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Reinforced Soil Structures Volume I. Design and Construction Guidelines



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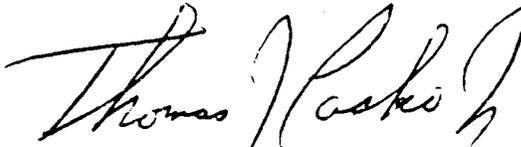


FOREWORD

Soil is a poor structural material because it is weak in tension. Reinforced soil is a generic term that is applied to structures or systems constructed by placing reinforcing elements (e.g., steel strips, plastic grids, or geotextile sheets) in soil to provide improved tensile resistance. Reinforced soil structures are very cost-effective which explains why the concept has emerged as one of the most exciting and innovative civil engineering technologies in recent times. In 1984 an FHWA Administrative Contract research study with STS Consultants, Ltd., was begun to develop practical design and construction guidelines from a technical review of extensive laboratory model and full scale field tests on several reinforced soil structures. This report should interest geotechnical and bridge engineers.

The guidelines are presented in a November 1990 Research Report No. FHWA-RD-89 043, "Reinforced Soil Structures Volume I. Design and Construction Guidelines." Results of the laboratory model and full-scale field tests to verify the design theory in Volume I are presented in Volume II.

Additional copies of the report are available from the National Technical Information Service (NTIS), U.S. Department of Commerce, Springfield, Virginia 22161.



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16. Abstract This report presents comprehensive guidelines for evaluating and using soil reinforcement techniques in the construction of retaining walls, embankment slopes, and natural or cut slopes. A variety of available systems for reinforced soil including in-situ soil nailing are described from information assembled from published literature and manufacturers' catalogs. Detailed guidelines are given for design of reinforced soil structures with inextensible and extensible reinforcements and soil nailing. Design examples are included. These guidelines were developed from technical review of extensive laboratory model tests, small and large scale centrifuge tests, finite element numerical studies and full scale field tests on eight 20-foot high walls and four 25-foot high sloping embankments. The manual contains descriptions of construction procedures, instrumentation and specifications for reinforced soil structures. A companion Volume II contains a technical summary of the research supporting theory to verify the design theory in Volume I and information on several proprietary reinforced soil systems.					
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REINFORCED SOIL STRUCTURES
VOLUME II. SUMMARY OF RESEARCH AND SYSTEMS INFORMATION

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CHAPTER 1

INTRODUCTION

1.1 PURPOSE AND SCOPE

a. Highway Construction and Soil Reinforcement

Retaining walls are an essential element 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 required for embankments. Not many years ago retaining walls were almost exclusively made of reinforced concrete, and were designed as gravity or cantilever walls. Such walls are essentially rigid structures and cannot accommodate significant differential settlements. With increasing height of soil to be retained and poor subsoil conditions, the cost of reinforced concrete retaining walls increases rapidly.

Reinforced soil walls and slopes are cost-effective soil retaining structures which 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 safely constructed. In some cases, the inclusions can also withstand bending or shear stresses providing additional stability to the system.

Modern applications of reinforced soil for construction of retaining walls were developed by H. Vidal in France in the mid 1960's. The Vidal system, called Reinforced Earth, used metal strips for reinforcement as shown schematically in figure 1.⁽¹⁾

Since the introduction of Reinforced Earth in the United States in the early 1970's, several types of reinforced soil systems, as well as several other systems for constructing retaining walls and stable, steep engineered and natural slopes have been developed and are being offered as alternatives to conventional retaining walls. However, there are no uniform standards for the design of reinforcement systems and, in fact, there are different design and construction criteria and procedures for every system. Moreover, each of these systems has a different performance record.

Geotechnical/civil engineers, including those in Highway Departments, often do not have appropriate means to make a technical evaluation of the different systems being offered as alternatives to the conventional retaining walls or to determine whether these systems meet the technical criteria established for a given project. This situation often complicates the selection of suitable earth retention systems.

¹ Reinforced Earth is a registered trademark.

