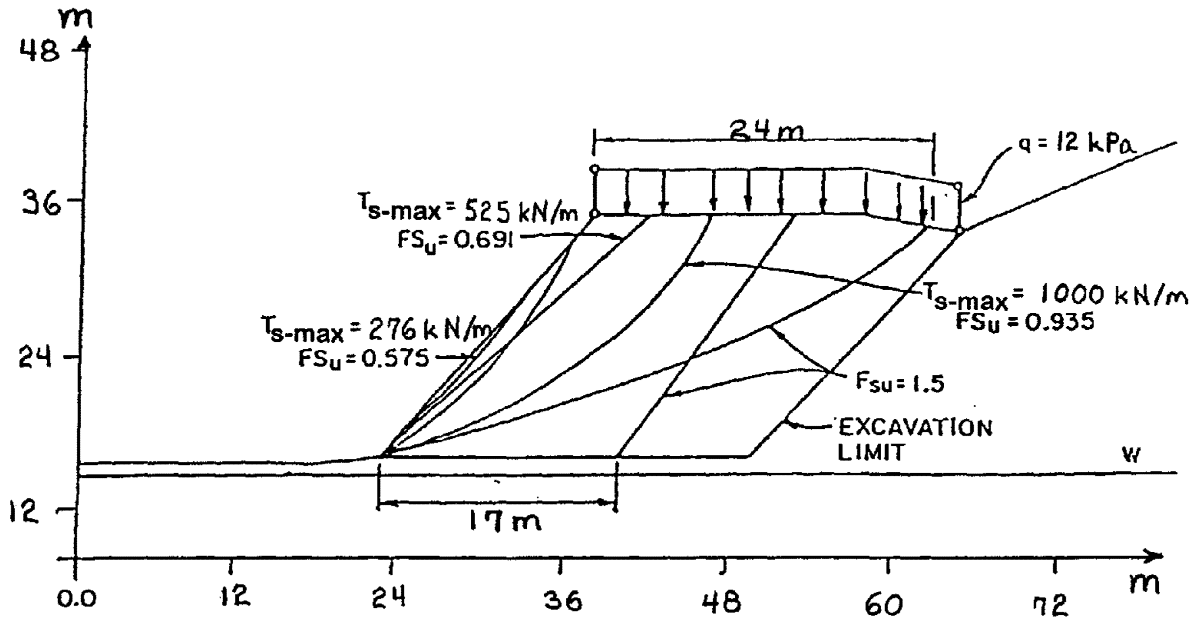
Pullout Resistance: FS = 1.5 for granular soils with a 1 m minimum length in the resisting zone.

Step 5. Check unreinforced stability .

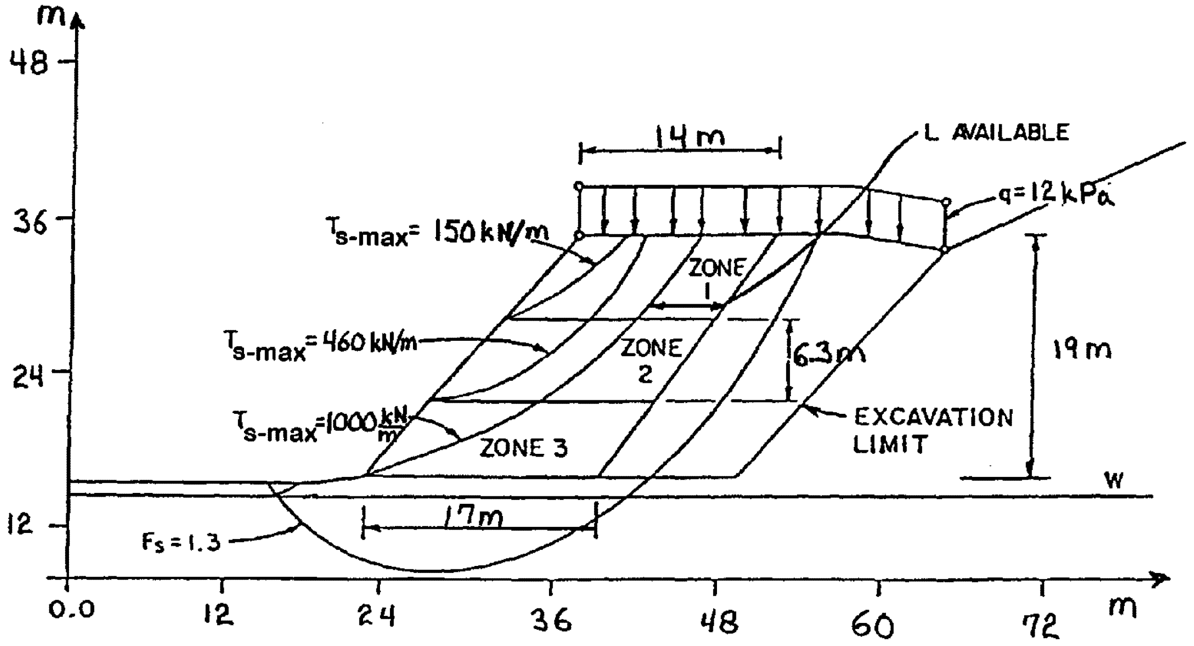
The unreinforced slope stability was checked using the rotational slip surface method, as well as the wedge shaped failure surface method, to determine the limits of the reinforced zone and the required total reinforcement tension to obtain a factor of safety of 1.5.

The proposed new slope was first analyzed without reinforcement using a hand solution (e.g., the FHWA *Soils and Foundations Manual*, 1993) or computer programs such as XSTABL or STABL4M. 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 71a. 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 71a. Based on the wedge shaped failure surface analysis, the limits of the critical zone to be reinforced are reduced to 14 m at the top and 17 m at the bottom for the required factor of safety.



A) Step 5a : Preliminary design length
Step 5b : Determine T_{max}



B) Step 5b : Determine T_{max}
Step 5f : Check reinforcement in upper 2/3 & 1/3 of slope

Figure 71. Design example 2: stability analysis.

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 $T_{S\text{-}MAX}$. An evaluation of all the surfaces in the critical zone indicated $T_{S\text{-}MAX}$ = 1000 kN/m as determined using the equation 73:

$$T_S = (FS_R - FS_U)\,\frac{M_D}{D} = (1.5 - FS_U)\,\frac{M_D}{R}$$

The most critical circle is where the largest $T_S = T_{S\text{-}MAX}$. As shown on the design example figure 71a, $T_{S\text{-}MAX}$ is obtained for $FS_U = 0.935$.

For this surface, M_D = 67,800 kN-m/m (as determined stability analysis).

D = R for geosynthetics = radius of critical circle
R = 38.3 m

$$T_{S\text{-}MAX} = (1.5 - 0.935)\,\frac{67{,}800\ kN\text{-}m/m}{38.3\ m} = 1000 kN/m$$

b. Check using chart design procedure:

For $\theta = 50°$,and
$\phi'_f = \tan^{-1}(\tan\phi_r/FS_R) = \tan^{-1}(\tan 34°/1.5) = 24.2°$

Force coefficient, K = 0.21 (from figure 63a)
and,

$H' = H + q/\gamma_r$ = 19 m + (12.5 kN/m^2)/(18.8 kN/m^3) = 19.7 m
then,

$T_{S\text{-}MAX} = 0.5\,K\gamma_r(H')^2$ = 0.5(0.21)(18.8 kN/m^3)(19.7 m)^2
= 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 $T_{S\text{-}MAX}$ = 1000 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 60 through 62.

$T_{bottom} = 1/2\ T_{S\text{-}MAX} = (1/2)(1000\ kN/m) = 500\ kN/m$

$T_{middle} = 1/3\ T_{S\text{-}MAX} = (1/3)(1000\ kN/m) = 330\ kN/m$

$T_{top} = 1/6\ T_{S\text{-}MAX} = (1/6)(1000\ kN/m) = 170\ kN/m$

d. Determine reinforcement vertical spacing S_v .

Minimum number of layers, $$N = \frac{T_{S\text{-}MAX}}{T_{allowable}} = \frac{1000\ kN/m}{22\ kN/m} = 45.5$$

Distribute at bottom 1/3 of slope:

$$N_B = \frac{500\ kN/m}{22\ kN/m} = 22.7\ use\ 23\ layers$$

At middle 1/3 of slope:

$$N_M = \frac{330\ kN/m}{22\ kN/m} = 15\ layers$$

At upper 1/3 of slope:

$$N_T = \frac{170\ kN/m}{22\ kN/m} = 7.7\ use\ 8\ layers$$

Total number of layers: 46 > 45.5 OK

Vertical spacing:

Total height of slope = 19 m

Height for each zone = 19/3 = 6.3 m

Required spacing:

At bottom 1/3 of slope:

$$S_{required} = \frac{6.3\ m}{23\ layers} = 0.27\ m \ \ use\ 250\ mm\ spacing$$

At middle 1/3 of slope:

$$S_{required} = \frac{6.3\ m}{15\ layers} = 0.42\ m \ \ use\ 400\ mm\ spacing$$

At top 1/3 of slope:

$$S_{required} = \frac{6.3\ m}{8\ layers} = 0.79\ m \ \ use\ 800\ mm\ spacing$$

Provide 2 m length of intermediate reinforcement layers in the upper 1/3 of the slope, between primary layers (based on primary reinforcement spacing at a 400 mm 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 71b.

Top 2/3 of slope: $T_{S\text{-}MAX}$ = 460 kN/m < T_{avail} = 23 layers x 22 kN/m = 506 kN/m

Top 1/3 of slope: $T_{S\text{-}MAX}$ = 150 kN/m < T_{avail} = 8 layers x 22 kN/m = 176 kN/m

f. Determine the reinforcement length required beyond the critical surface for the entire slope from figure 71a, used to determine T_{max} from equation 77,

$$L_e = \frac{T_{max}\ FS}{F^*\ \alpha\ \sigma_v\ C} = \frac{(22 kN/m)\ (1.5)}{(0.8 \tan 34°)(0.66)(18.8\ kN/m^2 \cdot Z)\ (2)} = \frac{2.5\ m}{Z}$$

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 = 0.6 m, L_e = 4.2 m, available length, L_e = 5.2 m OK
Z = 1.2 m, L_e = 2.1 m, available length, L_e = 4.9 m OK
Z = 1.8 m, L_e = 1.4 m, available length, L_e = 4.9 m OK
Z = 2.0 m, L_e = 1.3 m, available length, L_e = 4.9 m OK
Z = 2.8 m, L_e = 0.9 m, available length, L_e = > 5 m OK

Further checks of Z are unnecessary.

Checking the length using figure 63b for $\phi_f = 24°$

$L_T/H' = 0.65 \Rightarrow L_T = 12.8$ m
$L_B/H' = 0.80 \Rightarrow L_B = 15.6$ m

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 = 14 m and bottom length L_B = 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 14 m length at the top and a 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 71b). 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, either 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 72.

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 100 kPa and extended to a depth of 10 m, at which point the granular soils were encountered.

Then, in accordance with equation 71,

$$FS_{squeezing} = \frac{2\,c_u}{\gamma\, D_s \tan\theta} = \frac{2\ (100\ kPa)}{(18.8\ kN/m^3)(10.0\ m)(\tan 50°)} = 0.89$$

Since $FS_{squeezing}$ 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.

7.5 PROJECT COST ESTIMATES

Cost estimates for reinforced slope systems are generally per square meter of vertical face. Table 12 can be used to develop a cost estimate.

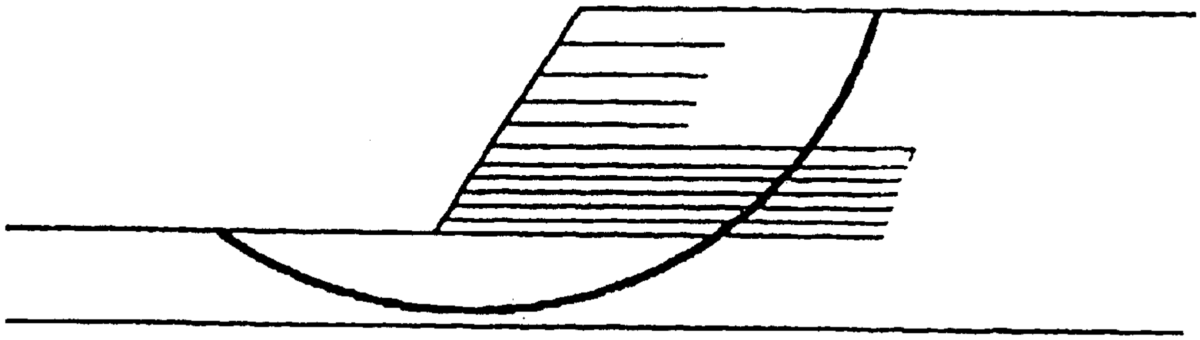


Figure 72. Design example 2: global stability.

As an example, the following provides a cost estimate for design example 1 in chapter 7. Considering the 12 layers of reinforcement at a length of 5 m, the reinforced section would require a total reinforcement of 60 m^2 per meter length of embankment or 12 m^2 per vertical meter of height. Adding 10 percent to 15 percent for overlaps and overages results in an anticipated reinforcement quantity of 13.5 m^2 per meter embankment height. Based on the cost information in Appendix C, reinforcement with an allowable strength $T_a \geq 4.14$ kN/m would cost on the order of \$1.00 to \$1.50/m^2. Assuming \$0.50 m^2 for handling and placement, the in-place cost of reinforcement would be approximately \$25/m^2 of vertical embankment face. Approximately 18.8 m^3 of additional backfill would be required for the reinforced section per meter of embankment length. Using a typical in-place cost for locally available fill with some hauling of \$8/m^3 (about \$4 per 1000 kg), \$30/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 \$2/m^3 for overexcavation and replacement will add approximately \$4/m^2 of vertical face. The erosion protection for the face would also add a cost of \$5/m^2 of vertical face. Thus, the total estimated cost for this option would be on the order of \$64/m^2 of vertical embankment face.

Table 12. Estimated Project Costs.

Item	Total Volume	Unit Cost	Extension	Per Vertical square meter
Backfill (in place)	m^3			
Overexcavation	m^3			
Reinforcement (in place)	m^2			
Facing system Support Vegetation Permanent erosion control mat	 m^2 m^2			
Alternate facing systems	m^2			
Groundwater control system	m^2			
Guardrail	m			
Total	-------	---------		
Unit cost per vertical square meter				

Note Slope Dimensions: Height H =
Length L =
Face Surface Area, A =
Reinforcement Area = $L_{reinforcement}$ * Number of Layers

CHAPTER 8

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 and the frequency of design and construction problems have increased. Some problem areas that have been identified include misapplication of wall technology; poor specifications; lack of specification enforcement; inequitable bidding procedures; and inconsistent selection, review, and acceptance practices on the part of public agencies. Although the actual causes of each particular problem are unique, the lack of formal agency procedures that address the design and construction of earth retaining systems has repeatedly been an indirect cause.

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 explicitedly 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 a normal working drawing submittal.

Some user agencies prefer one approach over 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.

8.1 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 agency uniformity.
- Establish standard policies and procedures for design technical review and acceptance of MSEW and RSS systems or components.
- Establish responsibility for the 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.
- 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 proscriptive or end-result methods.

8.2 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, subsequently offered.

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 contractors 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.

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 an 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 D-5262.

- Laboratory test results and methodology for extrapolation of creep data for 75- and 100-year design life as described in appendix B.

- Laboratory test results documenting ultimate strength in accordance with ASTM D-4595, or GRI-GG1 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 PET, oxidative resistance of PP and 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. Refer to the companion *Corrosion/Degradation* document for recommended methods.

- Field and laboratory test results along with literature review documenting values for partial factor of safety 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 partial factor of safety for biological degradation.

- Laboratory test results documenting joint (seams and connections) strength and values for partial factor of safety for joints and seams (ASTM D-4884 and GRI: GG2).

- Laboratory tests documenting pullout interaction coefficients for various soil types or site-specific soils in accordance with GRI: GG5 and GT7. Appendix A details analysis procedures and methods.

- Laboratory tests documenting direct sliding coefficients for various soil types or project specific soils in accordance with ASTM D-5321.

- 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:

Test	Test Procedure	Minimum Conformance Requirement
Wide Width Tensile (geotextiles)	ASTM D-4595	*To be provided*
Specific Gravity (HDPE only)	ASTM D-1505	*by material*
Melt Flow index (PP & HDPE)	ASTM D-1238	*supplier or*
Intrinsic Viscosity (PET only)	ASTM D-4603	*specialty company*
Carboxyl End Group (PET only)	ASTM D-2455	
Single Rib Tensile (geogrids)	GRI:GG1	

- 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 D-1248 shall be identified. For example type III, class A, grade E5, category 5.

  For PP resins, group, class and grade in accordance with ASTM D-4101 shall be identified. For example group 1, class 1, grade 4.

  For Polyester (PET) resins minimum production intrinsic viscosity (ASTM-4603) and maximum carboxyl end groups (ASTM D-2455) shall be identified.

  For all products the minimum UV resistance as measured by ASTM D-4355 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 deviations to current engineering practice. For reinforced slope systems to current geotechnical practice.

- Past experience in construction and performance of the proposed system.

- The adequacy of the data in support of allowable strength (T_a) for geosynthetic reinforcements.

- The adequacy of the QA/QC plan for the manufacture of geosynthetic reinforcements.

8.3 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 Section 5.8 *Mechanically Stabilized Earth (MSE) 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 Division II of current AASHTO, *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 7. 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.

8.4 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 and potentially less expensive 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:

a. 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.

- Elevation views indicating elevations at top and bottom of walls or slopes. beginning and end stations, horizontal and vertical break points, 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 layout and the designation of the type or module, the elevation of the top of levelling pad and footings, the distance along the face of the wall to all steps in the footings and levelling 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.

b. 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.

c. 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.

d. Design Computations

The plans shall be supported by detailed computations for internal and external stability and life expectancy for the reinforcement.

e. 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 for fills compaction criteria.

- Groundwater or free water conditions and required drainage schemes if required.

f. 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.

8.5 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 preapproved 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. Design of the MSE structure is performed by trained and experienced staff. 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 and potentially less expensive 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:

a. **Geometric Requirements**

- Plan and elevation of the areas to be retained, including beginning and end stations.

- 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.

b. **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 stability, allowable bearing and settlement analyses, as they would be the same regardless of the system used. The contractor/supplier would assume responsibility for both internal and external stability for the designed structures.

c. **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.

d. Performance Requirements

- Tolerable movement of the structure both horizontal and vertical.

- Tolerable face panel movement.

- Monitoring and measurement requirements.

8.6 REVIEW AND APPROVALS

Where agency design is based on suppliers 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.

8.7 CONSTRUCTION SPECIFICATIONS AND SPECIAL PROVISIONS FOR MSEW 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 8.8 - Guide specifications for MSE walls with segmental precast concrete facings and steel reinforcements (grid or strip).
- Section 8.9 - Guide specifications for concrete modular block (MBW) facing.
- Section 8.10 - Guide specifications for geosynthetic reinforcement materials.
- Section 8.11 - Construction specifications and special provisions 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.

8.8 GUIDE SPECIFICATIONS FOR MSE WALLS WITH SEGMENTAL PRECAST CONCRETE FACINGS

Description

This work shall consist of mechanically stabilized walls and abutments constructed in accordance with these specifications in reasonably close conformity with the lines, grades, and dimensions shown on the plans or established by the engineer. Design details for these structures such as specified strip or mesh length, concrete panel thickness, and loading appurtenances shall be as shown on the plans. This specification is intended to cover all steel strip or mesh stabilized earth wall systems utilizing discrete concrete face panels, some of which may be proprietary.

Working Drawings

Working drawings and design calculations shall be submitted to the engineer for review and approval at least 4 weeks before work is to begin. Such submittals shall be required (1) for each alternative proprietary or nonproprietary earth retaining system proposed as permitted or specified in the contract, (2) when complete details for the system to be constructed are not included in the plans, and (3) when otherwise required by the special provisions of these specifications. Working drawings and design calculations shall include the following:

(1) Existing ground elevations that have been verified by the Contractor for each location involving construction wholly or partially in original ground.

(2) Layout of wall that will effectively retain the earth but not less in height or length than that shown for the wall system in the plans.

(3) Complete design calculations substantiating that the proposed design satisfies the design parameters in the plans and in the special provisions.

(4) Complete details of all elements required for the proper construction of the system, including complete material specifications.

(5) Earthwork requirements including specifications for material and compaction of backfill.

(6) Details of revisions or additions to drainage systems or other facilities required to accommodate the system.

(7) Other information required in the plans or special provisions or requested by the Engineer.

The contractor shall not start work on any earth retaining system for which working drawings are required until such drawings have been approved by the engineer. Approval of the contractor's working drawings shall not relieve the contractor of any of his responsibility under the contract for the successful completion of the work.

Materials

General. The contractor shall make arrangements to purchase or manufacture the facing elements, reinforcing mesh or strips, attachment devices, joint filler, and all other necessary components. Materials not conforming to this section of the specifications or from sources not listed in the contract document shall not be used without written consent from the engineer.

Reinforced Concrete Facing Panels. The panels shall be fabricated in accordance with Section 8.13 of AASHTO, Division II, with the following exceptions and additions.

(1) The Portland cement concrete shall conform to Class A, (AE) with a minimum 27.6 MPa compressive strength at 28 days. All concrete shall have air entrainment of 6 percent $\pm$ 1.5 percent with no other additives.

(2) The units shall be fully supported until the concrete reaches a minimum compressive strength of 6.9 MPa. The units may be shipped after reaching a minimum compressive strength of 23.4 MPa. At the option of the contractor, the units may be installed after the concrete reaches a minimum compressive strength of 23.4 MPa.