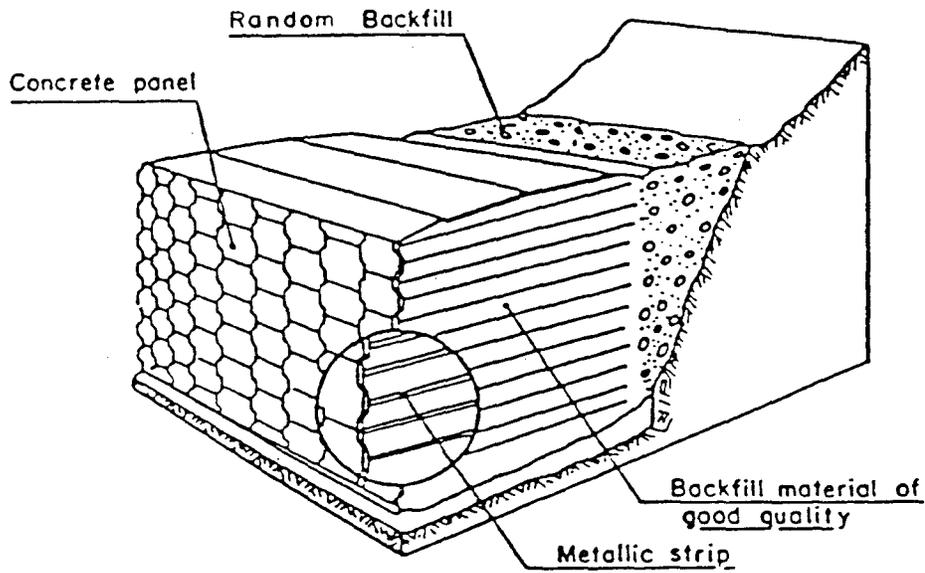


Figure 1. Schematic diagram of a Reinforced Earth wall.

This manual was developed to assist highway engineers and others in determining the feasibility of using reinforced soil systems for walls and embankment slopes on a specific project, evaluating different alternative reinforcement systems, and performing preliminary design of simple systems. The manual also provides a basis for evaluation and preliminary design of new earth reinforcement systems that may be proposed in the future. The design methods provided in the manual are not meant to replace private and proprietary system-specific design methods, but they should provide a basis for evaluating such designs.

b. Terminology

Reinforced soil is any wall or slope supporting system in which reinforcing elements (inclusions) are placed in a soil mass to improve its mechanical properties.

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, steel tendons between anchorage elements. The term reinforcement is used only for those inclusions where soil-inclusion stress transfer occurs continuously along the inclusion. Other inclusions may act simply as tendons between the wall face and an anchorage element.

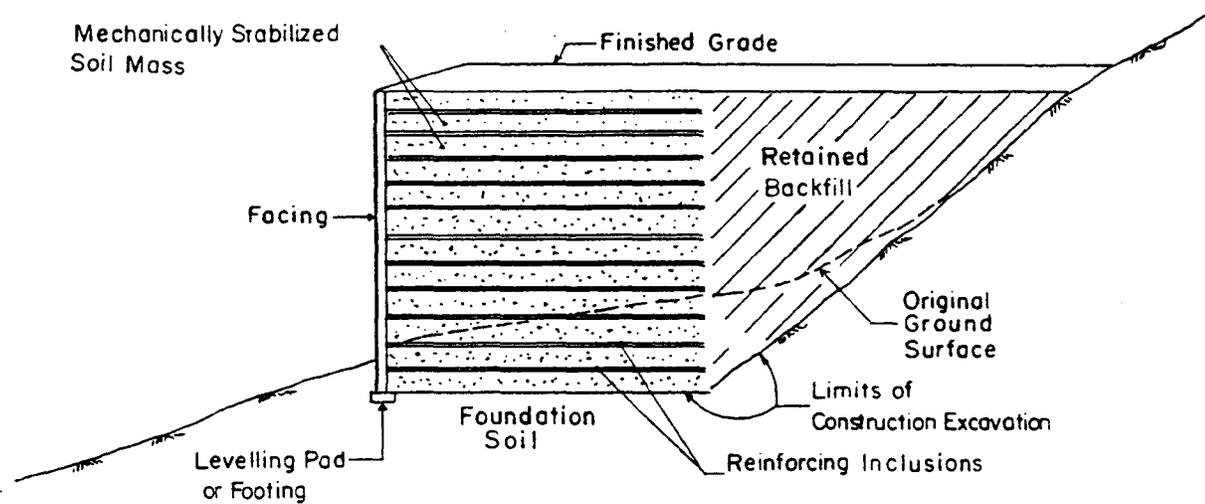
Mechanically stabilized soil mass is a generic term that includes reinforced fill (a term used when multiple layers of inclusions act as reinforcement in soils placed as fill), and multianchored soil mass (a term used when multiple layers of inclusions act as anchored tendons in soils placed as fill). "Reinforced Earth" is a trademark for a specific reinforced soil system.

Soil nailing is a method of reinforcing in-situ soil by the insertion of long metal rods (nails) into an otherwise undisturbed natural soil mass. The technique is used to stabilize existing potentially unstable slopes and to support the side walls of excavations.

Geosynthetics is a generic term that encompasses flexible synthetic materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and polymer 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, metal sheets and plates, gabions, welded wire mesh, shotcrete, wood lagging and panels, and wrapped sheets of geosynthetics.

A generic cross section of a mechanically stabilized soil mass in its geotechnical environment is shown in figure 2.



MECHANICALLY STABILIZED SOIL MASS - PRINCIPAL ELEMENTS

Figure 2. Generic cross section of a reinforced soil structure and its geotechnical environment.

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

c. Basis for the Manual

This manual is based on the results of two recent research projects that were undertaken to examine the design, construction and performance aspects of a number of mechanically stabilized earth systems for use in retaining structures.