(3) Unless otherwise indicated on the plans or elsewhere in the specification, the concrete surface for the front face shall have a Class 1 finish as defined by section 8.12 and for the rear face a uniform surface finish. The rear face of the panel shall be screened to eliminate open pockets of aggregate and surface distortions in excess of 6 mm. The panels shall be cast on a flat area. the strips or other galvanized attachment devices shall not contact or be attached to the face panel reinforcement steel.

(4) Marking - The date of manufacture, the production lot number, and the piece mark shall be clearly scribed on an unexposed face of each panel.

(5) Handling, Storage, and Shipping - All units shall be handled, stored, and shipped in such a manner as to eliminate the dangers of chipping, discoloration, cracks, fractures, and excessive bending stresses. Panels in storage shall be supported in firm blocking to protect the panel connection devices and the exposed exterior finish.

(6) Tolerances - All units shall be manufactured within the following tolerances:

- Panel Dimensions - Position panel connection devices within 25 mm, except for all other dimensions within 5 mm.

- Panel Squareness - Squareness as determined by the difference between the two diagonals shall not exceed 13 mm.

- Panel Surface Finish - Surface defects on smooth formed surfaces measured over a length of 1.5 m shall not exceed 3 mm. Surface defects on the textured-finish surfaces measured over a length of 1.5 m shall not exceed 8 mm.

(7) Steel - In accordance with section 9.

(8) Compressive Strength - Acceptance of concrete panels with respect to compressive strength will be determined on the basis of production lots. A production lot is defined as a group of panels that will be represented by a single compressive strength sample and will consist of either 40 panels or a single day's production, whichever is less.

During the production of the concrete panels, the manufacturer will randomly sample the concrete in accordance with AASHTO T-141. A single compressive strength sample, consisting of a minimum of four cylinders, will be randomly selected for every production lot.

Compression tests shall be made on a standard 152 mm by 305 mm test specimen prepared in accordance with AASHTO T-23. Compressive strength testing shall be conducted in accordance with AASHTO T-22.

Air content testing will be performed in accordance with AASHTO T-152 or AASHTO T-196. Air content samples will be taken at the beginning of each day's production and at the same time as compressive samples are taken to ensure compliance.

The slump test will be performed in accordance with AASHTO T-119. The slump will be determined at the beginning of each day's production and at the same time as the compressive strength samples are taken.

For every compressive strength sample, a minimum of two cylinders shall be cured in accordance with AASHTO T-23 and tested at 28 days. The average compressive strength of these cylinders, when tested in accordance with AASHTO T-22, will provide a compressive strength test result that will determine the compressive strength of the production lot.

If the contractor wishes to remove forms or ship the panels prior to 28 days, a minimum of two additional cylinders will be cured in the same manner as the panels. The average compressive strength of these cylinders when tested in accordance with AASHTO T-22 will determine whether the forms can be removed or the panels shipped.

Acceptance of a production lot will be made if the compressive strength test result is greater than or equal to 27.6 MPa. If the compressive strength test result is less than 27.6 MPa, then acceptance of the production lot will be based on its meeting the following acceptance criteria in their entirety:

- Ninety percent of the compressive strength test results for the overall production shall exceed 28.6 MPa.

- The average of any six consecutive compressive strength test results shall exceed 29.3 MPa.

- No individual compressive strength test result shall fall below 24.8 MPa.

Rejection. Units shall be rejected because of failure to meet any of the requirements specified above. In addition, any or all of the following defects shall be sufficient cause for rejection:

- Defects that indicate imperfect molding.

- Defects indicating honeycombing or open texture concrete.

- Cracked or severely chipped panels.

- Color variation on front face of panel due to excess form oil or other reasons.

Soil Reinforcing and Attachment Devices. All reinforcing and attachment devices shall be carefully inspected to ensure they are true to size and free from defects that may impair their strength and durability.

(1) Reinforcing Strips - Reinforcing strips shall be hot rolled from bars to the required shape and dimensions. Their physical and mechanical properties shall conform to either ASTM A-36 or ASTM A-572 grade 65 (AASHTO M-223) or equal. Galvanization shall conform to the minimum requirements or ASTM A-123 (AASHTO M-111).

(2) Reinforcing Mesh - Reinforcing mesh shall be shop-fabricated of cold drawn steel wire conforming to the minimum requirements of ASTM A-82 (AASHTO M-32)and shall be welded into the finished mesh fabric in accordance with ASTM A-185 (AASHTO M-55). Galvanization shall be applied after the mesh is fabricated and conform to the minimum requirements of ASTM A-123 (AASHTO M-111).

(3) Tie Strips - The tie strips shall be shop-fabricated of a hot rolled steel conforming to the minimum requirements of ASTM 570, Grade 50 or equivalent. Galvanization shall conform to ASTM A-123 (AASHTO M-111).

(4) Fasteners - Fasteners shall consist of hexagonal cap screw bolts and nuts, which are galvanized and conform to the requirements of ASTM A-325 (AASHTO M-164) or equivalent.

(5) Connector Pins - Connector pins and mat bars shall be fabricated from A-36 steel and welded to the soil reinforcement mats as shown on the plans. Galvanization shall conform to ASTM A-123 (AASHTO M-111). Connector bars shall be fabricated of cold drawn steel wire conforming to the requirements of ASTM A-82 (AASHTO M-32) and galvanized in accordance with ASTM A-123 (AASHTO M-111).

Joint Materials. Installed to the dimensions and thicknesses in accordance with the plans or approved shop drawings.

(1) If required, provide flexible foam strips for filler for vertical joints between panels, and in horizontal joints where pads are used, where indicated on the plans.

(2) Provide in horizontal joints between panels preformed EPDM rubber pads conforming to ASTM D-2000 for 4AA, 812 rubbers, neoprene elastomeric pads having a Durometer Hardness of 55 ± 5, or high density polyethylene pads with a minimum density of 0.946 g/cm^3 in accordance with ASTM 1505.

(3) Cover all joints between panels on the back side of the wall with a geotextile meeting the minimum requirements for filtration applications as specified by AASHTO M-288. The minimum width and lap shall be 300 mm.

Select Granular Backfill Material. All backfill material used in the structure volume shall be reasonably free from organic or otherwise deleterious materials and shall conform to the following gradation limits as determined by AASHTO T-27.

U.S. Sieve Size	Percent Passing
102 mm	100
No. 40 mesh sieve	0 - 60
No. 200 mesh sieve	0 - 15

The backfill shall conform to the following additional requirements:

(1) The plasticity index (P.I.) as determined by AASHTO T-90 shall not exceed 6.

(2) The material shall exhibit an angle of internal friction of not less than 34°, as determined by the standard direct shear test AASHTO T-236 on the portion finer than the No. 10 sieve, using a sample of the material compacted to 95 percent of AASHTO T-99, Methods C or D (with oversized correction as outlined in Note 7 at optimum moisture content). No testing is required for backfills where 80 percent of sizes are greater than 19 mm.

(3) Soundness - 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, measured in accordance with AASHTO T-104, or a sodium sulfate loss of less than 15 percent after five cycles determined in accordance with AASHTO T-104.

(4) Electrochemical Requirements - The backfill materials shall meet the following criteria:

Requirements	Test Methods
Resistivity >3,000 ohm-cm	AASHTO T-288-91
pH 5-10	AASHTO T-289-91
Chlorides <100 parts per million	AASHTO T-291-91
Sulfates <200 parts per million	AASHTO T-290-91
Organic Content <1%	AASHTO T-267-86

If the resistivity is greater or equal to 5000 ohm-cm, the chloride and sulfates requirements may be waived.

Concrete Leveling Pad. The concrete footing shall conform to AASHTO Division II, section 8.2 for Class B concrete.

Acceptance of Material. The contractor shall furnish the engineer a Certificate of Compliance certifying the above materials, comply with the applicable contract specifications. A copy of all test results performed by the contractor necessary to assure contract compliance shall be furnished to the engineer.

Acceptance will be based on the Certificate of Compliance, accompanying test reports, and visual inspection by the engineer, or tests performed independently by the engineer.

Construction

Wall Excavation. Unclassified excavation shall be in accordance with the requirements of AASHTO Division II, Section 1 and in reasonably close conformity to the limits and construction stages shown on the plans. Temporary excavation support as required shall be the responsibility of the contractor.

Foundation Preparation. The foundation for the structure shall be graded level for a width equal to the length of reinforcement elements plus 300 mm or as shown on the plans. Prior to wall construction, except where constructed on rock, the foundation shall be compacted with a smooth wheel vibratory roller. Any foundation soils found to be unsuitable shall be removed and replaced with select granular backfill as per Materials of these specifications.

At each panel foundation level, a precast reinforced or a cast-in-place unreinforced concrete leveling pad of the type shown on the plans shall be provided. The leveling pad shall be cured a minimum of 12 hours before placement of wall panels.

Wall Erection. Where a proprietary wall system is used, a field representative shall be available during the erection of the wall to assist the fabricator, contractor, and engineer.

Precast concrete panels shall be placed so that their final position is vertical or battered as shown on the plans. For erection, panels are handled by means of lifting devices connected to the upper edge of the panel. Panels should be placed in successive horizontal lifts in the sequence shown on the plans as backfill placement proceeds. As backfill material is placed behind the panels, the panels shall be maintained in position by means of temporary wedges or bracing according to the wall supplier's recommendations. Concrete facing vertical tolerances and horizontal alignment tolerances shall not exceed 20 mm when measured with a 3 m straight edge. During construction, the maximum allowable offset in any panel joint shall be 20 mm. The completed wall shall have overall vertical tolerance of the wall (top to bottom) shall not exceed 13 mm per 3 m of wall height. Reinforcement elements shall be placed normal to the face of the wall, unless otherwise shown on the plans. Prior to placement of the reinforcing elements, backfill shall be compacted in accordance with these specifications.

Backfill Placement. Backfill placement shall closely follow erection of each course of panels. Backfill shall be placed in such a manner as to avoid any damage or disturbance of the wall materials or misalignment of the facing panels or reinforcing element. Any wall materials that become damaged during backfill placement shall be removed and replaced at the contractor's expense. Any misalignment or distortion of the wall facing panels due to placement of backfill outside the limits of this specification shall be corrected at the contractor's expense. At each reinforcement level, the backfill shall be placed to the level of the connection. Backfill placement methods near the facing shall assure that no voids exist directly beneath the reinforcing elements.

Backfill shall be compacted to 95 percent of the maximum density as determined by AASHTO T-99, Method C or D (with oversize corrections as outlined in Note 7 of that test). For backfills containing more than 30 percent retained on the 19 mm sieve, a method compaction consisting of at least four passes by a heavy roller shall be used. For applications where spread footings are used to support bridge or other structural loads, the top 1.5 m below the footing elevation should be compacted to 100 percent AASHTO T-99.

The moisture content of the backfill material prior to and during compaction shall be uniformly distributed throughout each layer. Backfill materials shall be placed at a moisture content not more than 2 percentage points less than or equal to the optimum moisture content. Backfill material with a placement moisture content in excess of the optimum moisture content shall be removed and reworked until the moisture content is uniformly acceptable throughout the entire lift.

The maximum lift thickness before compaction shall not exceed 300 mm. The contractor shall decrease this lift thickness, if necessary, to obtain the specified density. Compaction within 1 m of the back face of the wall shall be achieved by at least three passes of a lightweight mechanical tamper, roller, or vibratory system.

At the end of each day's operation, the contractor shall slope the level of the backfill away from the wall facing to rapidly direct runoff away from the face. The contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

Measurement

Wall Materials. The unit of measurement for furnishing and fabricating all materials for the walls, including facing materials, reinforcement elements, attachment devices, joint materials, and incidentals will be the square meter of wall face constructed.

Wall Erection. The unit of measurement for wall erection will be per square meter of wall face. The quantity to be paid for will be the actual quantity erected in place at the site. Payment shall include compensation for foundation preparation, technical representatives, reinforcement elements, and erection of the panel elements to the lines and grade shown on the plans.

Concrete Leveling Pad. The unit of measurement for the concrete leveling pad will be the number of linear meters, complete in place and accepted, measured along the lines and grade of the footing.

Select Granular Backfill. The unit of measurement for select granular backfill will be the embankment plan quantity in cubic meters.

Payment

The quantities, determined as described above, will be paid for at the contract price per unit of measurement, respectively, for each pay item listed below and shown in the bid schedule, which prices and payment will be full compensation for the work prescribed in this section, except as provided below:

Excavation of unsuitable foundation materials will be measured and paid for as provided in AASHTO Division II, Section 1. Select backfill for replacement of unsuitable foundation materials will be paid for under item (4).

Payment will be made under:

	Pay Item	Pay Unit
1.	Wall materials	Square meter
2.	Wall erection	Square meter
3.	Concrete leveling pad	Linear meter
4.	Select granular backfill	Cubic meter
5.	Coping barrier	Linear meter
6.	Traffic barriers	Linear meter

MSE walls have been contracted on a lump sum or per wall basis to include compensation for all excavation, temporary support as required, materials, labor and incidental construction. For equitable bidding this method requires accurate quantity determinations and a method of compensation for changed conditions and or overruns/underruns of quantities.

8.9 GUIDE SPECIFICATIONS FOR CONCRETE MODULAR BLOCK (MBW) FACING AND UNIT FILL

Where MBW units are specified for a project, the primary specification detailed in Section 8.8, requires a deletion of *Reinforced Concrete Facing Panels* and the insertion of a new section detailed below. *Wall erection* requires the deletion of the first two sentences from the second paragraph. A specification for unit fill placed within the MBW units must be added.

It is presently recommended that the format of National Concrete Masonry Association (NCMA) TEK 2-4 (1994) specifications be used, except that the compressive strength for units should be increased to 28 MPa to increase durability, maximum water absorption be limited to 5 percent, requirements added for freeze-thaw testing, and tolerance limits expanded.

The full amended specification is included as follows:

Scope

This specification covers hollow and solid concrete structural retaining wall units, machine made from portland cement, water, and mineral aggregates with or without the inclusion of other materials. The units are intended for use in the construction of mortarless, modular block (MBW) retaining walls.

Referenced ASTM Documents

C-33	Specifications for Concrete Aggregates
C-140	Methods of Sampling and Testing Concrete Masonry Units
C-150	Specification for Portland Cement
C-331	Specification for Lightweight Aggregates for Concrete Masonry Units
C-595	Specification for Blended Hydraulic Cements
C-618	Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete.
C-989	Specification for Ground Granulated Blast Furnace Slag Cement
C-666	Standard Method of Evaluating the Freeze-Thaw Durability Resistance of Concrete to Rapid Freezing and Thawing.

Materials

1.0 Cementious Materials - Materials shall conform to the following applicable specifications.

1.1 Portland Cement - Specification C-150.

1.2 Modified Portland Cement - Portland Cement conforming to Specification C-150, modified as follows:

1.2.1 Limestone - Calcium carbonate, with a minimum 85% ($CaCO_3$) content, may be added to the cement, provided the requirements of Specification C 150 as modified are met:

1) Limitation on Insoluble Residue - 1.5%
2) Limitation on Air Content of Mortar - Volume percent, 22% max.
3) Limitation on Loss of Ignition - 7%

1.3 Blended Cements - Specification C-595.

1.4 Pozzolans - Specification C-618.

1.5 Blast Furnace Slag Cement - Specification C-989.

NOTE: *Sulphate resistant cement should be used in the manufacture of units to be used in areas where the soil has high sulphate content such as arid regions of the western United States.*

2.0 Aggregates - Aggregates shall conform to the following specifications, except that grading requirements shall not necessarily apply:

2.1 Normal Weight Aggregates - Specification C-33.

2.2 Lightweight Aggregates - Specification C-331.

3.0 Other Constituents - Air-entraining agents, coloring pigments, integral water repellents, finely ground silica, and other constituents shall be previously established as suitable for use in concrete segmental retaining wall units and shall conform to applicable ASTM Standards or, shall be shown by test or experience to be not detrimental to the durability of the concrete segmental retaining wall units or any material customarily used in masonry construction.

Physical Requirements

1.0 At the time of delivery to the work site, the units shall conform to the following physical requirements:

Table 1. Physical Requirements

Minimum required compressive strength (Average 3 coupons) MPa	=	28 MPa
Minimum required compressive strength (Individual coupon) MPa	=	24.5 MPa
Maximum water absorption	=	5%
Maximum number of blocks per lot	=	2,000

Note: *Freeze thaw requirements may be omitted in areas of insignificant freeze thaw. Blocks may be sealed with a water resistant coating in lieu of meeting the freeze thaw requirements of C-666.*

2.0 Tolerances. Blocks shall be manufactured within the following tolerances:

2.1 The length and width of each individual block shall be within ± 3.2 mm of the specified dimension. Interior dimensions shall be within ± 25 mm.

2.2 The height of each individual block shall be within ± 1.6 mm of the specified dimension.

2.3 When a broken face finish is required, the dimension of the front face shall be within ± 25 mm of the theoretical dimension of the unit.

2.4 Finish and Appearance. 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 incidental to the usual method of manufacture or minor chipping resulting from shipment and delivery, are not grounds for rejection.

The face or faces of units that are to be exposed shall be free of chips, cracks or other imperfections when viewed from a distance of 10 m under diffused lighting. Up to five percent of a shipment may contain slight cracks or small chips not larger than 25 mm.

3.0 Sampling and Testing. Acceptance of the concrete block with respect to compressive strength, will be determined on a lot basis. The lot will be randomly sampled in accordance with ASTM C-140. Compressive strength tests shall be performed by the manufacturer and submitted to the Owner. Compressive strength test specimens shall be cored or shall conform to the saw-cut coupon provisions of section 5.2.4 of ASTM C-140. Blocks represented by test coupons that do not reach an average compressive strength of 28 MPa will be rejected.

3.1 Rejection. Blocks shall be rejected because of failure to meet any of the requirements specified above. In addition, any or all of the following defects shall be sufficient cause for rejection.

- Defects that indicate imperfect molding.
- Defects indicating honeycomb or open texture concrete.
- Cracked or severely chipped blocks.
- Color variation on front face of block due to excess form oil or other reasons.

Unit Fill

The unit fill and drainage agggregate shall be a well graded crushed stone or granular fill meeting the following gradation:

U.S. Sieve Size	Percent Passing
25 mm	100-75
19 mm	50-75
No. 4	0-60
No. 40	0-50
No.200	0-5

8.10 GUIDE SPECIFICATIONS FOR GEOSYNTHETIC REINFORCEMENT MATERIALS

Where geosynthetic reinforcements are used for the construction of MSE walls with either modular block facings (MBW) or segmental precast concrete units, the primary specification under *Materials, Soil Reinforcement and Attachment Devices* should be replaced as follows:

Materials

1.0 *Geotextiles and Thread for Sewing*

Woven or nonwoven geotextiles shall consist only of long chain polymeric filaments or yarns formed into a stable network such that the filaments or yarns retain their position relative to each other during handling, placement, and design service life. At least 95 percent by weight of the long chain polymer shall be a polyolefin or polyester. The material shall be free of defects and tears. The geotextile shall conform as a minimum to the properties indicated for Separation, Medium Survivability indicated under AASHTO T-288. The geotextile shall be free from any treatment or coating that might adversely alter its physical properties after installation.

2.0 *Geogrids*

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 or rock. The geogrid structure shall be dimensionally stable and able to retain its geometry under manufacture, transport and installation.

3.0 *Required Properties*

The specific geosynthetic material(s) shall be preapproved by the agency and shall have certified long-term strength (T_{al}) as shown on table 1 for each geosynthetic specified and for the fill type shown.

4.0 Certification: 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 and as set forth in these specifications.

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. The certificates shall be attested to by a person having legal authority to bond the manufacturer. 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.

Table 1. Required Geosynthetic Properties.

Geosynthetic	Ultimate Strength ASTM 4595[1] (T_{ULT}) GRI:GG1	Long-Term Strength[2] (T_{al})	Pullout Resistance Factor [3] F*	For use with these Fills[4]
A				GW-GM
A				SW-SM-SC
B				GW-GM
B				SW-SM-SC

(1) Based on minimum average roll values (MARV) (kN/m)

(2) Long-Term strength (T_{al}) based on (kN/m)

$$T_{al} = \frac{T_{ULT}}{RF_D \cdot RF_{ID} \cdot RF_{CR}}$$

where RF_{CR} is developed from creep tests performed in accordance with ASTM D-5262, 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.

(3) Pullout Resistance Factor developed in accordance with chapter 3 of this manual.

(4) Unified Soil Classification.

5.0 Manufacturing Quality Control: The geosynthetic reinforcement shall be manufactured with a high degree of quality control. The Manufacturer is responsible for establishing and maintaining a quality control program to ensure compliance with the requirements of the specification. The purpose of the QC testing program is to verify that the reinforcement geosynthetic being supplied to the project is representative of the material used for performance testing and approval by the agency.

Conformance testing shall be performed as part of the manufacturing process and may vary for each type of product. As a minimum the following index tests shall be considered as applicable for an acceptable QA/QC program:

Property	Test Procedure
Specific Gravity (HDPE only)	ASTM D-1505
Wide Width Tensile	ASTM D-4595; GRI:GG1
Melt Flow (HDPE and PP only)	ASTM D-1238
Intrinsic Viscosity (PET only)	ASTM D-4603
Carboxyl End Group (PET only)	ASTM D-2455

6.0 Sampling, Testing, and Acceptance

Sampling and conformance testing shall be in accordance with ASTM D-4354. Conformance testing procedures shall be as established under 5.0. Geosynthetic product acceptance shall be based on ASTM D-4759.

The quality control certificate shall include:

- Roll numbers and identification
- Sampling procedures
- Result of quality control tests, including a description of test methods used.

7.0 Granular Backfill

The backfill shall conform to the specified fill under section 8.8 except that the maximum size of backfill shall be 20 mm, unless full scale installation damage tests are conducted in accordance with ASTM D-5818.

Additional requirements include:

- pH > 4.5 < 9

8.11 CONSTRUCTION SPECIFICATIONS FOR REINFORCED SLOPE SYSTEMS

The recent 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 that 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:

(a) Specification Guidelines for RSS Construction (Agency design).
(b) Specifications for Erosion Control Mat or Blanket.
(c) Specifications for Geosynthetic Drainage Composite.
(d) Specification Guidelines for Proprietary Geosynthetic RSS Systems.

a. Specification Guidelines For RSS Construction (Agency Design)

Description

Work shall consist of furnishing geosynthetic soil reinforcement for use in construction of reinforced soil slopes.

Geosynthetic Reinforcement Material

The specific geosynthetic reinforcement material and supplier shall be preapproved by the agency as outlined in the agency's reinforced 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 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.

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.

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. 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. However, if the contractor is unable to complete a required length with a single continuous length of geogrid, a joint may be made with the engineer's approval. Only one joint per length of geogrid shall be allowed. This joint shall be made for the full width of the strip by using a similar material with similar strength. Joints in geogrid reinforcement shall be pulled and held taut during fill placement. Joints shall not be used with geotextiles.

In the case of 100% coverage in plan view, adjacent strips need not be overlapped. The minimum horizontal coverage is 50 percent, with horizontal spacings between reinforcement no greater than 1 m. Horizontal coverage of less than 100 percent shall not be allowed unless specifically detailed in the construction drawings.

- 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 150 mm of soil has been placed.

- 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 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 m^3 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 300 mm maximum lift thickness where heavy compaction equipment is to be used, and 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 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 16 kmh. 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 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 meter in-place.

Payment will be made under:

Pay Item			Pay Unit
Geogrid Soil Reinforcement	-	Type A	square meter
Geogrid Soil Reinforcement	-	Type B	square meter
or			
Geotextile Soil Reinforcement	-	Type A	square meter
Geotextile Soil Reinforcement	-	Type B	square meter

Table S-1. Required Geotextile Properties.

Geotextile	Ultimate Strength ASTM 4595[1] (T_{ULT})	Long-Term Strength[2] (T_{al})	Pullout Resistance Factor [3] F*	For use with these Fills[4]
A				GW-GM
A				SW-SM-SC
B				GW-GM
B				SW-SM-SC

(1) Based on minimum average roll values (MARV) (kN/m)

(2) Long-Term strength (T_{al}) based on (kN/m)

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \ x \ RF_{ID} \ x \ RF_{D}}$$

where RF_{CR} is developed from creep tests performed in accordance with ASTM D 5262, 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.

(3) Pullout Resistance Factor developed in accordance with chapter 3 of this manual.

(4) Unified Soil Classification

Table S-2. Required Geogrid Properties.

Geogrid	Ultimate Strength GRI:GG1 (T_{ult})	Long-Term Strength[2] (T_{al})	Pullout Resistance Factor [3] F*	For use with these Fills[4]
A				GW-GM
A				SW-SM-SC
B				GW-GM
B				SW-SM-SC

(1) Based on minimum average roll values (kN/m)

(2) Long-Term strength (T_{al}) based on (kN/m)

$$T_{al} = \frac{T_{ULT}}{RF_{CR} \; x \; RF_{ID} \; x \; RF_{D}}$$

where RF_{CR} is developed from creep tests performed in accordance with ASTM D-5262, 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.

(3) Pullout Resistance Factor developed in accordance with chapter 3 of this manual.

(4) Unified Soil Classification

b. 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.

Erosion Control Materials

The specific erosion control material and supplier shall be preapproved by the Agency according to their policy on reinforced slopes.

Geosynthetic (Permanent) Erosion Mat. The geosynthetic erosion mat shall be:

[insert approved materials ***which meet the project requirements]***

Degradable (Temporary) Erosion Blanket. The degradable erosion blanket shall be:

[insert approved materials ***which meet the project requirements]***

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 D-4759, shall be used for conformance.

Construction

Delivery, Storage, and Handling. The contractor shall check the erosion control material upon delivery to ensure that the proper material has been received. During all periods of shipment and storage, the erosion mat shall be protected from temperatures greater than 60° C, mud, dirt, and debris. Follow manufacturer's recommendations in regards to protection from direct sunlight. At the time of installation, the erosion mat/blanket shall be rejected if it has defects, tears, punctures, flaws, deterioration, or damage incurred during manufacture, transport, or storage. If approved by the engineer, torn or punctured sections may be removed by cutting a cross section of the mat out. The remaining ends should be overlapped and secured with 500 mm pins. Any erosion mat/blanket damaged during storage or installation shall be replaced by the contractor at no additional cost to the owner.

Placement. 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 on maximum 0.5 m centers. (Topsoil, if required by construction drawings, placed over final grade prior to installation of the erosion control material shall be limited to a depth not exceeding 75 mm.)

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 minimum of 300 mm length recommended, and shall be spaced as designated on the construction drawings, with a maximum spacing of 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 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 are an incidental item.

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 meter
and/or	
Degradable (Temporary) Erosion Control Blanket	square meter

c. 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 (1995).]

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 D-4759, shall be used for conformance.

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 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 of 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 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 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 300 mm wide strip of geotextile may be used in the same manner, fastening it to the exposed fabric 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.

Table S3
Minimum Physical Property Criteria
For Geosynthetic Drainage Composites In
Reinforced Soil Slopes

PROPERTY	TEST METHOD	VALUE[1]
Composite		
Flow Capacity[2]	ASTM D 4716	___ m^2/s width (min)
Geotextile		
AOS[3]	ASTM D 4751	___ Max. Diameter (mm)
Permeability[4]	ASTM D 4491[5]	_____ m/s
Trapezoidal Tear	ASTM D 4533	
CLASS 2[6]		250 N
CLASS 3[7]		180 N
Grab Strength	ASTM D 4632	
CLASS 2[6]		700 N
CLASS 3[7]		500 N
Puncture	ASTM D 4833	
CLASS 2[6]		250 N
CLASS 3[7]		180 N
Burst	ASTM D 3786	
CLASS 2[6]		1300 kPa
CLASS 3[7]		950 Kpa

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 criteria for improved clogging resistance. See FHWA Geosynthetic Design and Construction Guidelines (1995) 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).

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:

Pay Item	Pay Unit
Geosynthetic Drainage Composite	square meter

d. 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 meter 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 meter in place.

Payment will be made under:

Pay Item	Pay Unit
Geosynthetic RSS System	Vertical square meter

CHAPTER 9

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. Most material suppliers provide training for construction of their systems. A checklist of general requirements for monitoring and inspecting MSE and RSS systems is provided in table 13.

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 9.1, preconstruction reviews.
- Section 9.2, prefabricated materials inspection.
- Section 9.3, construction control.
- Section 9.4, performance monitoring programs.

9.1 PRECONSTRUCTION REVIEWS

Prior to erection of the structure, personnel responsible for observing the field construction of the retaining structure should become thoroughly familiar with the following items:

- The plans and specifications.

Table 13. MSE/RSS field inspection checklist.

☐ 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 protections systems
- 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. Review construction sequence for the reinforcement system.**

☐ 4. **Check site conditions and foundation requirements. Observe:**
- preparation of foundations
- facing 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
- imperfect molding
- honey-combing
- severe cracking, chipping or spalling
- color of finish variation
- out-of-tolerance dimensions
- 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 (typically 19 mm ± 6 mm)
- precast face panels: (6 mm per m horizonal and vertical; 4 mm per m overall vertical)
- wrapped face walls: (15 mm per m horizontal and vertical; 8 mm overall vertical)
- line and grade

- The site conditions relevant to construction requirements.
- Material requirements.
- Construction sequences for the specific reinforcement system.

a. Plans and Specifications

Specification requirements for MSE and RSS are reviewed in chapter 8. 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 and unique and complex project details identified and reviewed with the designer and contractor, if possible. Special attention should be given to the construction sequence, corrosion protection systems for metallic reinforcement, special placement requirements to reduce construction damage for polymeric 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.

b. 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.

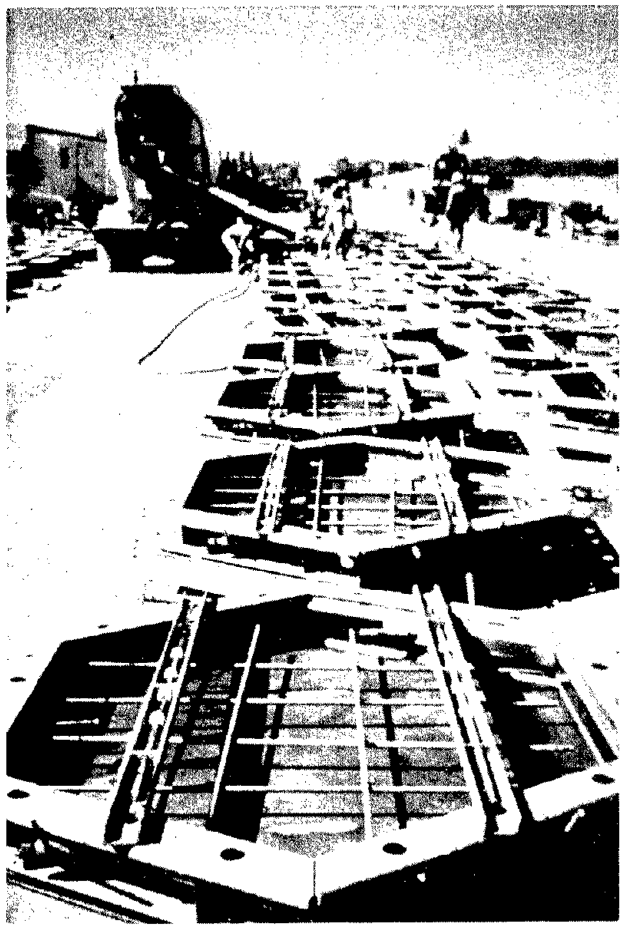
The groundwater level found in the site investigation should be reviewed along with levels of any nearby bodies of water that might affect drainage requirements. Slopes into which a cut is to be made should be carefully observed, especially following periods of precipitation, for any signs of seeping 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 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. Arrangement is generally provided for drainage to the base of the fill as shown on figures 42 and 69, to prevent water exiting the face of the wall and causing 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.

9.2 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 73. Material acceptance should be based on a combination of material testing, certification, and visual observations.



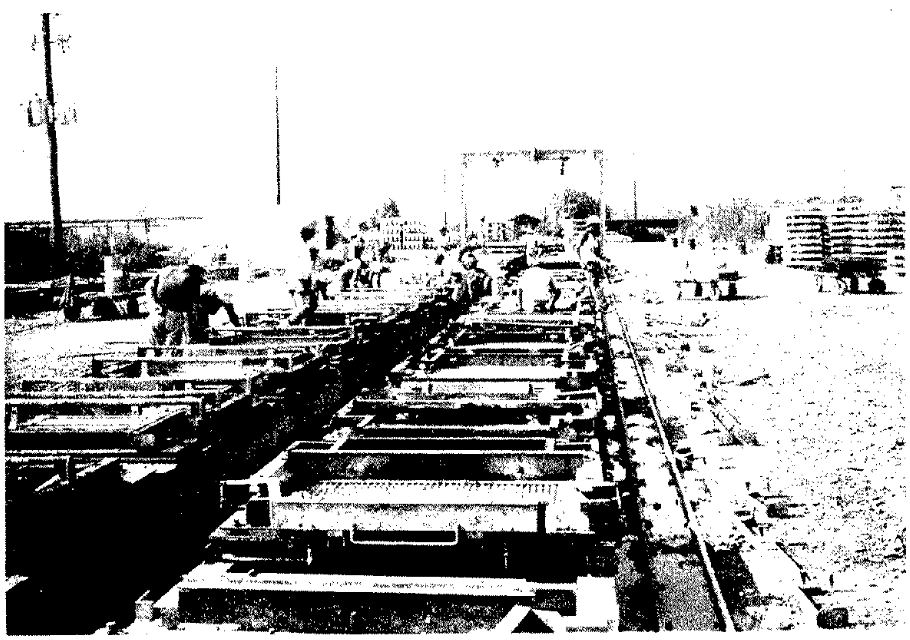


Figure 73. Casting yard for precast facing elements.

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.

a. **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. To minimize corrosion, it is especially important that coil embeds, tie strip guides, and other connection devices do not contact or be attached to the facing element reinforcing steel.

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.
- Imperfect molding.
- Honey-combing.
- Severe cracking, chipping, or spalling.
- Color of finish variation on the front face.
- Out-of-tolerance dimensions.
- Misalignment of connections.

The following maximum facing element dimension tolerances are usually specified for precast concrete:

- Overall dimensions - 13 mm.
- Connection device locations - 25 mm.
- Element squareness - 13 mm difference between diagonals.

- Surface finish - 2 mm in 1 m (smooth surface).

- Surface finish - 5 mm in 1 m (textured surface)

In cases where repair to damaged facing elements is possible, it should be accomplished to the satisfaction of the inspector.

For drycast modular blocks, it is essential that compressive strengths and water absorption by carefully checked on a lot basis. The following dimensional tolerances are usually specified:

- Overall dimensions - $\pm$ 3.2 mm

- Height of each block - $\pm$ 1.6 mm

b. **Reinforcing Elements**. Reinforcing elements (strips, mesh, sheets) should arrive at the project site securely bundled or packaged to avoid damage (see figure 74). 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). Material verification is especially important for geotextiles and geogrids where many product styles look similar but have different properties. Mesh reinforcement should be checked for gross area and length, width, and spacing of transverse members. For strip reinforcements, the length and thickness should be checked. Geogrids or geotextile samples should be sent to the laboratory for verification testing.

Protective coatings, i.e., galvanization (thickness 610 gm/m) or epoxy (thickness 18 mils [457 μm]), should be verified by certification or agency conducted tests and checked for defects.

c. **Facing Joint Materials**. Bearing pads (cork, neoprene, SBR rubber), joint filler (synthetic foam) and joint cover (geotextile) should be properly packaged to minimize damage in unloading and handling. For example, polymer filler material and geotextiles must be protected from sunlight during storage.

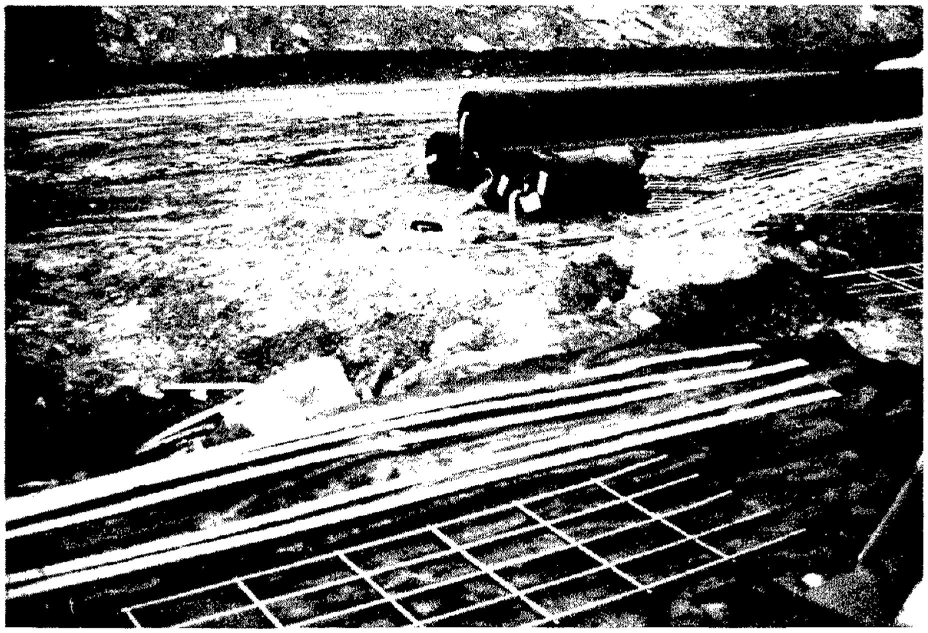


Figure 74. Inspect reinforcing elements.

Although these items are often considered as miscellaneous, it is important for the inspector to recognize that use of the wrong material or its incorrect placement can result in significant structure distress.

d. **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 properties. 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 of one test per 1500 m^3 of material placed and whenever the appearance and behavior of the backfill changes noticeably.

9.3 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 follows with requirements that apply to RSS systems noted.

a. Leveling Pad

A concrete leveling pad should have minimum dimensions of 150 mm thick by 300 mm wide and should have a minimum 13.8 MPa compressive strength. Cast-in-place pads should cure a minimum of 12 hours before facing panels are placed. Careful inspection of the leveling pad to assure correct line, grade, and offset is important. A vertical tolerance of 3 mm to the design elevation is recommended. If the leveling pad is not at the correct elevation, the top of the wall will not be at the correct elevation. An improperly placed leveling pad can result in subsequent panel 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 wall construction. Typical installations are shown on figure 75.

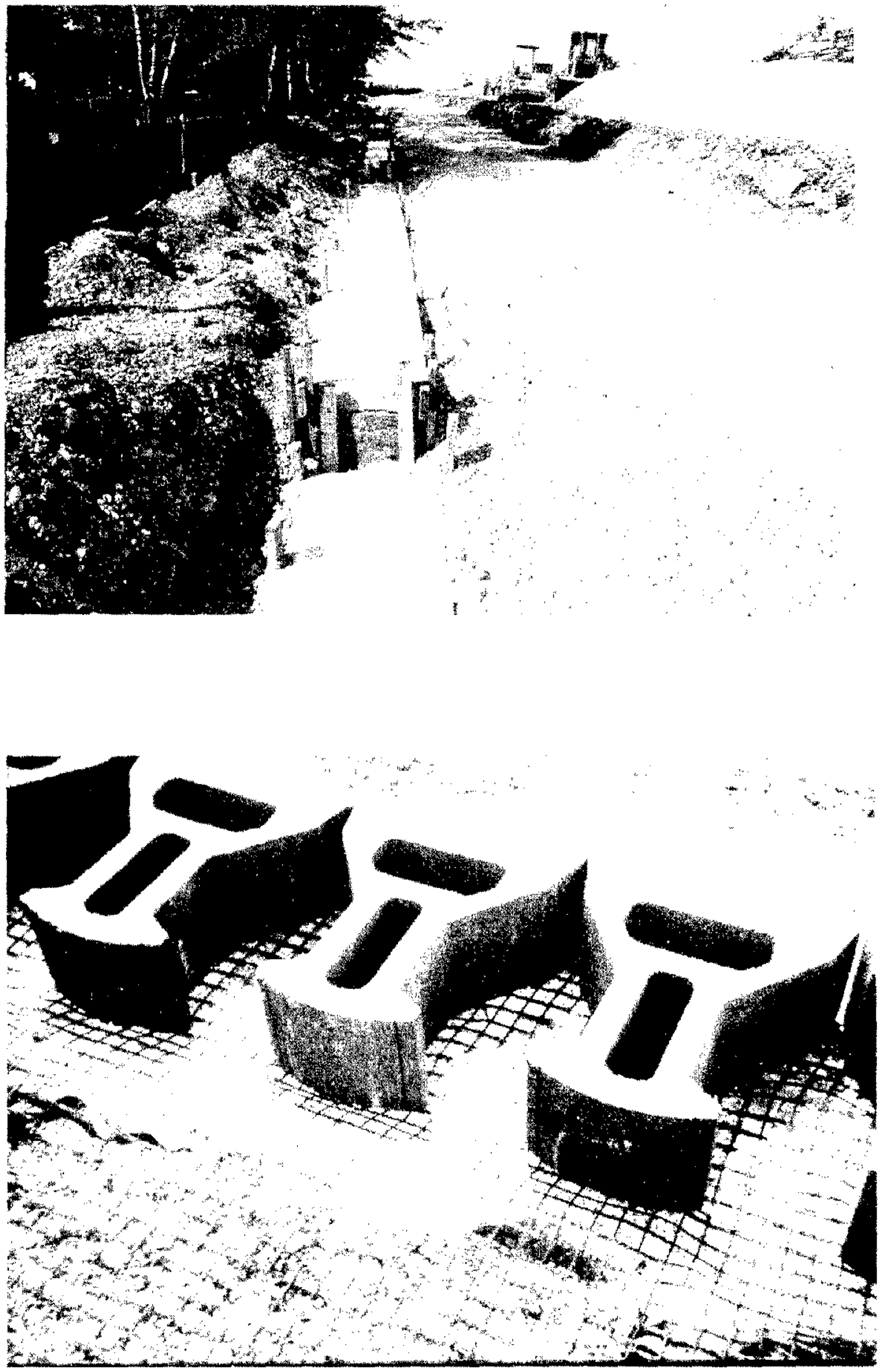


Figure 75. Leveling pads: a) concrete pad b) compacted gravel pad.

b. 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 76. 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. Most systems with segmental precast panels also have some form of construction alignment dowels between adjacent elements that aid in proper erection. Typical backward batter for segmental precast panels is 20 mm per meter of panel height.

Full height precast panels as shown on figure 77 are more susceptible to 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.

First Row of Facing Elements. Setting the first row of facing elements is a key detail as shown on figure 78. Construction should always begin adjacent to any existing structure and proceed toward the open end of the wall. The panels should be set directly on the concrete leveling pad. Horizontal joint material or wooden shims should not be permitted between the first course of panels and the leveling pad. Temporary wood wedges may be used between the first course of panels and the leveling pad to set panel batter, but they must be removed during subsequent construction. Some additional important details are:

- For segmental panel walls, panel spacing bars, which set the horizontal spacing between panels, should be used so that subsequent panel rows will fit correctly.