The first project, an extensive literature review and evaluation of available systems and design methods, was undertaken under the sponsorship of the National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board (TRB) and resulted in NCHRP Publication 290.⁽²⁾ That state-of-the-art report provides in-depth background on soil reinforcement for engineers seeking an understanding of this important subject.

The purpose of the second project was to develop guidelines for mechanically stabilized soil systems to provide Highway engineers with guidance for selection, design and construction of the different systems of retaining wall alternatives. This project was titled "The Behavior of Reinforced Soil" and was sponsored by the Federal Highway Administration (FHWA). The study was performed by reviewing and evaluating existing design methods in terms of field experience, laboratory testing, analytical studies and a field evaluation program. The results of the research program were then used to develop and substantiate the design procedures provided in this manual.

The background information and design procedure for soil nailing was primarily developed from work performed under a separate FHWA contract (Manual of Practice for Soil Nailing to be completed in 1989).

Finally, information and design procedures for anchored systems were mainly developed through a literature review with a limited amount of laboratory evaluation in the "Behavior of Reinforced Soil" study.

d. Scope and Organization of the Manual

This manual is concerned with different systems for soil mechanical stabilization and their design. A list of systems discussed in this manual is given in section 1.3. The intent of this manual is to provide guidance for design evaluation and to ensure that engineers using mechanically stabilized soil systems follow a safe, rational, and economical procedure from site investigations through construction.

The manual is divided into two volumes, Design and Construction Guidelines and Summary of Research and Systems Information.

This volume is divided into nine chapters. The first chapter includes a brief history of the development of reinforced soil systems and presents a classification of the various types of systems. Brief reference is also made to alternate systems of retaining walls other than the reinforced soil systems. The advantages and disadvantages of reinforced soil systems are discussed, and potential applications are reviewed. Chapter 1 also includes a brief discussion of design philosophy and practical design considerations.

Background information and material requirements necessary for design are reviewed in chapter 2. Soil and site evaluation requirements, including subsurface exploration to evaluate stability, settlement and behavior of the selected system are given. Properties of various reinforcement, retained fill requirements and soil-reinforcement interaction evaluation are also discussed.

The next four chapters are concerned with design methods. Chapter 3 is devoted to reinforced soil walls, chapter 4 to reinforced soil slopes, chapter 5 to nailed soil structures, and chapter 6 to multianchored structures. Each of these four chapters includes design examples.

Chapters 7 through 9 are devoted to practical aspects. Chapter 7 deals with the construction aspects of the different systems. Chapter 8 deals with monitoring programs to assist highway engineers in evaluation of the systems used in their regions. The final chapter, chapter 9, presents suggested general specifications and recommended bidding procedures.

A bibliography of the references cited in the manual is provided at the end of this volume.

Volume II of the manual, Summary of Research and Systems Information, contains the supporting information for the design methods contained in the Design and Construction Guidelines, along with a detailed description of the different types of soil reinforcement systems.

1.2 GENERAL CONSIDERATIONS

a. Historical Development

Inclusions have been utilized since prehistoric times for the improvement of soil. The use of straw to improve the quality of adobe bricks dates back to earliest human time. Many primitive people used sticks and branches for reinforcement of mud dwellings. During the 17th and 18th centuries, French settlers along the Bay of Fundy in Canada used sticks for reinforcement of 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 1880's. Other examples include wood pegs for erosion and landslide control in England, and bamboo or wire

mesh, used universally for revetment erosion control. Soil reinforcing can be achieved by plant roots.

The modern methods of soil reinforcement were pioneered by the French architect and engineer Henri Vidal as a result of his research in the early 1960's which 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 15 years, more than 12,000 Reinforced Earth structures representing over 50 million ft² (4.6 million m²) of wall facing have been completed in 37 different countries. More than 4,500 walls have been built in the United States since 1972.

Since the introduction of Reinforced Earth, several other proprietary and nonproprietary systems have been developed and used. Table 1 provides a summary of many of the current systems by proprietary name, reinforcement type and facing system. Some of these systems are reviewed in the following paragraphs. A detailed description of each system is included in volume II, section 1.

The Hilfiker Retaining Wall, which uses welded wire mesh type reinforcement and facing system, was developed in the mid-1970's, and the first experimental wall was built in 1975. The first commercial use was for a wall built for the Southern California Edison Power Company in 1977 for repair of some roads along a power line in the San Gabriel Mountains of Southern California. In 1980, the use of these walls expanded to larger projects, and to date about 1,600 walls totaling over 1.5 million ft² (140,000 m²) have been completed in the United States.

Hilfiker also developed the Reinforced Soil Embankment (RSE) system, which uses continuous welded wire reinforcement and a precast concrete facing system. The first experimental Reinforced Soil Embankment system was constructed in 1982. Its first use on a commercial project was in 1983 on State Highway 475 near the Hyde Park ski area northeast of Santa Fe, New Mexico. At that site, four reinforced soil structures were constructed with a total of 17,400 ft² (1600 m²) of wall face. Over 50 other RSE projects totaling some 290,000 ft² (27,000 m²) have been constructed in the United States.

A system using strips of steel grid (or "bar mat") type reinforcement, VSL Retained Earth, was first constructed in the United States in 1981 in Hayward, California. Since then, 150 VSL Retained Earth projects containing over 600 walls totaling some 5 million ft² (465,000 m²) of facing have been built in the United States.

The Mechanically Stabilized Embankment (MSE), a bar mat system, was developed by the California Department of Transportation based on their research studies started in 1973 on Reinforced Earth walls. The first wall using this bar mat type reinforcement

Table 1. Summary of reinforcement and face panel details for various reinforced soil systems.

<u>System Name</u>	<u>Reinforcement Detail</u>	<u>Typical Face Panel Detail</u> ¹
Reinforced Earth: (The Reinforced Earth Company 2010 Corporate Ridge McLean, VA 22102)	Galvanized Ribbed Steel Strips: 0.16 in (4 mm) thick, 2 in (50 mm) wide. Epoxy-coated strips also available.	Facing panels are cruciform shaped precast concrete 4.9 ft x 4.9 ft x 5.5 in (1.5 m x 1.5 m x 14 cm). Half size panels used at top and bottom.
VSL Retained Earth (VSL Corporation, 101 Albright Way, Los Gatos, CA 95030)	Rectangular grid of W11 or W20 plain steel bars, 24 in x 6 in (61 cm x 15 cm) grid. Each mesh may have 4, 5 or 6 longitudinal bars. Epoxy-coated meshes also available.	Precast concrete panel. Hexagon shaped, (59-1/2 in high, 68-3/8 in wide between apex points, 6.5 in thick (1.5 m x 1.75 m x 16.5 cm).
Mechanically Stabilized Embankment. (Dept. of Transportation, Div. of Engineering Services, 5900 Folsom Blvd., PO Box 19128 Sacramento, CA 95819).	Rectangular grid, nine 3/8 in (9.5 mm) diameter plain steel bars on 24 in x 6 in (61 cm x 15 cm) grid. Two bar mats per panel (connected to the panel at four points).	Precast concrete; rectangular 12.5 ft (3.81 m) long, 2 ft (61 cm) high and 8 in (20 cm) thick.
Georgia Stabilized Embankment (Dept. of Transportation, State of Georgia, No. 2 Capitol Square Atlanta, GA 30334-1002)	Rectangular grid of five 3/8 in diameter (9.5 mm) plain steel bars on 24 in x 6 in (61 cm x 15 cm) grid 4 bar mats per panel	Precast concrete panel; rectangular 6 ft (1.83 m) wide, 4 ft (1.22 m) high with offsets for interlocking.
Hilfiker Retaining Wall: (Hilfiker Retaining Walls, PO Drawer L Eureka, CA 95501)	Welded wire mesh, 2 in x 6 in grid (5 cm x 15 cm) of W4.5 x W3.5 (.24 in x .21 in diameter), W7 x W3.5 (.3 in x .21 in), W9.5 x W4 (.34 in x .23 in), and W12 x W5 (.39 in x .25 in) in 8 ft wide mats.	Welded wire mesh, wraps around with additional backing mat and 1.4 in (6.35 mm) wire screen at the soil face (with geotextile or shotcrete, if desired).
Reinforced Soil Embankment (The Hilfiker Company 3900 Broadway Eureka, CA 95501)	6 in x 24 in (15 cm x 61 cm) welded wire mesh: W9.5 to W20 - .34 in to .505 in (8.8 mm to 12.8 mm) diameter.	Precast concrete unit 12 ft 6 in (3.8 m) long, 2 ft (61 cm) high. Cast in place concrete facing also used.
Websol: (Soil Structures International, Ltd. 58 Highgate High St. London N65HX England)	5.3 in (135 mm) wide Paraweb: made from high tenacity polyester fibers by Imperial Chemical Industries.	T-shaped precast concrete panel 34.4 sq. ft. (3.2 m ²) area, 6.3 in (160 mm) thick.
York Method: (Transport and Road Research Laboratory, Crowthorne, Berkshire, England)	Galvanized mild steel or stainless steel or glass fiber reinforced plastic or Paraweb or Terram.	Hexagonal; glass fiber reinforced cement; 24 in (600 mm) across the flat; 9 in (225 mm) deep.
Anda Augmented Soils (Anda Augmented Soils Ltd. Oaklands House, Solarton Road, Farnborough Hants GU14 7QL England)	Fibretrain straps (pultruded fiberglass reinforced plastic strip, developed by Pilkington Brothers, 1.6, 3.1 or 6.3 in wide, .08, 0.10 or .16 in thick (40, 80, or 160 mm wide 2, 2.5 or 4 mm thick).	Precast concrete crib units with 12 in (30 cm high) headers 4 ft (1.2 m) apart.
Tensar Geogrid System (The Tensar Corporation 1210 Citizens Parkway, Morrow, GA 30260)	Non-metallic polymeric grid mat made from high density polyethylene of polypropylene	Non-metallic polymeric grid mat (wrap around of the soil reinforcement grid with shotcrete finish, if desired), precast concrete units.
Miragrid System (Mirafi, Inc. PO Box 240967 Charlotte, NC 28224)	Non-metallic polymeric grid made of polyester multifilament yarns coated with latex acrylic.	Precast concrete units or grid wrap around soil.
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.