- The first row of panels must be continuously braced until several layers of reinforcements and backfills have been placed. Adjacent panels should be clamped together to prevent individual panel displacement.

- After setting the battering the first row of panels, horizontal alignment should be visually checked with survey instruments or with a stringline.

- When using full-height panels, initial bracing alignment and clamping are even more critical because small misalignments cannot be easily corrected as construction continues.

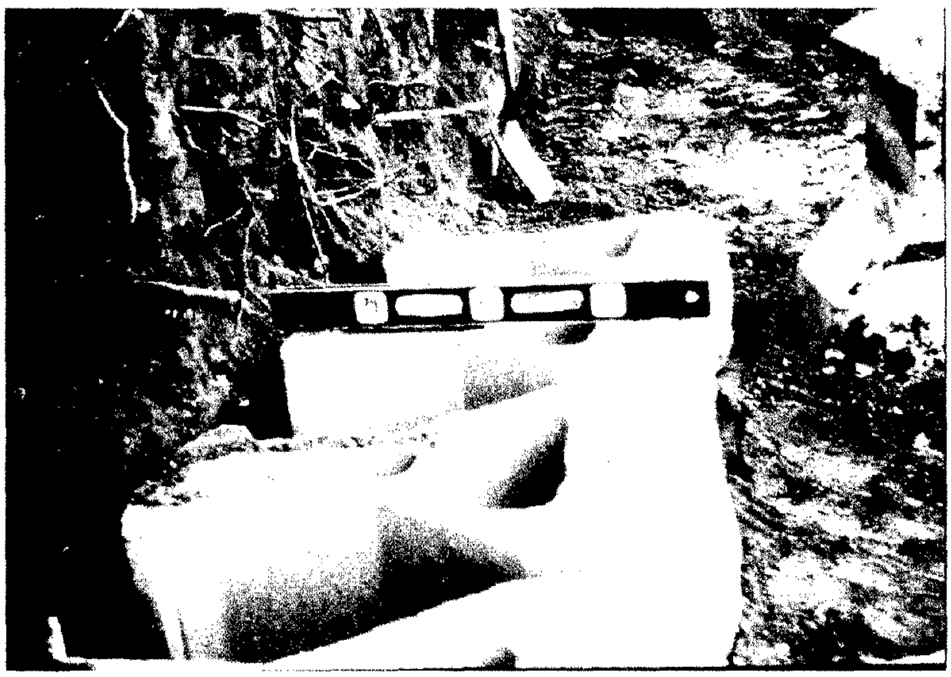
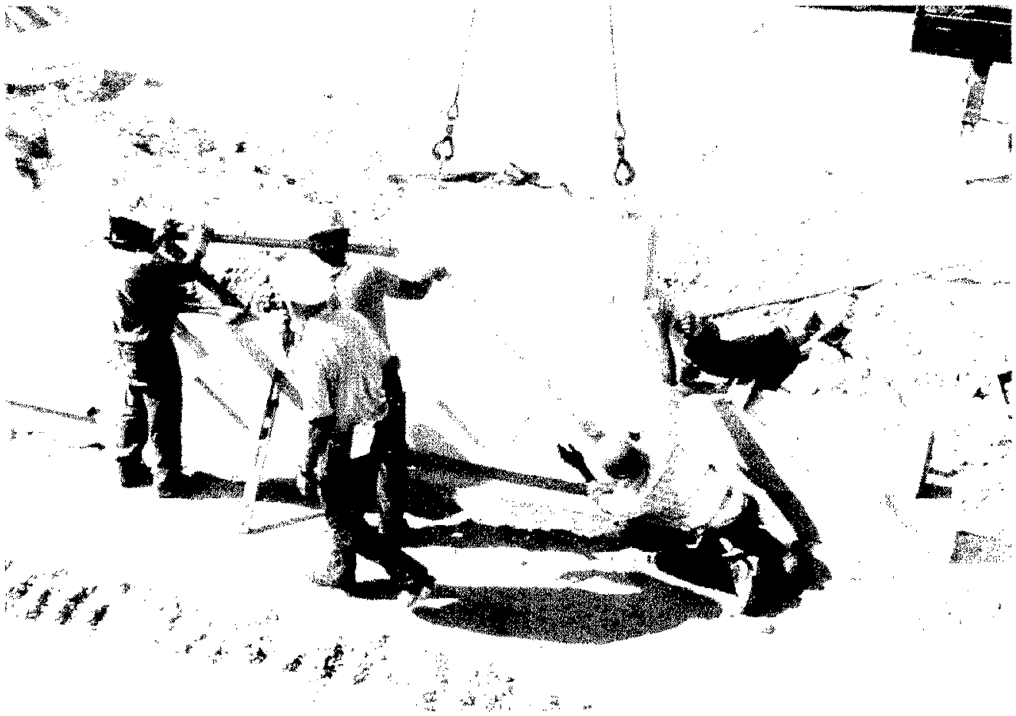


Figure 76. Checking facing element batter and alignment.

Figure 77. Full height facing panels require special alignment care.



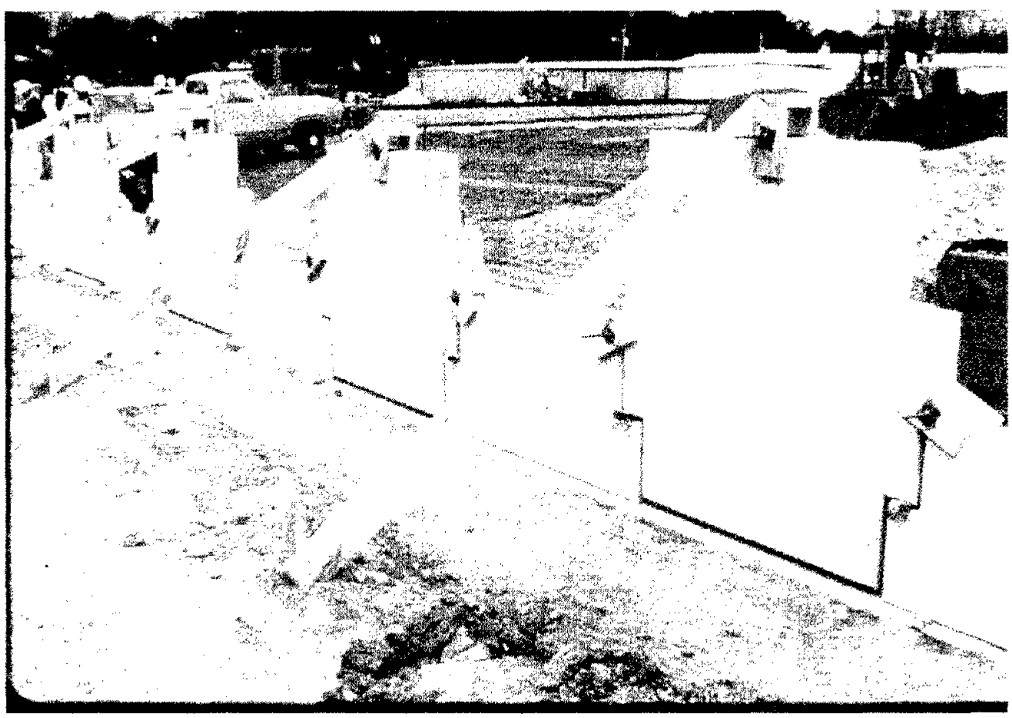


Figure 78. Setting first row of precast facing elements.

- Most MSE systems use a variety of panel types on the same project to accommodate geometric and design requirements (geometric shape, size, finish, connection points). The facing element types must be checked to make sure that they are installed exactly as shown on the plans.

c. **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 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 could result in significant settlement during periods of precipitation.

A density of 95 percent of T-99 maximum value is recommended for retaining walls and slopes, and 100 percent of T-99 is recommended for abutments and walls or slopes supporting structural foundations abutments. A procedural specification is preferable where a significant percentage of coarse material, generally 30 percent or greater retained on the 19 mm sieve, prevents the use of the AASHTO T-99 or T-180 test methods. In this situation, typically three 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 field trials.

Reinforced backfill should be dumped onto or parallel to the rear and middle of the reinforcements and bladed toward the front face as shown on figure 79. At no time should any construction equipment be in direct contact with the reinforcements because protective coatings and reinforcements can be damaged. Soil layers should be compacted up to or even slightly above the elevation of each level of reinforcement connections prior to placing that layer of reinforcing elements.

Figure 79. Placement of reinforced backfill.

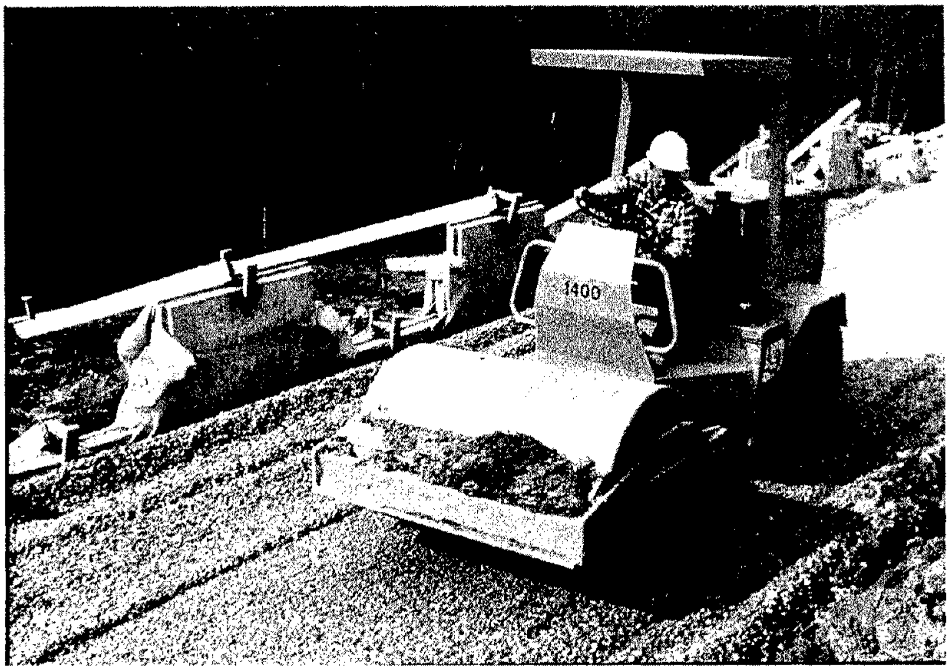
Compaction Equipment - With the exception of the 1-m zone directly behind the facing elements or slope face, large, smooth-drum, vibratory rollers should generally be used to obtain the desired compaction as shown on figure 80a. Sheepsfoot rollers should not be permitted because of possible damage to 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. The use of large vibratory compaction equipment with this type of backfill material will make wall alignment control difficult.

Within 1 m of the wall or slope face, use small single or double drum, walk-behind vibratory rollers or vibratory plate compactors as shown on figure 80b. 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 a chimney-shaped vertical void immediately behind the facing elements. Within this 1 m zone, quality control should be maintained by a methods specification such as three passes of a light drum compactor. 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 panel movement (modular panels) or structural damage (full-height, precast panels), and overstressing of reinforcement layers.

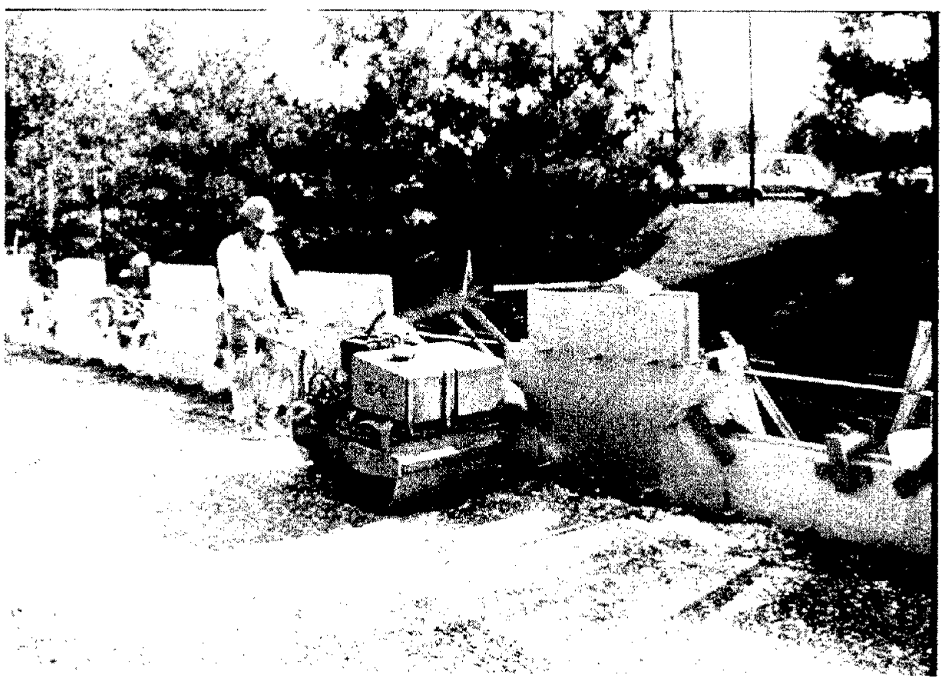
Inconsistent compaction and undercompaction caused by insufficient compactive effort or allowing the contractor to "compact" backfill with trucks and dozers will lead to gross misalignments and settlement problems and should not be permitted. Flooding of the backfill to facilitate compaction should not 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 every 1.5 m of wall height for every 30 m of wall is recommended.

d. 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, abutments and curved walls, for example, it may be permissible to skew the reinforcements from their design location in either the horizontal or vertical direction. In all cases, overlapping layers of reinforcements should be separated by a 75 mm minimum thickness of fill.



a.



b.

Figure 80. Compaction equipment showing a) large equipment permitted away from face and b) lightweight equipment within 1 m of the face.

Curved walls create special problems with MSE panel and reinforcement details. Different placement procedures are generally required for convex and concave curves. For reinforced fill systems with precast panels, joints will either be further closed or opened by normal facing movements depending on whether the curve is concave or convex.

Other difficulties 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 can also be performed but is risky and may damage reinforcing elements.

Connections. Each MSE system has a unique facing connection detail. Several types of connections are shown on figure 81. All connections must be made in accordance with the manufacturer's recommendations. For example on Reinforced Earth structures bolts must fit and be located between tie strips, be perpendicular to the steel surfaces, and be seated flush against the flange to have full bearing of the bolt head. Nuts are to be securely tightened.

Flexible reinforcements, such as geotextiles and geogrids, usually require pretensioning to remove any slack in the reinforcement or in the panel. 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.

e. 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.

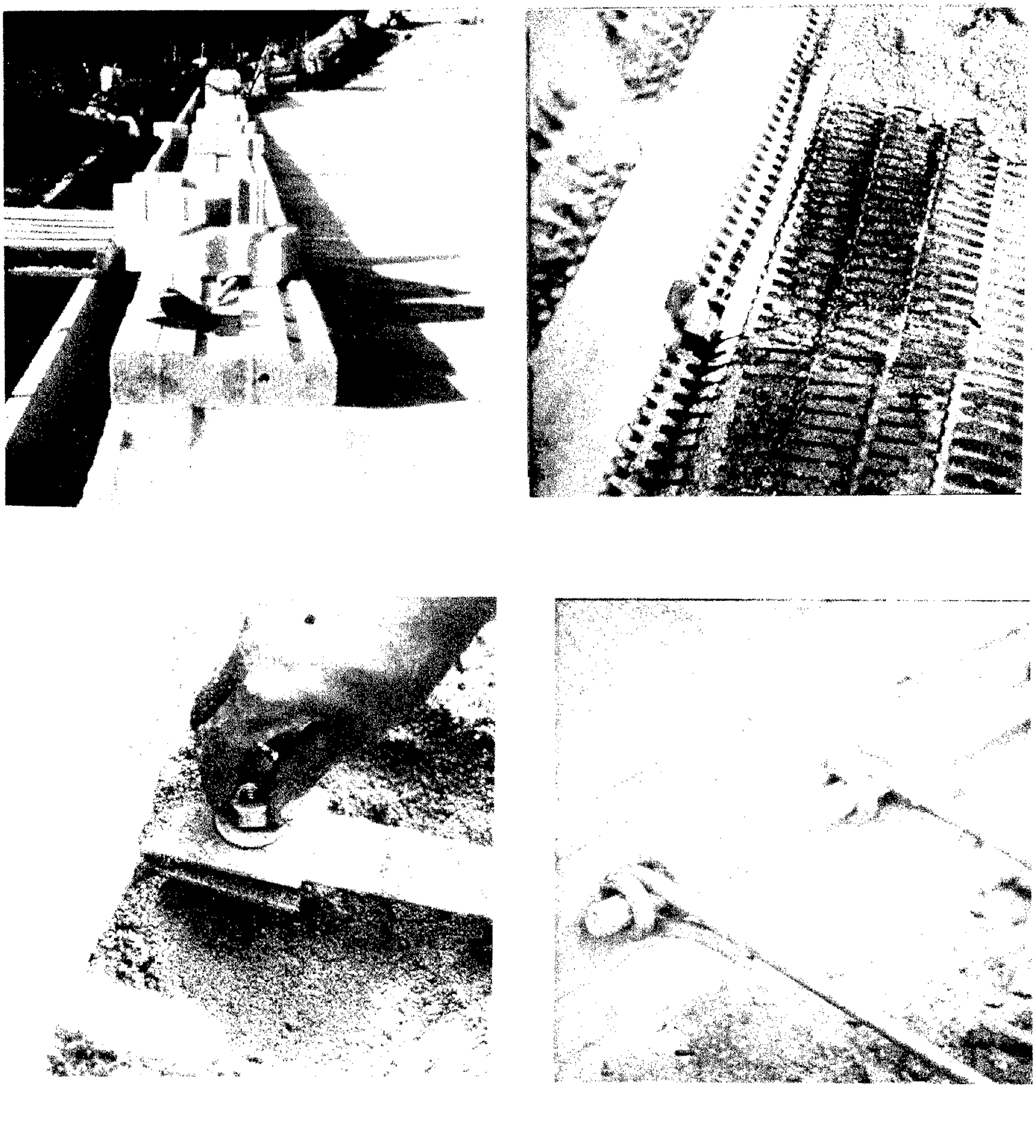


Figure 81. Facing connection examples.

The following alignment tolerances are recommended:

- Adjacent facing panel joint gaps (all reinforcements) - 19 mm $\pm$ 6 mm.
- Precast face panel (all reinforcements) - 6 mm per m (horizontal and vertical directions).
- Wrapped face walls and slopes (e.g., welded wire or geosynthetic facing) - 15 mm per m (horizontal and vertical directions).
- Wrapped face walls and slopes (e.g., welded wire or geosynthetic facing) overall vertical - 8 mm per m.
- Reinforcement placement elevations - 25 mm of connection elevation.

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.

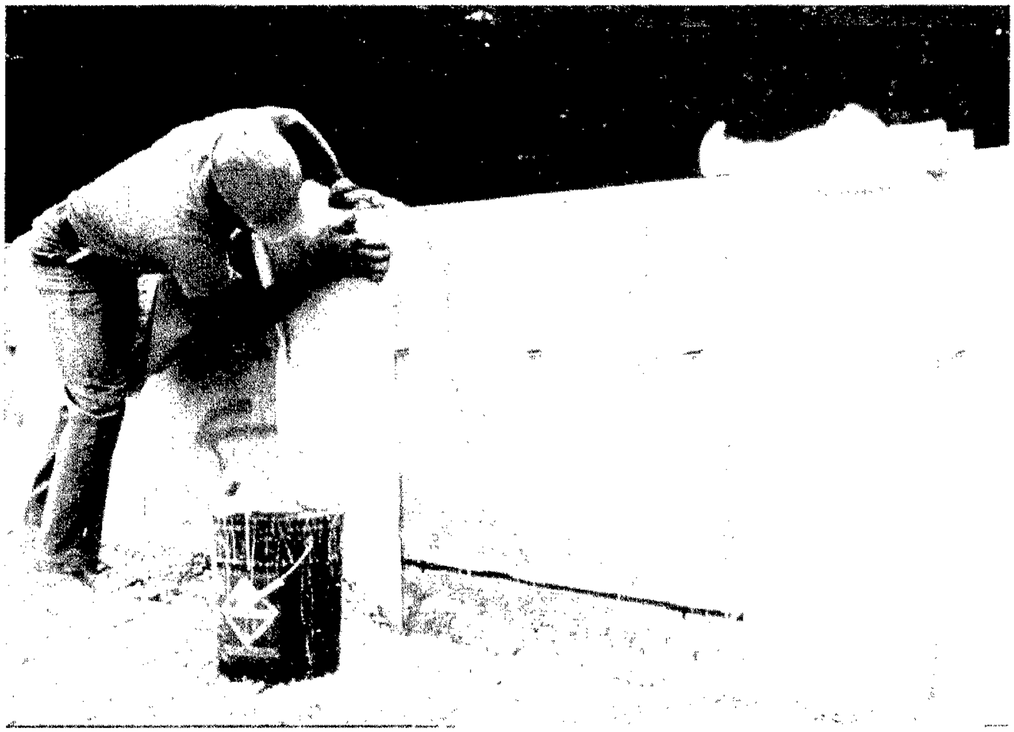
Facing elements that are out of alignment should not be pulled back into place because this may damage the panels and reinforcements and, hence, weaken the system. Appropriate measures to correct an alignment problem are the removal of reinforced fill and reinforcing elements, followed by the resetting of the panels. Decisions to reject structure sections that are out of alignment should be made rapidly because panel resetting and reinforced fill handling are time consuming and expensive. Occasionally, lower modular panels may experience some movement after several lifts of panels have been placed. This could be due to foundation settlement, excess moisture content following heavy rain, or excessive compaction. Construction should be stopped immediately and the situation evaluated by qualified geotechnical specialists when these "post erection" deformations occur.