¹ Many other facing types as compared to those listed, are possible with any specific system.

system was built near Dunsmuir, California, where two walls were built for the realignment and widening of Interstate Highway No. 5. Since then, the California Department of Transportation has built numerous reinforced soil walls using several different types of reinforcement.

Another bar mat reinforcing system, the Georgia Stabilized Embankment System (GASE) was developed recently by the Georgia Department of Transportation, and the first wall using their technology was built for the abutment at I-85 and I-285 Interchange in southwest Atlanta. Many additional walls have been constructed using this system.

Polymeric geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement started in 1981. Extensive marketing of geogrid products in the United States started in about 1983, and since then over 300 walls and slope projects have been constructed.

The use of geotextiles in reinforced soil walls 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, with over 80 projects completed in North America.

The use of nonmetallic strips in reinforced soil was started from experiments carried out in the mid-1970's in France and in the United Kingdom Transport and Road Research Laboratory. During the late 1970's and early 1980's, two reinforced soil walls varying in height from 13 ft to 26 ft (4 to 8 m) were constructed in Europe using nonmetallic reinforcing strips. The only nonmetallic reinforcing strip currently available commercially is the Paraweb strip used in the WEBSOL frictional reinforced soil system. The use of this type of reinforcement for reinforced soil in the United States has so far been limited to experimental walls only.

Soil nailing is an in-situ reinforcement technique which consists of inserting long rods or "nails" into otherwise undisturbed natural soil to stabilize the soil mass. The method has emerged essentially as an extension of rock bolting techniques. Nailing differs from tie back support systems in that the soil nails are passive elements that are not pretensioned as are the tendons in the case of tiebacks. The method can be used to support the sides of excavations or to improve the stability of relatively unstable natural slopes, and when combined with reinforced shotcrete or precast panel facings, the system can provide permanent support of vertical cuts. In North America, the system was first used in Vancouver, B.C. in the late 1960's for temporary excavation support for industrial and residential buildings.

b. Advantages and Disadvantages

Reinforced soil structures and multianchored soil structures have many advantages compared to conventional reinforced concrete and gravity retaining walls. Reinforced walls:

- Use simple and rapid construction which does not require large equipment.
- Do not require experienced craftsmen with special skills for construction.
- Require little site preparation.
- Need little space in front of the structure for construction operations.
- Reduce right-of-way acquisition by constructing or excavating steeper slopes.
- Do not need rigid, unyielding foundation support, because reinforced or multianchored structures are tolerant to deformations.
- Offer a cost advantage when using the soil nailing method for excavation stabilization over conventional systems such as ground anchors and bracing systems, because the structural elements (nails and shotcrete facing) are relatively inexpensive.

The relatively small quantities of manufactured materials required, rapid construction, and in addition, competition among the developers of different proprietary systems has resulted in a cost reduction relative to traditional types of retaining walls. Reinforced or multianchored systems are likely to be more economical than other wall systems for walls higher than about 15 ft (4.6 m) or where special foundations would be required for a conventional wall.

One of the greatest advantages of mechanically stabilized soil structures is their flexibility and capability to absorb deformations due to poor subsoil conditions in the foundations. Also, based on observations in seismically active zones, reinforced soil structures have demonstrated a higher resistance to seismic loading than rigid concrete structures.

Precast concrete facing elements for stabilized soil structures can be made with various shapes and textures (with little extra cost) for aesthetic considerations. Masonry units, timber and gabions can also be utilized with advantage.

The general disadvantages may be associated with reinforced soil structures. These structures:

- Require a relatively large space behind the wall face to obtain enough wall width for internal and external stability.
- Require granular fill at the present time for many of the reinforcement soil systems. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical).
- May require permanent underground easements for soil nailing. (This may specifically limit the use of soil nailing in applications where the required easements extend beneath existing structures).
- Usually require a drainage system for ground nailing which may be difficult to construct and maintain.

Corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as fabrics or plastics by ultra violet rays, and degradation of plastic reinforcement in the ground must be addressed in each project by means of suitable design criteria.

c. General Application of Reinforced Soil

Reinforced soil structures may be cost-effective alternatives for all 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.

Reinforced soil walls offer significant technical advantage over conventional reinforced concrete retaining structures at sites with poor foundation conditions. In such cases, the reduced cost of reinforced soil versus conventional construction, plus 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. In situations where a steep reinforced slope can replace a conventional wall, cost savings can be 70 percent or more. (5)

Some additional successful uses of reinforced soil include:

- Temporary reinforced soil structures which have been especially cost effective for temporary detours necessary for major 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 not only economical but it can also result in savings of land, because a vertical face can be used, and reduce construction time).
- Dams and seawalls and to increase the height of existing dams.