Improper horizontal and vertical joint openings can result in face panel misalignment, and cracking and spalling due to point stresses. Wedging of stones or concrete pieces to level face panels should not be permitted. All material suppliers use bearing pads on horizontal joints between segmental facing panels to prevent point stresses (cork, neoprene, or rubber are typically used). These materials should be installed in strict accordance with the plans and specifications, especially with regard to thickness and quantity. Other joint materials are used to prevent point stresses and erosion of fill through the facing joints (synthetic foam and geotextiles details are typically used). Excessively large panel joint spacings or joint openings that are highly variable result in a very unattractive end product. Bearing pads and geotextile joint covers are shown on figure 82.

Wooden wedges shown on figure 74 placed during erection to aid in alignment should remain in place until the third layer of modular 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.

Table 14 gives a summary of several out-of-tolerance conditions and their possible causes.



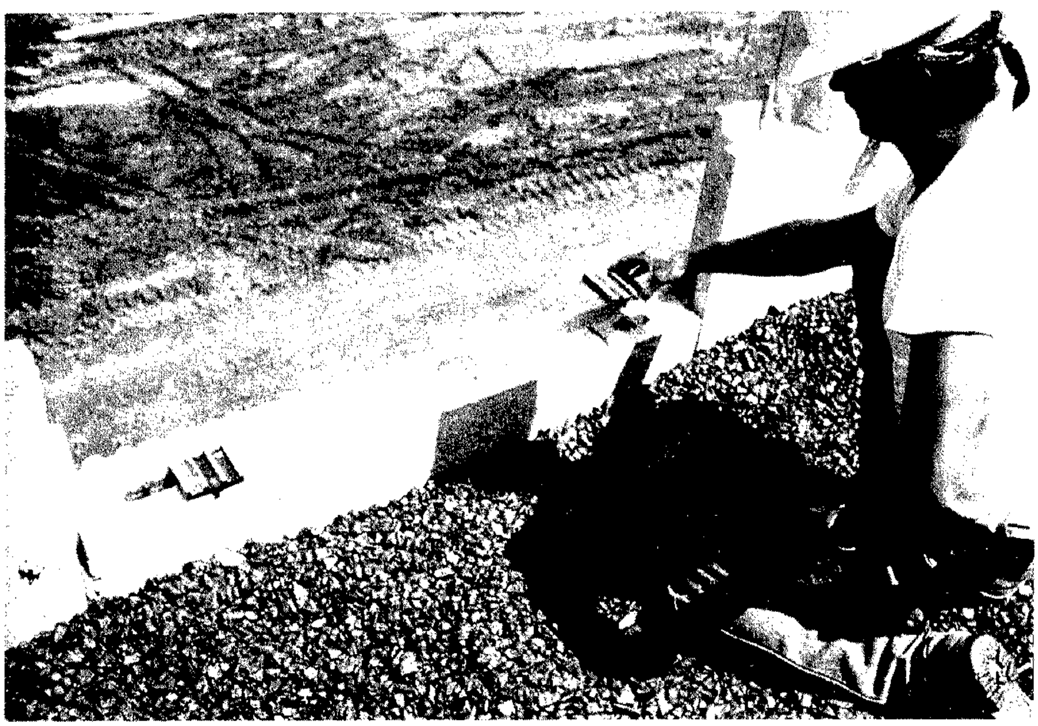


Figure 82. Geotextile joint cover and neoprene pads.

Table 14. 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.

CONDITION	POSSIBLE CAUSE
1. Distress in wall: a. Differential settlement or low spot in wall. b. Overall wall leaning beyond vertical alignment tolerance. c. Panel contact, resulting in spalling/chiping.	1. a. Foundation (subgrade) material too soft or wet for proper bearing. Fill material of poor quality or not properly compacted.
2. First panel course difficult (impossible) to set and/or maintain level. Panel-to-panel contact resulting in spalling and/or chiping.	2. a. Leveling pad not within tolerance.
3. Wall out of vertical alignment tolerance (plumbness), or leaning out.	3. a. Panel not battered sufficiently. b. Oversized backfill and/or compaction equipment working within 1 m zone of back of wall facing panels.

(cont'd.)

CONDITION	POSSIBLE CAUSE
	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 compacted above reinforcing elements.
	e. Excessive or vibratory 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 back of wall, 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.

(cont'd.)

CONDITION	POSSIBLE CAUSE
4. Wall out of vertical alignment tolerance (plumbness) or leaning in.	4. a. Excessive batter set in panels 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. 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.
	c. Failure to use spacer bar.
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.

9.4 PERFORMANCE MONITORING PROGRAMS

Since MSE technology is 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 and where degradation/corrosion rates of reinforcements require monitoring because of the use of marginal fills or anticipated changes in the in situ regime. Under the outlined conditions the monitoring can be used to:

- Confirm design stress levels and monitor safety during construction.
- Allow construction procedures to be modified for safety or economy.
- Control construction rates.
- Enhance knowledge of the behavior of MSEW or RSS structures to provide a base reference for future designs, with the possibility of improving design procedures and/or reducing costs.
- Provide insight into maintenance requirements, by long-term performance monitoring.

Degradation/Corrosion monitoring schemes are fully outlines in the companion *Corrosion/Degradation* document.

a. 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.

For all structures, important parameters that should 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.
- Total horizontal stress within the backfill and at the back of the reinforced wall section.
- Aging condition of reinforcement such as corrosion losses 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.

b. Limited Monitoring Program

Limited observations and monitoring will typically 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.

- 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 rigid reinforcement and H/75 for flexible reinforcement. Tilting due to differential lateral movement from the bottom to the top of the wall would be anticipated to be less than 4 mm per m of wall height for either system. Postconstruction 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.

c. 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 15.

Table 15. Possible instruments for monitoring reinforced soil structures.

PARAMETERS	POSSIBLE INSTRUMENTS
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 (e.g. transit) 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

(cont'd)

PARAMETERS	POSSIBLE INSTRUMENTS
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
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

d. 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.

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 30 cm 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 RD89-043.

e. 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

1. Elias V. "Corrosion/Durability of Soil Reinforced Structures" *FHWA RD 89-186*. Washington D.C. (1989)

2. Simac, M.R., Bathhust R.J. Berg R.R., Lothspeich S.E. "Design Manual for Segmental Retaining Walls" *National Concrete Masonry Association*. Herndon, VA (1993)

3. Berg, R.R., Chouery-Curties, V.E., and Watson, C.H., "Critical Failure Planes in Analysis of Reinforced Slopes," *Proceedings of Geosynthetics '89, Volume 1,* San Diego, February, 1989.

4. Wright, S.G. and Duncan, J.M., "Limit Equilibrium Stability Analyses for Reinforced Slopes," presented at the *Transportation Research Board 70th Annual Meeting,* January 13-17, Washington D.C., 1990.

5. Jurgenson, L., "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, 1934, pp 134-217.

6. Silvestri, V., "The Bearing Capacity of Dykes and Fills Founded on Soft Soils of Limited Thickness", *Canadian Geotechnical Journal, Volume 20,* Number 3, 1983, pp. 428-436.

7. Bonarparte, R., Holtz, R.D. and Giroud, J.P., "Soil Reinforcement Design Using Geotextiles and Geogrids", Geotextile Testing and The Design Engineer, *ASTM, Special Technical Publication 952,* 1987.

8. Yarger and Barne "Montana Department of Transportation's Introduction to Geogrid Use for Steepened Embankment Design", *Proceedings of Geosynthetics '93,* Vancouver, B.C., Canada, 1993.

9. Zornberg, Barrows, Christopher and Wayne "Construction and Instrumentation of a Highway Slope Reinforced with High-Strength Geotextiles", *Proceedings of Geosynthetics '95*, Nashville, Tenn., 1995.

REFERENCES (continued)

10. ________ "Reinforced Soil Highway Slopes" *Tensar Technical Note.* Tensar Earth Technologies (1990). Atlanta GA.

11. Wayne and Wilcosky "An innovative use of a nonwoven geotextile in the repair of Pennsylvania State Route 54", *Geotechnical Fabrics Report*, Volume 13, Number 2, 1995.

12. Werner, G. and Resl, S., "Stability Mechanisms in Geotextile Reinforced Earth-Structures", *Proceedings of the 3rd International Conference on Geotextiles,* Vienna, Austria, Volume II, 1986, pp. 465-470.

13. Ruegger, R., "Geotextile Reinforced Soil Structures on which Vegation can be Established", *Proceedings of the 3rd International Conference on Geotextiles,* Vienna, Austria, Volume II, 1986, pp. 453-458.

14. Leshchinsky, E. and Boedeker, R.H., "Geosynthetic Reinforced Soil Structures", *Journal of Geotechnical Engineering, ASCE, Volume 115, Number 10,* 1989, pp. 1459-1478.

15. Jewell, R.A., "Revised Design Charts for Steep Reinforced Slopes, Reinforced Embankments: Theory and Practice in the British Isles", *Thomas Telford,* London, U.K., 1990.

16. Silvestri, V. " The Bearing Capacity of Dikes and Fills Founded on Soft Soils of Limited Thickness" *Canadian Geotechnical Journal Vol. 20 No. 3* (1983)

17. Chen, Y-H and G.K. Cotton, "Design of Roadside Channels with Flexible Linings", *FHWA-IP-87-7,* HEC15, PB89-122584, 1987.

18. Chen, Y-H and Anderson, B.A., Final Report - "Development of a Methodology for Estimating Embankment Damage Due to Flood Overtopping", *DTFH61-82-00104,* FHWA, Washington D.C., 1986.

19. Collin, J.G., "Controlling Surficial Stability Problems on Reinforced Steepened Slopes" *Geotechnical Fabrics Report* IFAI, 1996

APPENDIX A

DETERMINATION OF PULLOUT RESISTANCE FACTORS

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 (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. 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 1992 AASHTO for pullout design. The variables have been rearranged in the procedures provided in this appendix to unify the approach to pullout design as much as possible. However, the methodology provided herein for pullout design of inextensible and extensible grids and bar mats differs slightly from what is currently in the *1992 AASHTO Bridge Specifications* based on more recent data and analysis (Christopher, 1993).

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.

A.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:

$$F^* = \text{Frictional Resistance} + \text{Passive Resistance}$$
$$= \operatorname{Tan} \rho + F_q \alpha_\beta$$

where Tan ρ 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.

A.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 ρ_{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. An ASTM standard for pullout testing is currently under development. Until this standard is finalized, it is recommended that test procedures GRI GG-5 and GRI GT-6 using the controlled strain rate method, be used in the interim as pullout test procedures. 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 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., "Controlled Strain Rate Method for Short-Term Testing" per GGI:GG5 and GRI:GT6) and through-the-junction creep testing of the geogrid per GRI:GG3a.

- Using quick effective stress pullout tests (i.e., "Controlled Strain Rate Method for Short-Term Testing" per GRI:GG5 and GRI:GT6), but with the geogrid transverse ribs severed.

- Using quick effective stress pullout tests (i.e., "Controlled Strain Rate Method for Short-Term Testing" per GRI:GG5 and GRI:GT6) if the summation of the shear strengths of the joints occurring in a 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 (Koerner, 1988).

- Conduct long-term effective stress pullout tests of the entire geogrid structure in accordance with the constant stress (creep) method of GRI:GG5 (Koerner, 1991).

For pullout tests, a normalized pullout versus mobilized reinforcement length curve should be established as shown in figure A.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.

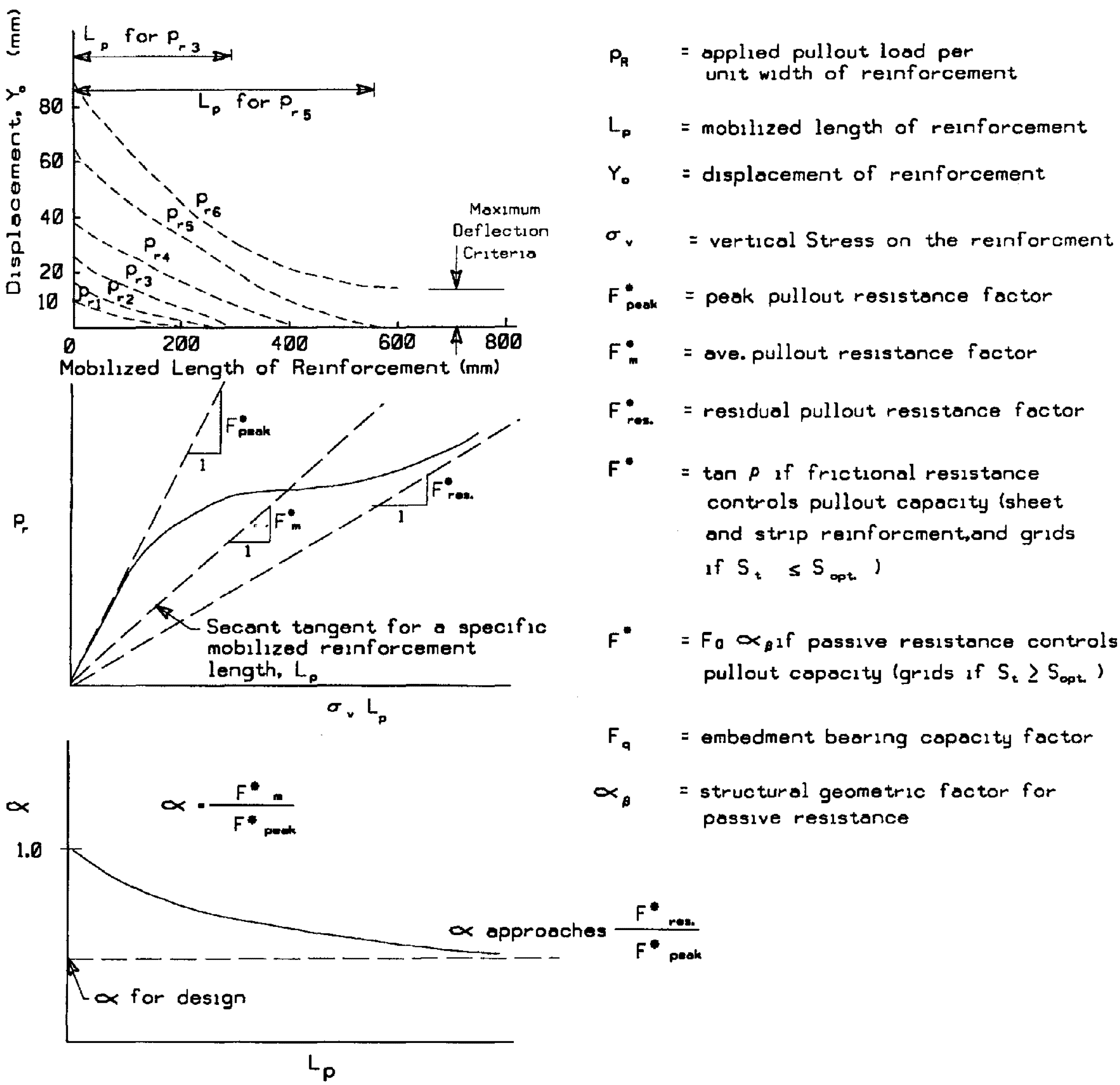


Figure A.1. Experimental procedure to determine F* and ∝ for Soil Reinforcement using pullout test.

It is recommended that for inextensible reinforcements, a maximum deflection of 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 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 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 A.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 A.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 A.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.

A.3 PULLOUT 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 resulting from pullout or rupture can be determined from NCMA Test Method SRWU-1. This test method is reported in the NCMA Design Manual for Segmental Retaining Walls (Simac, et. al., 1993). In this test, a tensile load is applied to the free end of geosynthetic reinforcement while the geosynthetic is confined between two layers of segmental concrete facing blocks. The test is performed in a manner similar to a wide width test (ASTM D 4595), except that the distance between the edge of the facing blocks and the specimen clamps is 200 mm rather than 100 mm, and the specimen width is up to 1,000 mm. Furthermore, deformation of the reinforcement is measured where the geosynthetic exits the facing blocks rather than between the facing blocks and the clamps. Typically, a minimum of three facing blocks are placed side by side below the reinforcement and two blocks placed side by side above the reinforcement. Block placement should be similar to how they would be placed in an actual structure. For blocks which are greater than 500 mm in width, the number of blocks used could be reduced to two blocks below and one block above the reinforcement. A constant confining pressure is placed on the blocks. Once a reinforcement specimen is in place, the specimen is loaded at a rate of 20 mm per minute (i.e., 10%/minute), measuring both load and deformation, until the specimen physically begins to pull out from between the blocks or until the specimen ruptures (see figure A.2 for an example). These tests are conducted at multiple confining pressures to simulate the range of confining pressures anticipated in an actual structure so that the connection pullout/rupture strength can be determined for the design confining pressure (see figure A.3 for an example). The range of confining pressures selected for testing should be based on the hinge height for the facing configuration anticipated.

The following modifications are recommended for NCMA Test Method SRWU-1:

- "Zero" tension is defined as 1.1 kN/m.
- The specimen width should be a minimum of 750 mm and a maximum of 1,000 mm.
- The specimen width should be an exact multiple of the facing block width.

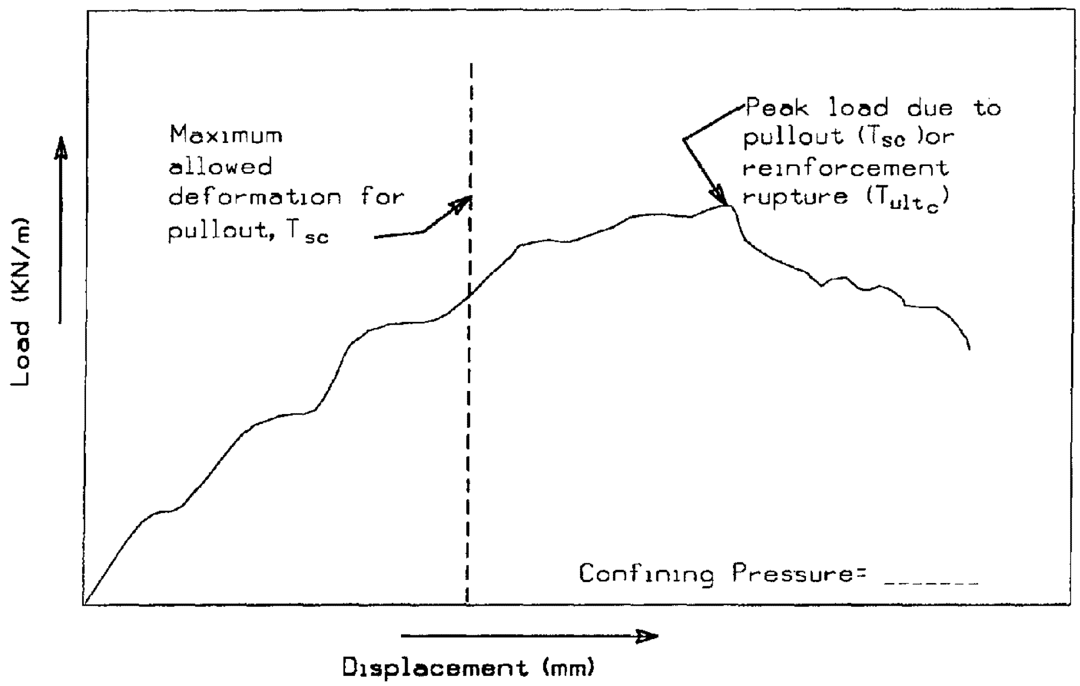


Figure A.2. Load versus displacement for connection strength test.

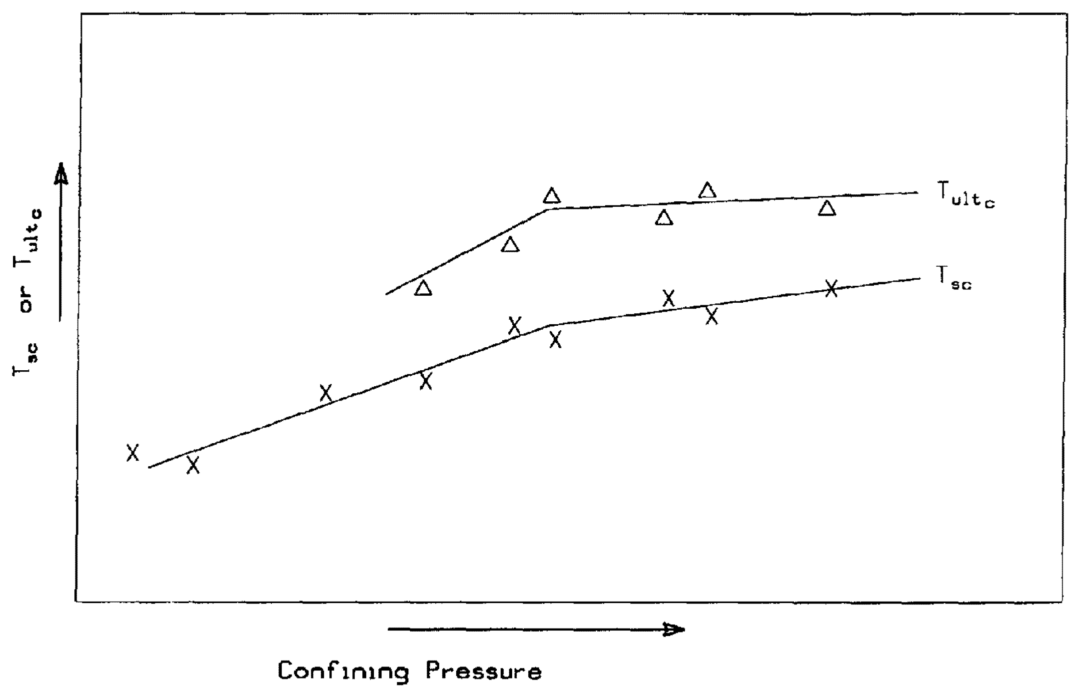


Figure A.3. Determination of T_{sc} and T_{ult_c} various facing block confining pressures.

From this test, the peak connection load and the load at a specified maximum deformation, in terms of load per unit reinforcement width, is obtained. At low confining stress, the peak connection load is governed by pullout without rupture of the reinforcement, whereas at moderate to high confining stresses, the peak connection load is governed by reinforcement rupture. T_{ultc} is defined as the load per unit reinforcement width which results in rupture of the reinforcement in this test at a specified confining pressure. Note that at low confining pressure, T_{ultc} may not occur. T_{sc} is defined as the load per unit of reinforcement width at a specified maximum deformation or at the peak pullout load, whichever occurs first. A maximum deformation of 20 mm is recommended for the determination of T_{sc}. The lessor of the loads at a deformation of 20 mm, and the peak pullout load if rupture does not occur, defines the pullout load. The reduction factors for connection design which include CR_u, the reduction factor to account for reduced ultimate strength resulting from the connection, and CR_s, the reduction factor to account for reduced strength due to connection pullout, are determined from the test data as follows:

$$CR_u = \frac{T_{ultc}}{T_{lot}} \tag{A-1}$$

$$CR_s = \frac{T_{sc}}{T_{lot}} \tag{A-2}$$

where, T_{lot} is the ultimate wide width tensile strength (ASTM D-4595) for the reinforcement material lot used for the connection strength testing, and other variables are as defined previously.

REFERENCES

AASHTO, 1992, *Standard Specifications for Highway Bridges, with 1994 Interims,* American Association of State Highway and Transportation Officials, Fifteenth Edition, Washington, D.C., USA, 686 p.

American Society for Testing and Materials (ASTM), 1994, *Annual Book of Standards,* Vol. 4.08 and 4.09, Soil and Rock, ASTM, Philadelphia, PA, USA.

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.

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 Dunnicliff, J. (1990), Reinforced Soil Structures, Vol. 1, Design and Construction Guidelines, Report to Federal Highway Administration, *No. FHWA-RD-89-043.*

Koerner, R. M. (1991), "Determination of the Pullout Resistance of Geogrids," *GRI Standard Practice GG-5.*

Koerner, R. M. (1991), Determination of the Pullout Resistance of Geotextiles," *GRI Standard Practice GT-6.*

Simac, M. R., Bathurst, R. J., Berg, R. R., and Lothspeich, S. E., 1993, *Design Manual for Segmental Retaining Walls (Modular Concrete Block Retaining Wall Systems),* First Edition, NCMA, Herndon, Virginia.

APPENDIX B

DETERMINATION OF CREEP STRENGTH REDUCTION FACTOR (RF_{CR})

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 minimum duration of 10,000 hours for a range of load levels in accordance with ASTM D 5262. 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 limit state design, the highest load level, designated T_l, which precludes both ductile and brittle creep rupture.

- 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.

- The creep reduction factor, RF_{CR}, is determined by comparing the long-term creep strength, T_l, to the ultimate tensile strength (ASTM D 4595) 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 which 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:

$$RF_{CR} = \frac{T_{ultlot}}{T_1} \qquad \text{(B-1)}$$

where, T_{ultlot} is the average lot specific ultimate tensile strength (ASTM D 4595) for the lot of material used for the creep testing.

At present, creep tests are conducted in-isolation (ASTM D 5262) rather than confined in-soil, even though in-isolation creep tests tend to overpredict creep strains and underpredict the true creep strength when used in a structure.

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 either stress rupture data or creep strain data. The specific steps required to determine T_1 differ substantially depending on which type of data is available. Creep strains are not typically monitored in stress rupture testing, although creep strain tests can be carried to rupture. Rupture data is necessary if the creep reduction factor for ultimate limit state conditions is to be determined. 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. However, extrapolation of creep strain data is complex and not fully defined. Therefore, no guidance is provided regarding extrapolation of creep strain data to determine T_1.

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; Koerner, 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 (Takaku, 1981; Popelar, et. al., 1991). Therefore, temperature accelerated creep data is strongly recommended for polyolefins. For polyester (PET) geosynthetics, limited evidence does appear to indicate that temperature increases of at least twice that needed for polyolefins to produce a given time acceleration may be feasible, based on data provided by den Hoedt, et. al., 1994. However, the stress rupture envelopes for PET geosynthetics tend to be flatter than polyolefin stress rupture envelopes, and accurate determination of time-shift factors may be difficult for PET geosynthetics. 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. A two log cycle extrapolation without elevated temperature data is an acceptable alternative for PET geosynthetics, provided an appropriate extrapolation safety factor is applied to account for any minor curvature in the long-term rupture envelope not observed in the 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. A two log cycle extrapolation without elevated temperature data is not recommended for polyolefin geosynthetics due to the potential for a "knee" to be present in the stress rupture envelope.

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 for polyolefins and 20° C for PET geosynthetics. The highest temperature tested, however, should be below any transitions for the polymer in question. If one uses test temperatures below 80° C for polypropylene (PP) and high density polyethylene (HDPE) and below 70° C for PET geosynthetics, significant polymer transitions will be avoided. 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.

B.2 STEP-BY-STEP PROCEDURES FOR EXTRAPOLATING STRESS RUPTURE DATA

Step 1: Plot the stress rupture data on a plot of log time to rupture versus log load level, as shown in figure B.1. Do this for each temperature in which creep rupture data is available. In general, 12 to 18 data points are required to establish a rupture envelope (Jewell and Greenwood, 1988; ASTM D 2837). The data points should be evenly distributed through each log cycle of time. Rupture points with a time to rupture of less than 5 to 10 hours should in general not be used, and at least one or two data points should have a time to rupture of 10,000 hours or more.

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.

For the elevated temperature rupture envelopes, it may not be necessary to establish the complete rupture envelope. If a knee is already present in the rupture envelope obtained at the design (ambient) temperature, only a few long-term rupture points need to be obtained at elevated temperature(s) to establish the slope of the envelope beyond the knee out to the desired design life. If a knee is not present in the ambient temperature rupture envelope, the elevated temperature stress rupture envelope(s) must be well enough defined to determine whether or not a knee is present.

Step 2: Extrapolate the stress rupture data. Stress rupture data can be extrapolated statistically using regression analysis (i.e., curve fitting) up to one log cycle for all geosynthetic polymers and up to 2 log cycles for PET geosynthetics. For PP and HDPE geosynthetics, stress rupture data at elevated temperatures should be obtained to allow time-temperature superposition principles to be used. Elevated temperature stress 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 B.1(a), the single time shift factor a_T will be adequate to perform the time-temperature superposition. If, however, the

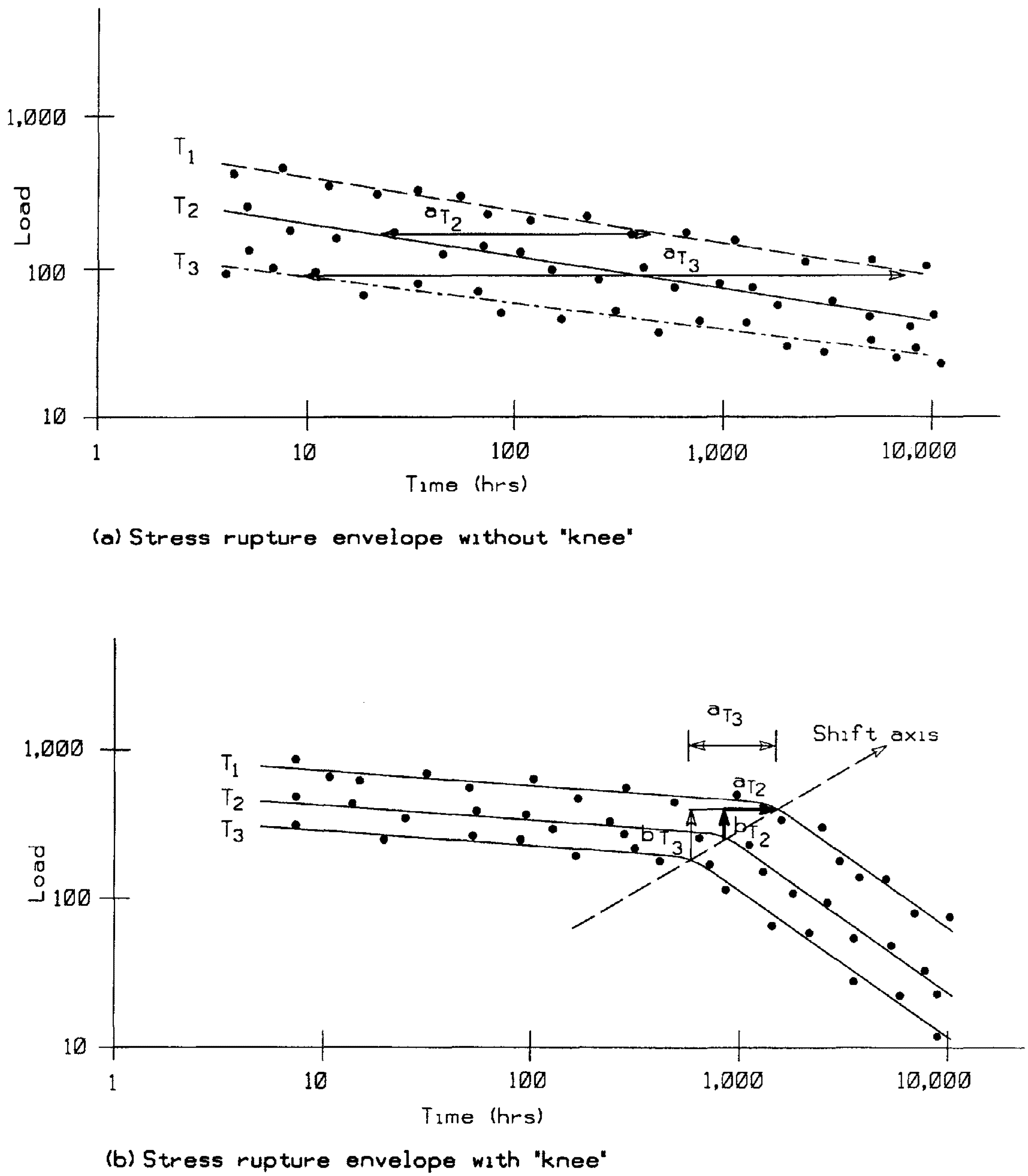


Figure B.1. Typical stress rupture data and the determination of shift factors for time-temperature superposition.

rupture envelope exhibits a "knee", resulting in a bilinear or curved envelope as illustrated in figure B.1(b), a vertical shift factor "b_T" along the load axis will also be required to make sure that the "knees" line up properly. In essence, the shift is performed along the shift axis shown in figure B.1(b). The shift axis simply connects the knee for each rupture envelope together.

The time to rupture for the elevated temperature rupture data is shifted in accordance with the following equation:

$$t_{amb} = (t_{elev})(a_T) \tag{B-2}$$

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. If a knee is present in the stress rupture envelope, the load for each elevated temperature rupture data point is also shifted using the following equation:

$$P_{amb} = (P_{elev})(b_T) \tag{B-3}$$

where, p_{amb} is the equivalent load level at in-situ (i.e., design) temperature at a given time to rupture, P_{elev} is the measured load level at elevated temperature at a given time to rupture, and b_T is the load level shift factor. The magnitude of the time shift and load shift factors can be determined graphically as illustrated in figure B.1(b). Adjust a_T and b_T such that the stress rupture envelopes at elevated temperature line up with the stress rupture envelope at the design (in-situ) temperature. If a knee in the stress rupture envelope only appears for the data obtained at the highest temperature, it must assumed that a knee in the rupture envelope must be possible at times beyond the available data for the lower temperature data as well. In this case, two options are available to determine a_T and b_T, considering that the slope of the shift axis must be determined:

- Obtain creep data at a temperature higher than the highest temperature previously tested.

- Assume that a knee in the rupture envelope occurs right at the end of the available data at the next lower temperature below the envelope which exhibited a knee.

Once rupture envelope knee locations at two temperatures have been established, the slope of the shift axis can be determined, and a_T and b_T can be determined as shown in figure B.1(b).

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 B.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:

$$T_1 = P_{cl} / (1.2)^{x-1} \qquad \text{(B-4)}$$

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. The factor $(1.2)^{x-1}$ is the extrapolation uncertainty factor. If extrapolating beyond the actual or time shifted data less than 1 log cycle, set the exponent equal to zero. 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. This extrapolation uncertainty factor should be increased to $(1.4)^x$ if there is a potential for a "knee" in the stress rupture envelope to occur beyond the actual time shift data.

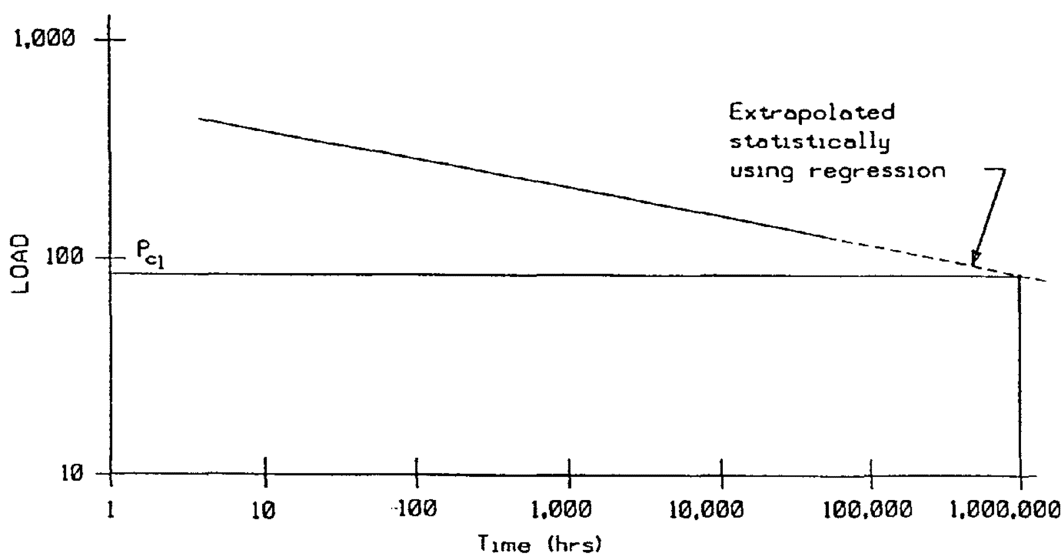


Figure B.2. Extrapolation of stress rupture data and the determination of creep limit load.

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 D 4595) of the sample tested for creep. The sample tested for ultimate tensile strength should be taken from the same lot, and preferably the same roll, of material which 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:

$$RF_{CR} = \frac{T_{ultlot}}{T_1} \tag{B-1}$$

where, T_{ultlot} is the average lot specific ultimate tensile strength (ASTM D 4595) 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 which is two standard deviations below the average value.

B.3. USE OF CREEP DATA FROM "SIMILAR" PRODUCTS

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.), 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, this limited testing program should include creep tests taken to at least 1,000 to 2,000 hours in length. These limited creep test results must show that the performance of the new or similar product is equal to or better than the performance of the product previously tested. If so, the results from the full testing program on the older or similar product could be used for the new/similar product. If not, then a full testing and evaluation program for the new product should be conducted.

B.4 CREEP EXTRAPOLATION EXAMPLES USING STRESS RUPTURE DATA

Two creep extrapolation examples using stress rupture data are provided. The first example uses hypothetical stress rupture data which is possible for PET geosynthetics to illustrate the simplest extrapolation case. The second example uses hypothetical stress rupture data which is possible for polyolefin geosynthetics to illustrate the most complex stress rupture data extrapolation situation, a stress rupture envelope which exhibits a "knee" in the envelope.

B.4.1 Stress Rupture Extrapolation Example 1

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 B.3. The stress rupture data is provided in figure B.4.

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 B.4).

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 B.4.

Step 3: Determine the design, lot specific, creep limit load from the stress rupture envelope provided in figure B.4. 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 B.4,

$$T_1 = (63.4\ kN/m)/(1.2)^{1.68-1} = \underline{56.0\ kN/m}$$

Step 4: The strength reduction factor to prevent long-term creep rupture RF_{CR} is determined as follows (see equation B.1):

$$RF_{CR} = T_{ultlot}\ /T_1$$

where, T_{utlot} is the average lot specific ultimate tensile strength for the lot material used for creep testing. From figure B.3, T_{utlot} is 110 kN/m. Therefore,

$$RF_{CR} = (110\ kN/m)/(56.0\ kN/m) = \underline{2.0}$$

In summary, using rupture based creep extrapolation, T_1 = 56.0 kN/m, and RF_{CR} = 2.0

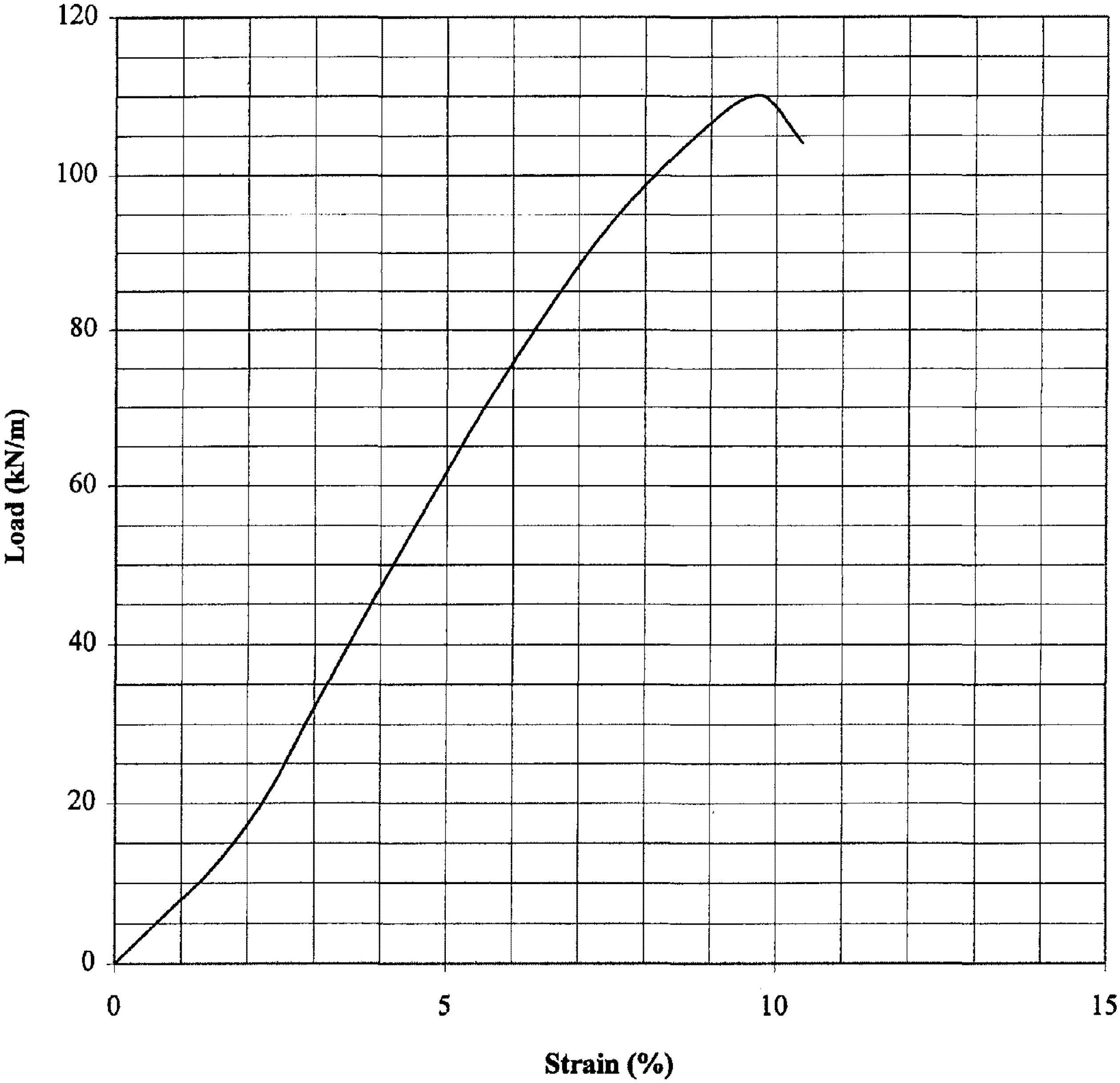


Figure B.3. Wide width load-strain data for PET geosynthetic at 20° C.

B.4.2 Stress Rupture Extrapolation Example 2

The following example utilizes hypothetical stress rupture data for a polyolefin geosynthetic. The data provided in this example is for illustration purposes only.

Given: A polyolefin geosynthetic is 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 geosynthetic has provided stress rupture data at three temperatures 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 creep strain tests. The wide width ultimate strength data for the lot is provided in figure B.5. The stress rupture data is provided in figure B.6.

Find: The long-term creep strength, T_l, 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 in Appendix B for stress rupture data extrapolation will be followed. Step 1 has already been accomplished (figure B.6).

Step 2: Extrapolate the stress rupture data. Using time-temperature superposition, shift the elevated temperature stress rupture envelopes along the shift axis as shown in figure B.6, since there is a "knee" present in the elevated temperature stress rupture envelopes, so that the elevated temperature rupture envelopes line up with the rupture envelope at 20° C. Doing this visually by trial and error results in the following shift factors:

Temperature (°C)	a_T	b_T
30° C	6.0	1.03
40° C	25.0	1.06

Using Equations B-2 and B-3, time and load levels for each of the elevated temperature rupture points are shifted to equivalent 20° C data as shown in table B-1.