Reinforcement of earth embankments allows use of steeper slopes. The reinforcement also gives resistance to surface erosion as well as to seismic shock. Horizontal layers of reinforcements at the face of a slope also permit heavy compaction equipment to operate close to the edge, thus improving compaction and decreasing the tendency for surface sloughing.

Soil nailing permits steep sided cut slopes and excavations. The method can be used for both temporary and permanent support with substantial reductions in construction disturbance and right-of-way acquisition. For example, in urban sites, the technique can sometimes be used to eliminate the need for underpinning nearby structures. Soil nailing can be cost effective for any temporary or permanent application where conventional retaining systems, such as slurry walls, sheetpile walls, soldier pile walls, or tieback walls are applicable. The system can also provide a cost-effective alternative for stabilization of in-place slopes.

Sketches showing the application of soil reinforcement systems for various applications are included as figures 3 to 7.

d. Factors in Selection of Soil Reinforcement System

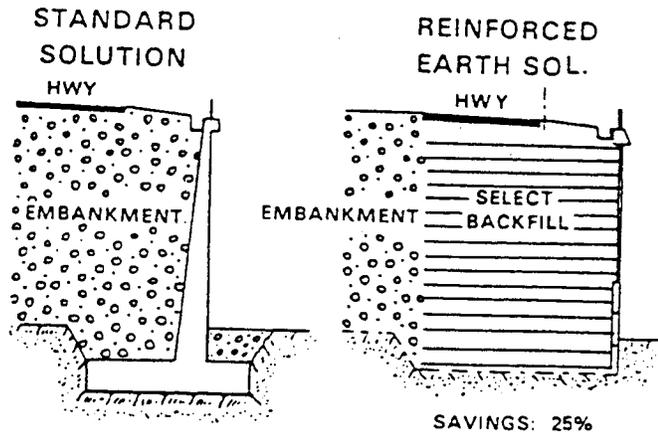
The factors which influence the selection of a soil reinforcement alternative for any project include:

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

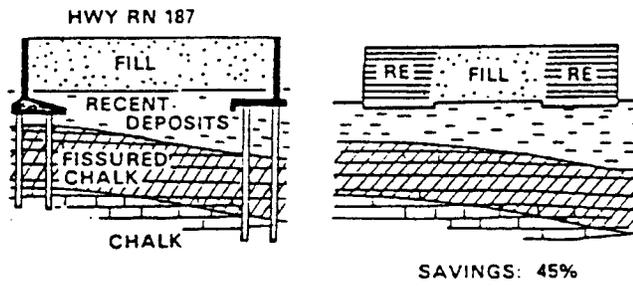
Many reinforced soil wall systems are patented or proprietary. Some companies provide services including design assistance, preparation of plans and specifications for the structure, supply of the manufactured wall components, and construction supervision.

The various systems have different performance histories, and this sometimes creates difficulty in adequate technical evaluation. Methods for handling the matter of specifications and obtaining

RETAINING WALL



BRIDGE APPROACH FILL OVER LOOSE RECENT DEPOSITS



INTERCHANGE WITH ACCESS RAMPS SETE INTERCHANGE.

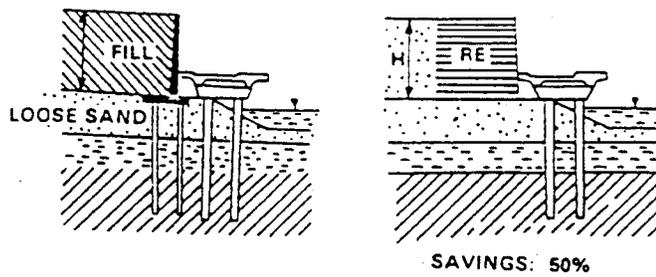


Figure 3. Soil reinforcement systems, urban applications. (4)