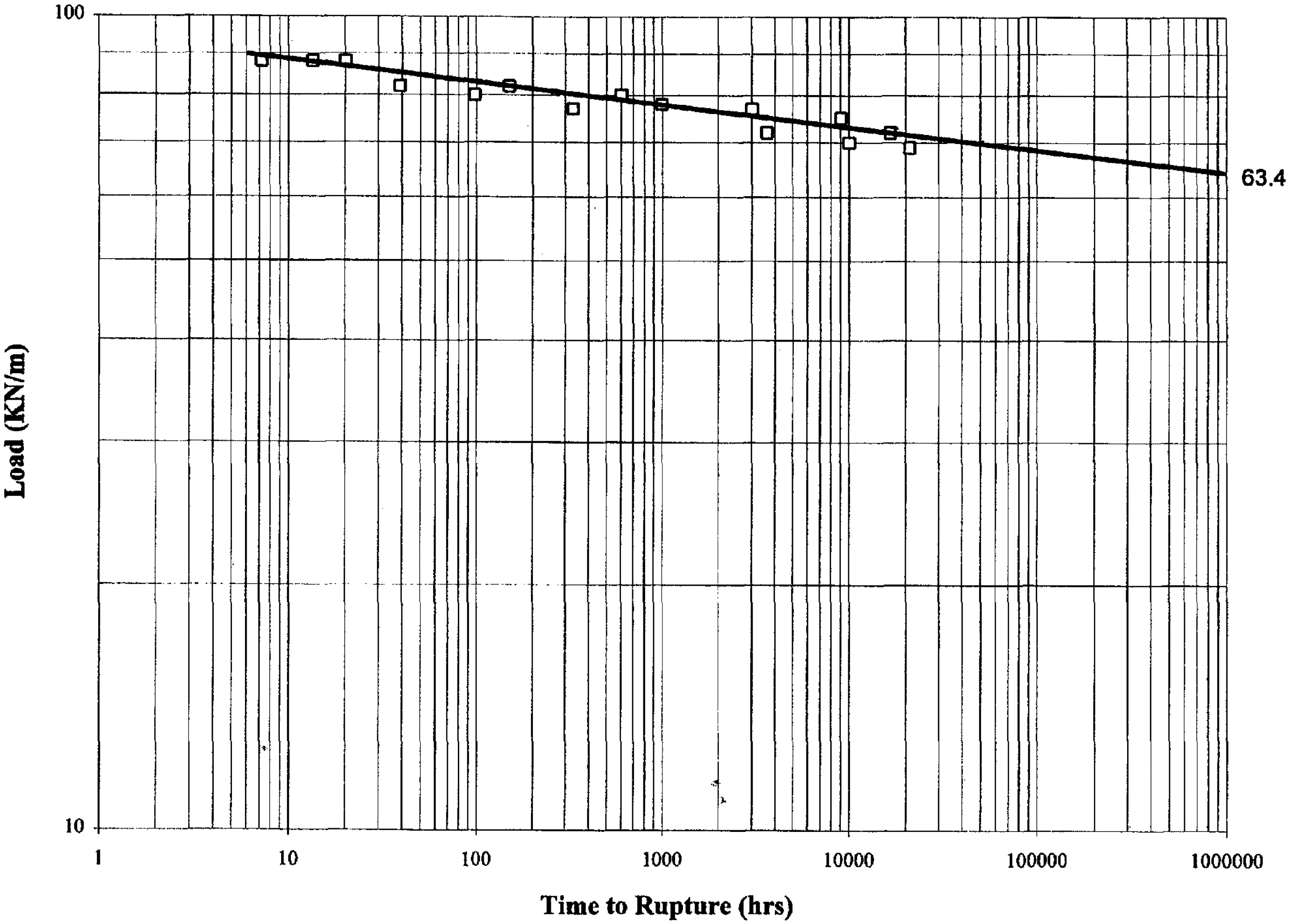


Figure B.4. Stress rupture data for PET geosynthetic at 20° C.

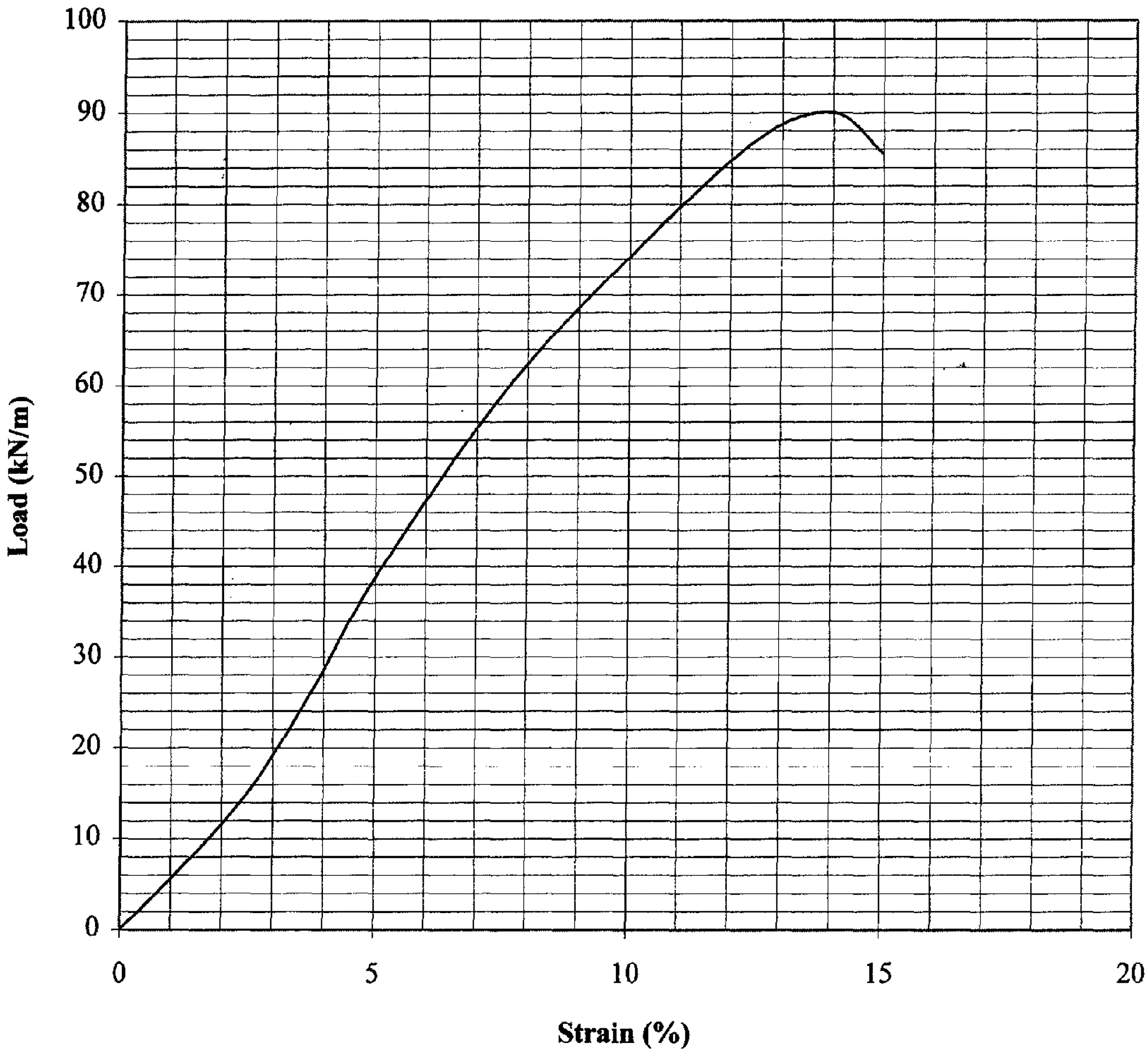


Figure B.5. Wide width load-strain data for polyolefin geosynthetic at 20^{o} C.

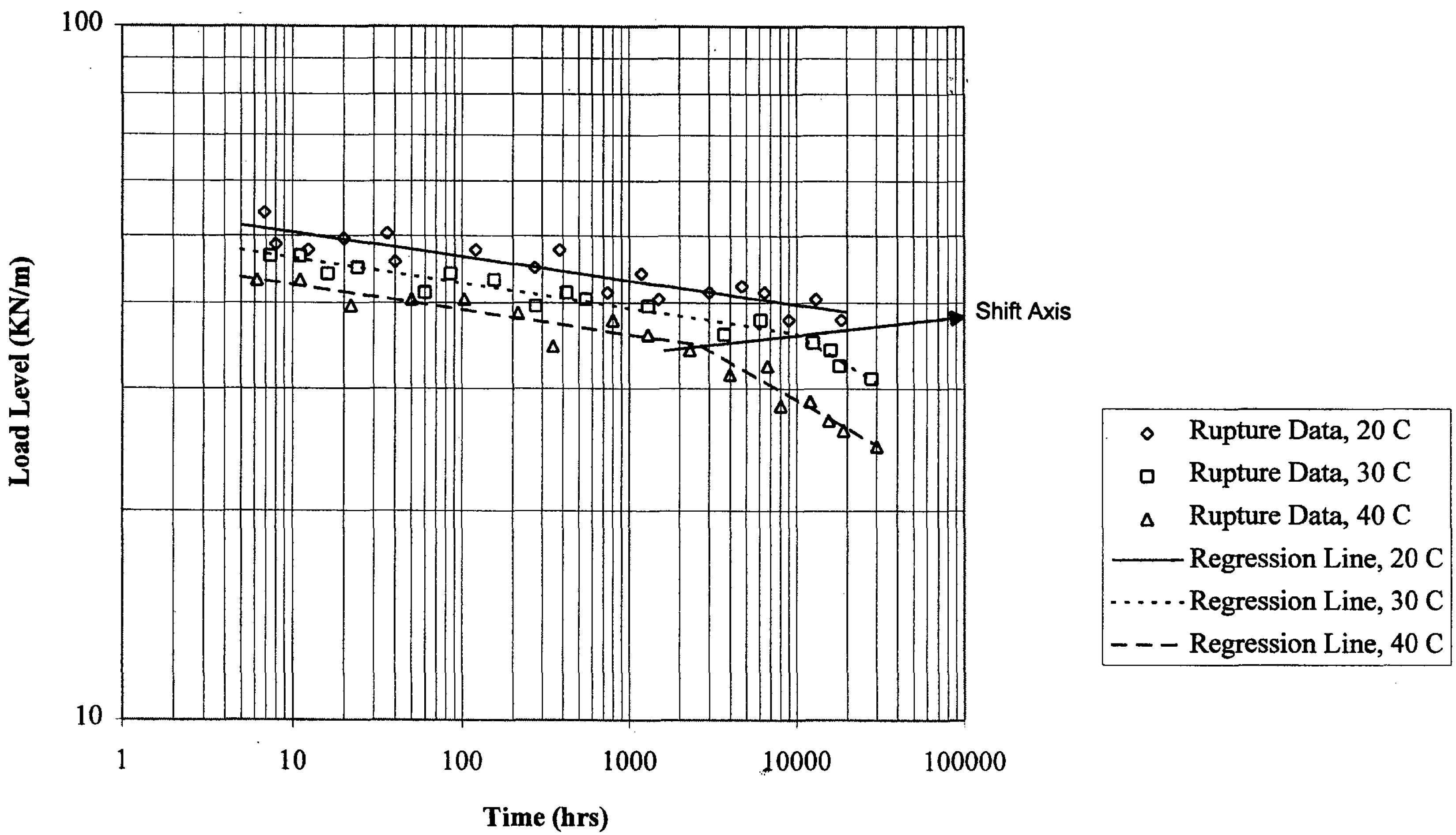


Figure B.6. Stress rupture data for polyolefin geosynthetic.

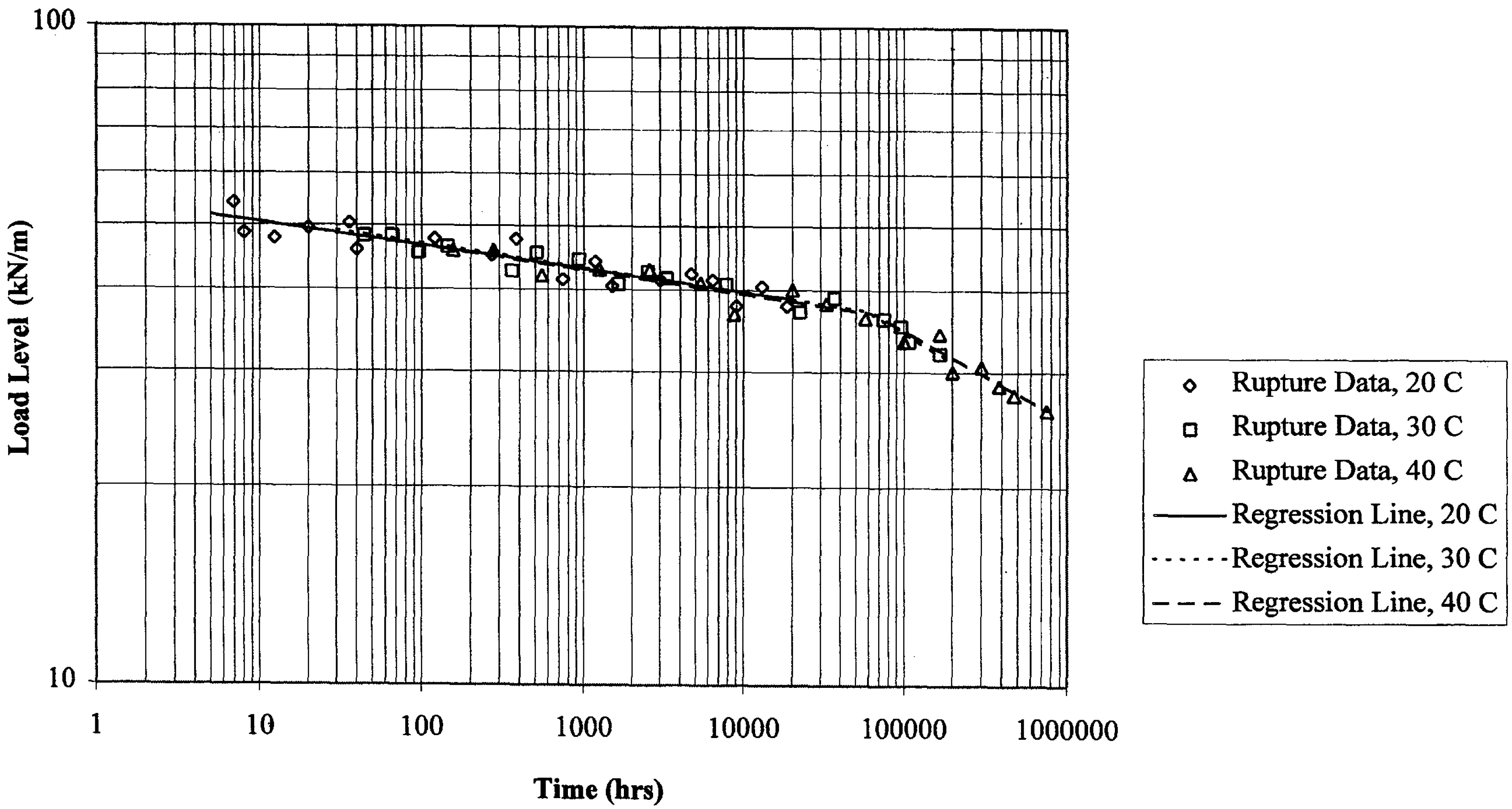


Figure B.7. Stress rupture data for polyolefin geosynthetic after time/load shifting.

Table B-1: Stress Rupture Data Before and After Time/Load Shifting to Equivalent 20° C Data for Polyolefin Geosynthetic

Original Stress Rupture Data						Rupture Data at 30° C After Shifting		Rupture Data at 40° C After Shifting	
Rupture Data at 20° C		Rupture Data at 30° C		Rupture Data at 40° C		Time Shift = 6.0	Load Shift = 1.03	Time Shift = 25	Load Shift = 1.06
Time (hrs)	Load level (kN/m)	Time (hrs)	Load level (kN/m)	Time (hrs)	Load level (kN/m)	Time (hrs)	Load level (kN/m)	Time (hrs)	Load level (kN/m)
6.9	54	7.4	46.8	6.2	43.2	44.4	48.204	155	45.792
8	48.6	11	46.8	11	43.2	66	48.204	275	45.792
12.3	47.7	16	44.1	22	39.6	96	45.423	550	41.976
20	49.5	24	45	50	40.5	144	46.35	1250	42.93
36	50.4	60	41.4	103	40.5	360	42.642	2575	42.93
40	45.9	85	44.1	215	38.7	510	45.423	5375	41.022
120	47.7	155	43.2	350	34.65	930	44.496	8750	36.729
270	45	275	39.6	800	37.8	1650	40.788	20000	40.068
380	47.7	420	41.4	1300	36	2520	42.642	32500	38.16
740	41.4	550	40.5	2300	34.2	3300	41.715	57500	36.252
1180	44.1	1300	39.6	4000	31.5	7800	40.788	100000	33.39
1500	40.5	3700	36	6700	32.4	22200	37.08	167500	34.344
3000	41.4	6100	37.8	8000	28.35	36600	39.964	200000	30.051
4700	42.3	12500	35.1	12000	28.8	75000	36.153	300000	30.528
6400	41.4	16000	34.2	15500	27	96000	35.226	387500	28.62
9000	37.8	18000	32.4	19000	26.1	108000	33.372	475000	27.666
13000	40.5	28000	31.05	30000	24.75	168000	31.9815	750000	26.235
18500	37.8								

The combined 20° C stress rupture envelope resulting from this shifting is shown in figure B.7.

Step 3: Determine the design, lot specific, creep limit load from the stress rupture envelope provided in figure B.7. The load taken directly from the rupture envelope at 1,000,000 hours is 23.9 kN/m. Since no extrapolation beyond the temperature shifted data was necessary, set the exponent to 0. Using Equation B-4,

$$T_1 = (23.9\ kN/m)/(1.2)^0 = \underline{23.9 kN/m}$$

Step 4: The strength reduction factor to prevent long-term creep rupture RF_{CR} is determined as follows:

$$RF_{CR} = T_{ultlot} / T_1$$

where, T_{ultlot} is the average lot specific ultimate tensile strength for the lot of material used for creep testing. From figure B.5, T_{ultlot} is 90 kN/m. Therefore,

$$RF_{CR} = (90\ kN/m)/(23.9\ kN/m) = \underline{3.8}$$

In summary, using rupture based creep extrapolation, T_l = 23.9 kN/m, and RF_{CR} = 3.8

REFERENCES

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

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, p. 673-676.

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.

Helwany, B., Wu, J.T.H., 1992, "A Generalized Creep Model for Geosynthetics," *International Symposium on Earth Reinforcement Practice,* Japan, pp.

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.

Koerner, R.M. (1990), "Determination of the Long-Term Design Strength of Stiff Geogrids," *GRI Standard Practice GG4(a).*

McGown, A., Andrawes, K. Z., and Kabir, M. H., 1982, "Load-Extension Testing of Geosynthetics Confined In-Soil," *Proceedings Second International Conference on Geotextiles*, Las Vegas, NV, Vol. III, pp. 793-798.

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.

Murray, R. T., McGown, A., 1988, "Assessment of the Time Dependent Behavior of Geotextiles for Reinforced Soil Applications," *Durability of Geotextiles RILEM,* Chapman and Hall.

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.

Takaku, A., 1981, "Effect of Drawing on Creep Fracture of Polypropylene Fibers", *Journal Applied Polymer Science,* Vol. 26, pp. 3565-3573.

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.

APPENDIX C

APPROXIMATE COST RANGE OF GEOTEXTILES AND GEOGRIDS

Geosynthetic	Material Cost [1,2] ($\$/m^2$)
Filtration Geotextiles - Class 2 - AASHTO M-288-96	1.25 - 1.75
Erosion Control Mats	3.50 - 6.00
Temporary Erosion Control Blankets	1.25 - 2.50
Roadway Geotextile Separators - Class 2- AASHTO M-288-96	1.25 - 1.75
Asphalt Overlay Geotextiles	0.60 - 1.25
Geotextile Embankment Reinforcement[3]	2.50 - 12.00
Geogrid/Geotextile Wall and Slope Reinforcement[4,5] - per 15 KN/m long term allowable strength	1.50 - 3.50

NOTES:

1. **Typical** costs for materials delivered on-site, for use in engineer's estimate. Costs are exclusive of installation and contractor's markup.

2. Installation cost of geosynthetics **typically** are \$0.30 to \$0.90, except for very soft ground and underwater placement.

3. Assumes design strength is based upon a 5% to 10% strain criteria with an ASTM D 4595 test.

4. Assumes allowable design strength is based upon a complete evaluation of partial safety factors.

5. Material costs of \$2.00 to \$6.00 should be anticipated if using the default procedure for determination of long-term design strength.

APPENDIX D

TYPICAL DIMENSIONS OF STEEL REINFORCEMENTS

Linear Strips

Reinforcement Type	Reinforcement Dimensions	F_y/F_u	Vertical Spacing	Horizontal Spacing
Steel Strips (ribbed)	4 mm thick by 50 mm wide	450/520 MPa	750 mm	Varies, but typically 300 to 750 mm

Welded Wire

Wire Designation	Wire Area (mm^2)	Wire Diameter (mm)	F_y/F_u	Longitudinal Wire Spacing	Transverse Wire Spacing	Mat Spacing
W3.5	22.6	5.4	450/520 MPa	Typically 150 mm	Typically varies 230 mm to 600 mm	For welded wire faced walls, vertically 300 mm, 450 mm, or 600 mm and continuous horizontally. For precast concrete faced walls, vertically 600 mm to 750 mm, horizontally 1.1 m to 1.2 m wide mats spaced at 1.9 m center-to-center or continuous
W4	25.8	5.7				
W4.5*	29.0	6.0				
W5	32.3	6.4				
W7	45.2	7.6				
W9.5	61.3	8.8				
W11	71.0	9.5				
W12	77.4	9.9				
W14	90.3	10.7				
W16	103	11.5				
W20	129	12.8				
*Typical min. size for permanent walls						

Bar Mats

Wire Designation	Wire Area (mm^2)	Wire Diameter (mm)	F_y/F_u	Longitudinal Wire Spacing	Transverse Wire Spacing	Mat Spacing
W11	71.0	9.5	450/520 MPa	Typically 150 mm, with 4 to 7 longitudinal bars per mat	Typically 150 mm to 600 mm	Typically 750 mm vertically and 1.5 m center-to-center horizontally
W15	96.8	11.1				
W20	129	12.8				

Specific wall manufacturers may be able to provide a much wider range of reinforcement configurations depending on the design needs.

APPENDIX E

WORKSHOP PROBLEMS

AND

SOLUTIONS

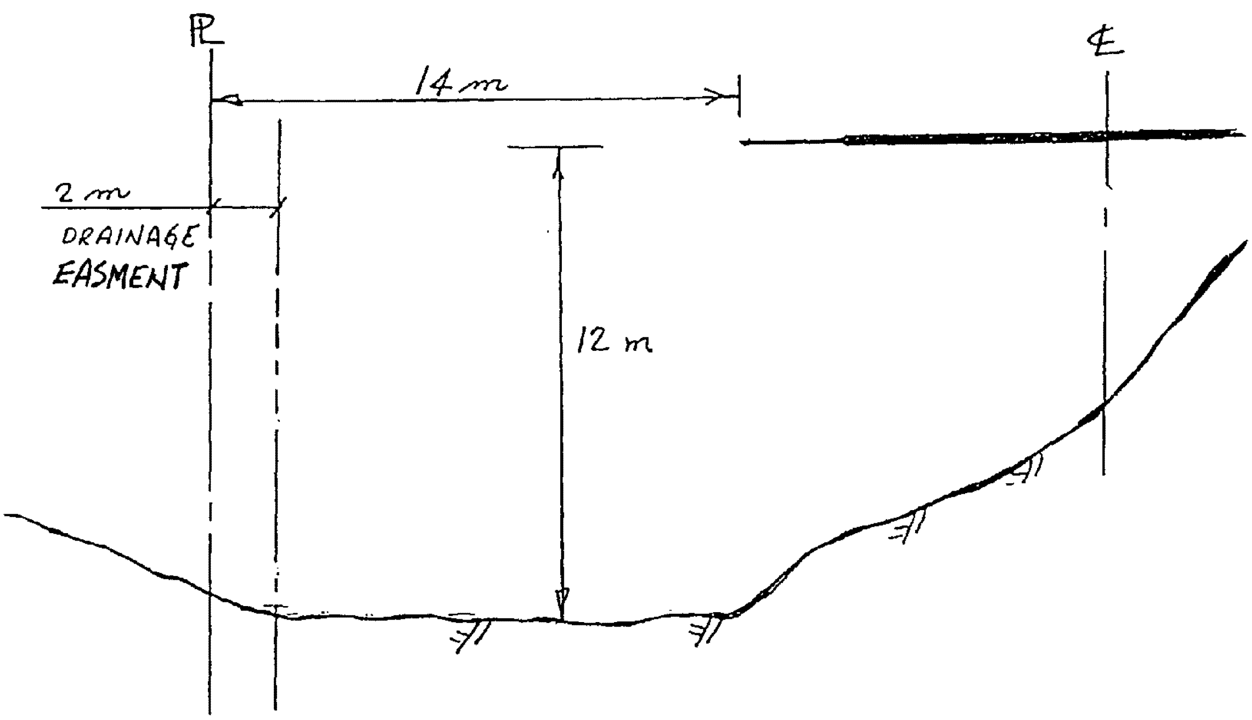
Workshop Problem 1.1

Given the following geometry, develop:

1. the location of 2 possible structure types which would fit the geometry, and

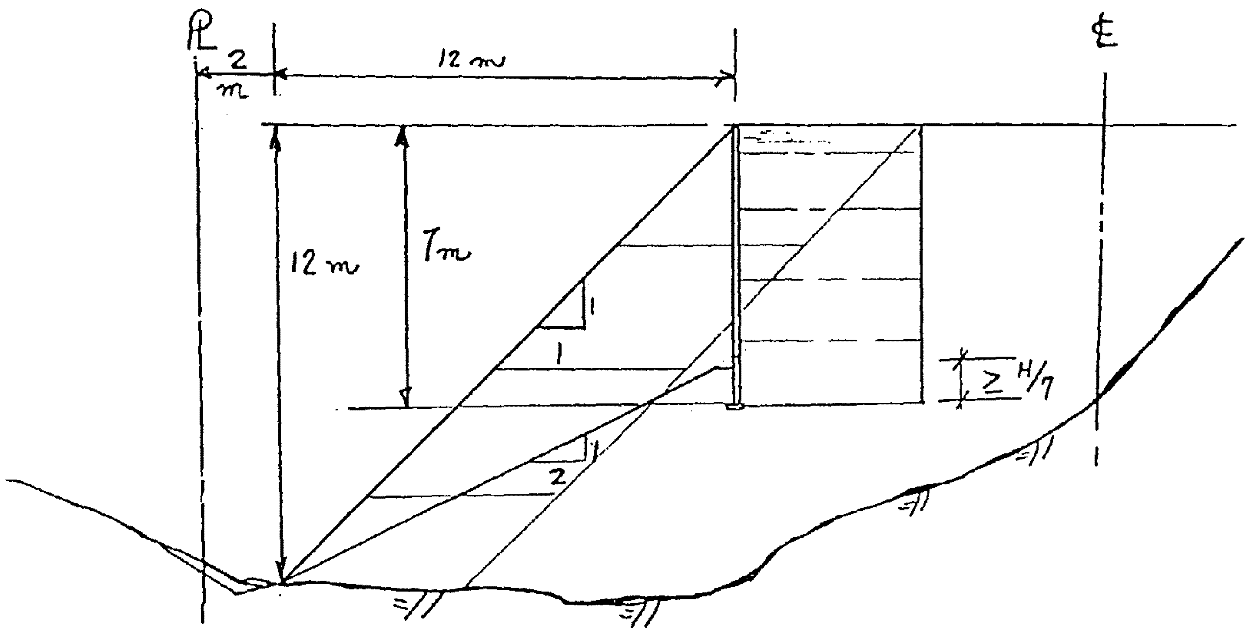
2. a comparative cost estimate based on the following average bid costs :

   MSEW= \$ 270 m^2 including off site select borrow
   RSS = \$ 160 m^2 including on site borrow at 1:1 slope



Solutions to Problem 1.1 :

1. Two types feasible as follows:



2. Comparative costs :

MSEW - 7 m^2/lm. @ $ 270 = $ 1890/lm.

RSS - 12 m^2/lm. @ $ 160 = $ 1920/lm.

Workshop Problem 2.1

Workshop problem 1 suggested that a MSE wall, with precast concrete facing 7 m high, would be cost effective at this location.

Preliminary settlement analyses indicate that differential settlements along the wall would be on the order of 1/100.

The agency construction specifications require that the overall verticality of this wall does not exceed 13 mm per 3 m of height.

1. Develop performance criteria for the design of this MSE wall, including any consideration for a concrete facing. (See Section 2.7)

2. What is the required width of panel joints? (See Table 3)

3. What type of reinforcement (extensible, inextensible) is likely to accommodate the verticality requirement? (See figure 10)

Solution to Problem 2.1

1. External Stability -

Sliding F.S.	$\geq$ 1.5
Eccentricity	$\leq$ L/6
Deep Seated Stability F.S.	$\geq$ 1.3
Bearing Capacity F.S.	$\geq$ 2.0
Embedment	$\geq$ H/7

Internal Stability -

Pullout Resistance F.S.	$\geq$ 1.5
Allowable Tensile Strength	= 0.55 F_y - strip
	= 0.48 F_y - grids
	= T_a - geosynthetic
Design Life	= 75 years

2. Facing Consideration -

Required joint width	- 20 mm

3. Deflection Considerations -

inextensible reinforcements -	$\delta_R = 1$
	@ L/H = 0.7

$$\delta_R = \frac{7000}{250} = 28 \text{ mm} < 30.3 \text{ mm per spec.}$$

ok. for inextensible reinforcement

Workshop Problem 3.1

Pullout Resistance

Given: a) continuous geogrid ($R_c = 1$)

b) continuous metallic grid (with W4 bars, $D_t = 5.7$ mm)

($S_l = 150$ mm, $S_t = 300$ mm, and $R_c = 1$)

See figure 20

c) ribbed steel strip ($b = 50$ mm, $S_h = 0.75$ mm, and $R_c = b/S_h = 0.05/0.75 = 1/15$)

See figure 20

Backfill Properties: $\gamma = 20$ kN/m^3, $\theta = 34^o$

Compute: Estimate the pullout length required for each reinforcement to obtain a pullout resistance of 10 kN per m width of structure at a depth of 1 m below the top of a structure.

Solution:

$$P_r = F^* \alpha \, \sigma_v' L_e C$$

$$L_e = \frac{P_r}{F^* \alpha \, \sigma_v' C}$$

where:

P_r = 10 kN/m of structure

$F^* \alpha =$?

$\sigma'_v = \gamma z$ with $\gamma \approx 20$ kN/m^3 & $z = 1$ m

$\approx (20 \text{ kN/m}^3)(1 \text{ m}) \approx 20 \text{ kN/m}^2$

$$L_3 = \frac{10\, kN/m}{F^* \alpha \; (20 kN/m^2(2)} = \frac{0.25\, m}{F^* \alpha}$$

For Geogrids

$$F^* \approx 0.8 \tan \phi \approx 0.8 \tan 34^\circ = 0.54$$

$$\alpha = 0.8 \text{ to } 1 \text{ for geogrids, use } 0.8$$

$$L_e = \frac{0.25\ m}{0.8\ (0.54)} = \underline{0.58\ m} \quad \Rightarrow \quad \underline{use\ 1\ m}$$

For Metallic Grids

$$\alpha = 1$$

$$F^* \approx \begin{bmatrix} 40\,\alpha_\beta & @ & 0\,m \\ ? & @ & 1\,m \\ 20\,\alpha_\beta & @ & 6\,m \end{bmatrix} = 36.7\,\alpha_\beta$$

$$\alpha_\beta = \frac{t}{2\ S_t} = \frac{D_t}{2\,S_t} = \frac{5.7\,mm}{2\ (300\,mm)} = 0.0095$$

$$F^* \approx 36.7\,(0.0095) \approx 0.35$$

$$L_e = \frac{0.25\ m}{(0.35)\ (1)} = \underline{0.71\ m} \quad \Rightarrow \quad \underline{use\ 1\ m}$$

For Steel Strips

F^* at the top = 1.2 + log C_u

C_u is unknown therefore assume = 4

F^* = 1.2 + log 4 = 1.8

$$F^* = \begin{bmatrix} 1.8 & @ & 0\,m \\ ? & @ & 1\,m \\ 0.67 & @ & 6\,m \end{bmatrix} = 1.61$$

$$\alpha = 1$$

$$L_e \neq \frac{0.25\,m}{F^*\alpha}$$

because P_r in this eq. is per width of wall

P_{rs} is the pullout resistance per strip

$$P_{rs} = P_r / R_c$$

$$R_c = 1/15$$

$$P_{rs} = \frac{10\ kN/m}{1/15} = 150\ kN/m$$

Back to the general equation

$$L_e = \frac{150\,kN/m}{1.61\ (20\,kN/m^2)\ (2)}$$

$$L_e = \underline{2.3\,m}$$

Workshop Problem 3.2

Allowable Strength of Steel Reinforcement

Given: 4 mm thick, 50 mm wide galvanized steel strip
Galvanization thickness = 86 μm

Compute: Calculate the allowable tensile force per unit width at the end of its anticipated 75 year design life.

From equation 11, the allowable tensile force per unit width is:

$$T_a = FS \frac{A_c F_y}{b} = \frac{0.55\, b\, E_c F_y}{b} = 0.55\ E_c F_y$$

In the above equation, what are:

F_y = ____________________

E_c = ____________________

Corrosion Losses: For mildly corrosive backfill (see section 3.5)

Zinc loss : 15 μm/year (first 2 years)
4 μm/year (thereafter)

Steel loss : 12 μm/year (thereafter)

Calculate: T_a = _______________ ?

Solution: Service life for 86 μm zinc coating?

$$\text{Life} = (86\ \mu\text{m} - 2\ \text{yrs}\ (15\ \mu\text{m/yr}))\ /\ 4\ \mu\text{m/yrs}$$
$$= 16\ \text{yrs}$$

Therefore, total required life of carbon steel is:

$$\text{Required life} = 75\ \text{yrs} - 16\ \text{yrs} = 59\ \text{yrs}$$

Then section lost of steel E_s is:

$$Es = 2\ (12\ \mu\text{m/yr})\ (56\ \text{yrs}) = 1.42\ \text{mm}$$
and, $E_c = 4.00 - 1.42 = 2.58$ mm

For 60 grade steel: $\sigma_y = 450$ MPa

Therefore, from the previous equation for T_a:

$$T_a = (0.55)(0.00258\ \text{m})(450{,}000{,}000\ \text{N/m}^2)$$

$$\underline{\underline{T_a = 639{,}000\ \text{N/m} = 640\ \text{kN/m}}}$$

As a side note: each reinforcement could thus support a force F of:

$$F = 0.05\ \text{m}\ (639{,}000\ \text{N/m}) = 32\ \text{kN}$$

Workshop Problem 3.3

Allowable Strength of Geosynthetics

Given: The following geosynthetic reduction factors are to be used for a reinforced slope design:

RF_D = durability reduction factor = 1.2

RF_{ID} = construction damage reduction factor = 1.8

RF_{CR} = creep reduction factor = 3.0

FS = 1 (note: FS = 1.5 on reinforcement is included in stability equation)

Compute: The allowable strength of a geogrid with an ultimate strength, T_{ult}, of 100 kN/m.

Which of these factors should be further evaluated to provide a significant reduction in the required quantity of reinforcement. If reduced, what limitations should be placed on construction.

Solution:

$$T_{al} = \frac{Tult}{RF \cdot FS} = \frac{100\ kN/m}{(1.8)\,(1.2)\,(3.0)\,(1)} = \underline{15.4\,kN/m}$$

RF_{ID} could be significantly reduced (up to 40%) by performing field trials and controlling grain size and lift thickness of the reinforced backfill.

Workshop problem 4.1

A 7 m high MSE wall retains a roadway section consisting of a 0.5 m clear zone, 2.5 m wide shoulder and 2 lanes of traffic 3 m wide.

Consider that the traffic surcharge is equivalent to a uniform live load of 10 kN/m^2, the unit weight of the retained fill is 20 kN/m^3, and the frictional strength of this fill and the foundation soil has been estimated at 30 degrees, with no cohesion.

Compute:

1. A preliminary length of reinforcement. (See Section 4.2)

2. The horizontal pressure on the reinforced fill volume. (See Section 4.2 and figure 22)

3. The F.S. for sliding. (See Section 4.2)

Solution to problem 4.1

1. For preliminary sizing consider L = 0.7 H
   L = 0.7(7) = 4.9 m. Since reinforcements are manufactured in 0.5 m increments, L = 5 m

2a. Traffic surcharges produce earth pressure loads if they are within a horizontal distance from the top of the structure equal to one-half the height of the wall (AASHTO 3.20.3).

   $H/2 = 3.5 \text{ m} > 0.5 + 2.5 = 3.0 \text{ m}$, LL applies

2b. Compute coefficient of earth pressure

   $$K_a = \tan^2 (45 - \phi/2) = \tan^2 (45 - 30/2) = 0.33$$

2c. Compute horizontal earth pressure loads

   a. earth pressure (soil) $F_1 = \gamma H^2 K_a /2 = 20(7)^2\, 0.33/2$
   $= 161.7$ kN/m

   b. earth pressure (traffic) $F_2 = qHK_a = 10(7)\, 0.33$
   $= 23.1$ kN/m

Solution (cont.) problem 4.1

3. The F.S. for sliding is the ratio of the sum of the horizontal resisting forces to sliding forces.

Resisting force

$$V_1 \tan\phi = \gamma HL = 20(7)(5)\tan 30$$
$$= 404.14 \text{ kN/m}$$

Driving force

$$F_T = F_1 + F_2 = 161.7 + 23.1 = 184.8 \text{ kN/m}$$

$$F.S. = \frac{V_1 \tan\phi}{F_1 + F_2} = \frac{404.14}{184.8} = 2.18 \qquad \underline{ok}$$

Workshop problem 4.2

From the previous problem it was determined that the 7 m high wall is externally stable with reinforcements of 5 m in length.

Consider that linear ribbed reinforcements will be used and that the frictional strength of the select fill was determined to be at least 34 degrees and the maximum friction factor $F^* = 1.5$

Compute:

1. For internal stability computations, the effective length of reinforcements at a depth of 3.5 m from the top. See figure 27.

2. The coefficient "K" for internal stability computations at the same depth. Use figure 28.

3. The coefficient F^* for internal stability computations at the same depth. See Section 3.3.

Solution to problem 4.2

1. Effective length L_e at a depth Z_i of 3.5 m

Depth ratio = Z_i / H = 3.5 / 7 = 0.5 H
at H/2, the active zone width = 0.3 H = 0.3 (7)
L_a = 2.1 m
therefore: $L_e = L - L_a = 5.0 - 2.1 = 2.9$ m

2. K coefficient at Z_i = 3.5 m

$K_a = \tan^2 (45 - \phi/2) = \tan^2 (45 - 34/2) = 0.28$

$K = 1.7\, K_a$ = 0.48 at the top of structure and
$K = 1.2\, K_a$ = 0.34 at a depth of 6 m

therefore by interpolation:

$$\frac{(1.7K_a - 1.2K_a)}{6.0} = \frac{x_{zi}}{(6.0 - Z_i)} =$$

$$\frac{(0.48 - 0.34)}{6.0} = \frac{x_{zi}}{(6.0 - 3.5)}$$

$$x_{zi} = 0.058$$

$$K_{zi} = K_a + x_{zi} = 0.34 + 0.058 = 0.40$$

Solution to problem 4.2 (cont.)

3. $F^* = 1.5$ at the top of the structure
   $F^* = \tan\phi = \tan 34 = 0.67$ at 6 m depth

   therefore by interpolation:

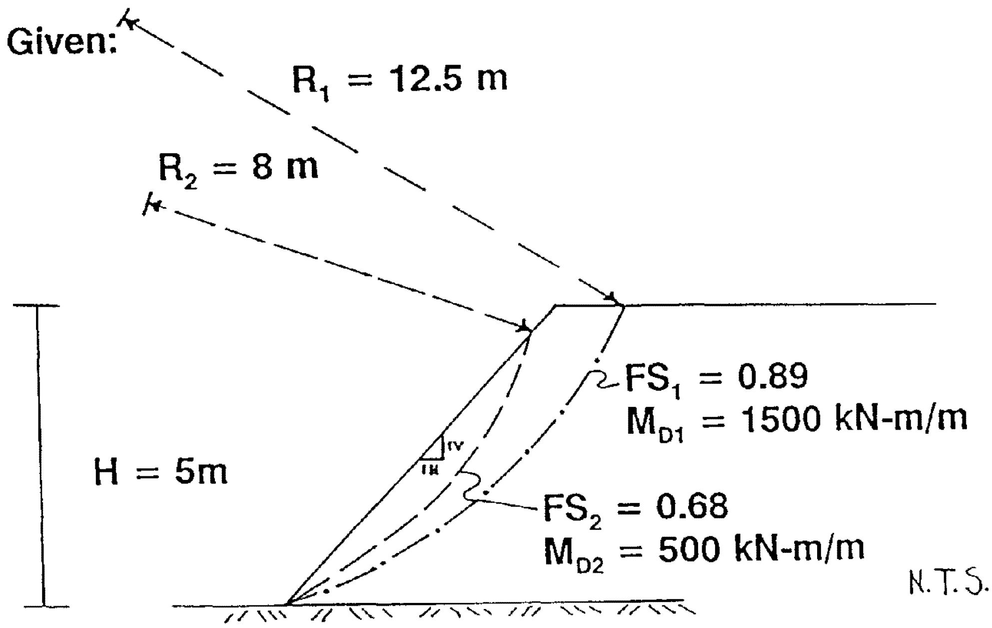
$$\frac{(1.5 - 0.67)}{6.0} = \frac{x_{zi}}{(6.0 - 3.5)} =$$

$$x_{zi} = 0.35$$

$$F^* = F^* \text{ @ } 6.0\ m + x_{zi} = 0.67 + 0.35 = 1.02$$

Workshop problem 7.1

Reinforcement Strength Determination for a Steepened Reinforced Soil Slope



Compute: What is the total reinforcement required for each failure surface to provide a $FS_R = 1.3$

Solution:

$$T_s = (1.3 - FS_U)\,\frac{M_D}{R}$$

$$T_{S-R1} = (1.3 - 0.89)\,\frac{1500\ kN\text{-}m/m}{12.5\ m} = \underline{49\ kN/m}$$

$$T_{S-R2} = (1.3 - 0.68)\,\frac{500\ kN\text{-}m/m}{8\ m} = \underline{39 kN/m}$$

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