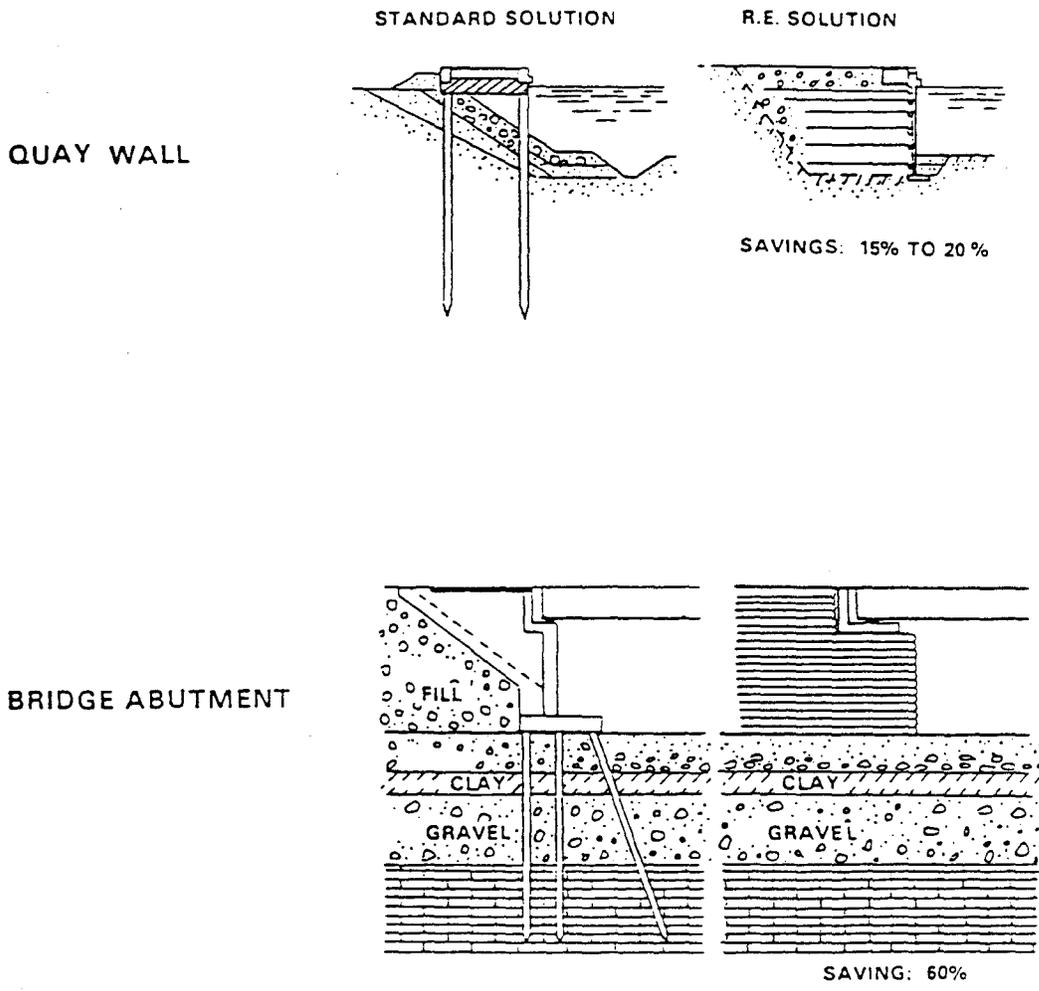
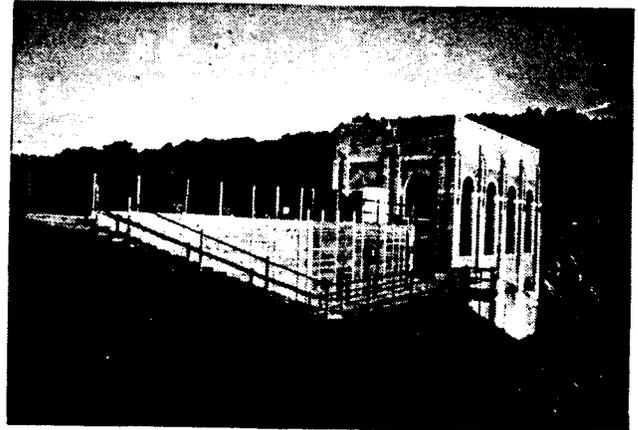


Figure 4. Soil reinforcement systems, abutments. (4)



(a) wrapped face geotextile wall



(b) Cascade Dam wall, Michigan

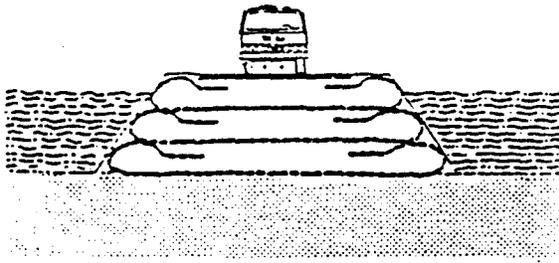


(c) Glenwood Canyon, Colorado

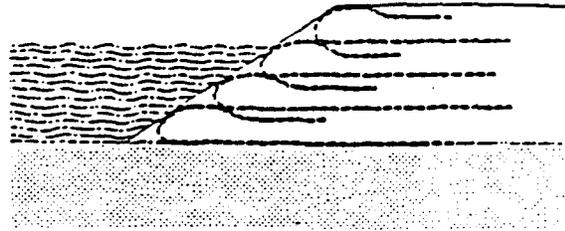


(d) N.Y. D.O.T. wall, New York

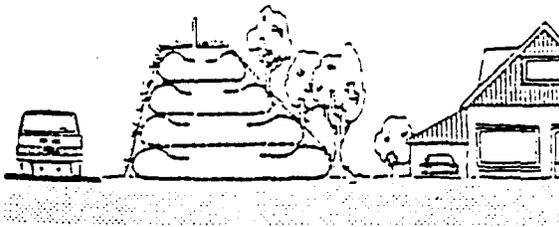
Figure 5. Soil reinforcement systems, geotextile reinforced soil walls.



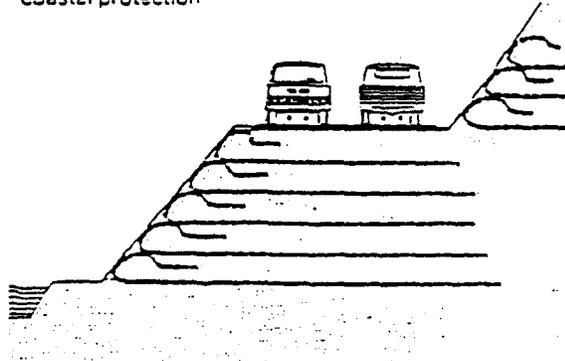
saturated fills



coastal protection

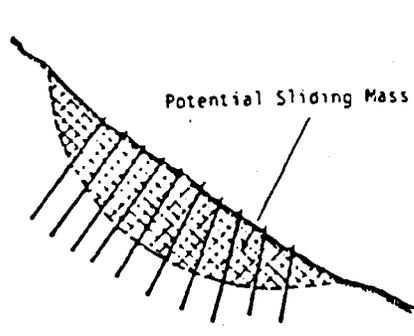


sound absorbing walls

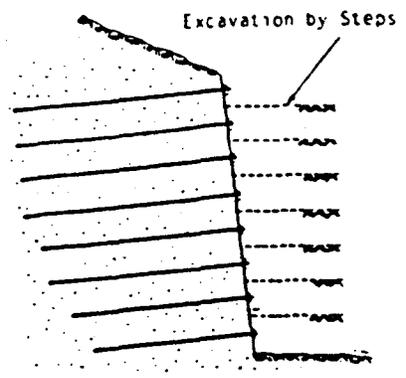


embankment slope stability

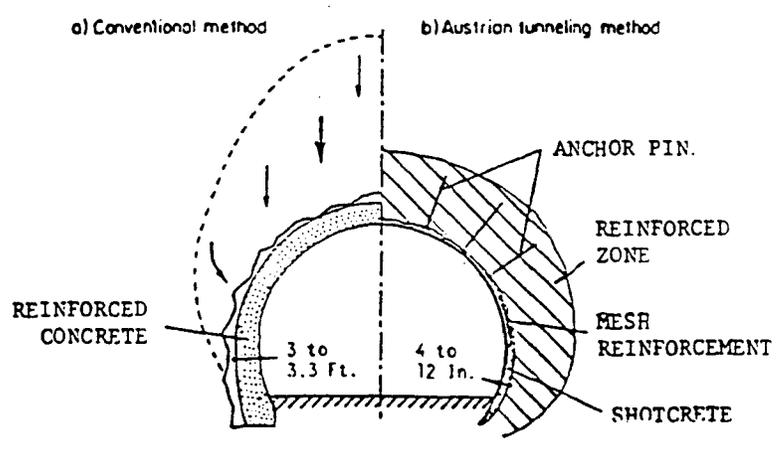
Figure 6. Soil reinforcement systems, embankment slopes.⁽⁵⁾



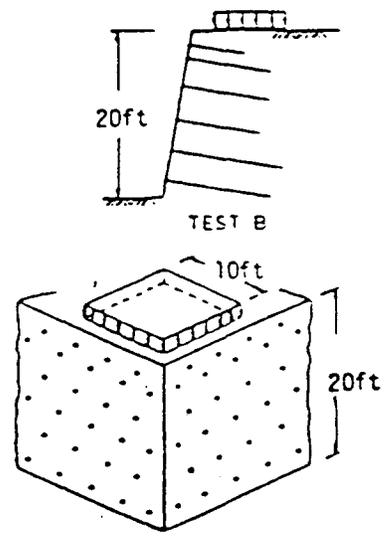
a) Land slide.



b) Retaining structures.



c) Tunnel portal



d) Abutments

Figure 7. Soil reinforcement systems, soil nailing applications.

the most cost competitive and technologically acceptable system are given later in chapters 2 and 9. 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.

e. Cost Comparisons

Costs of a structure are a function of many factors, including cut-fill requirements, wall size, wall type, in-situ soil type, available materials, facing finish, temporary or permanent. It has been found that reinforced soil walls are usually less expensive than reinforced concrete retaining walls for heights greater than about 10 to 15 ft (3 to 4.5 m) and average foundation conditions.

In general, the use of reinforced soil can result in savings on the order of 25 percent 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 structure 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 with some other retaining wall systems is shown in figure 8. The cost of soil nailing systems is typically of the same order as the cost of reinforced fill systems.

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

- . Reinforcing materials - 10 percent to 20 percent of cost.
- . Backfill materials including placement - 30 percent to 40 percent of cost.
- . Facing system - 40 percent to 50 percent of cost.

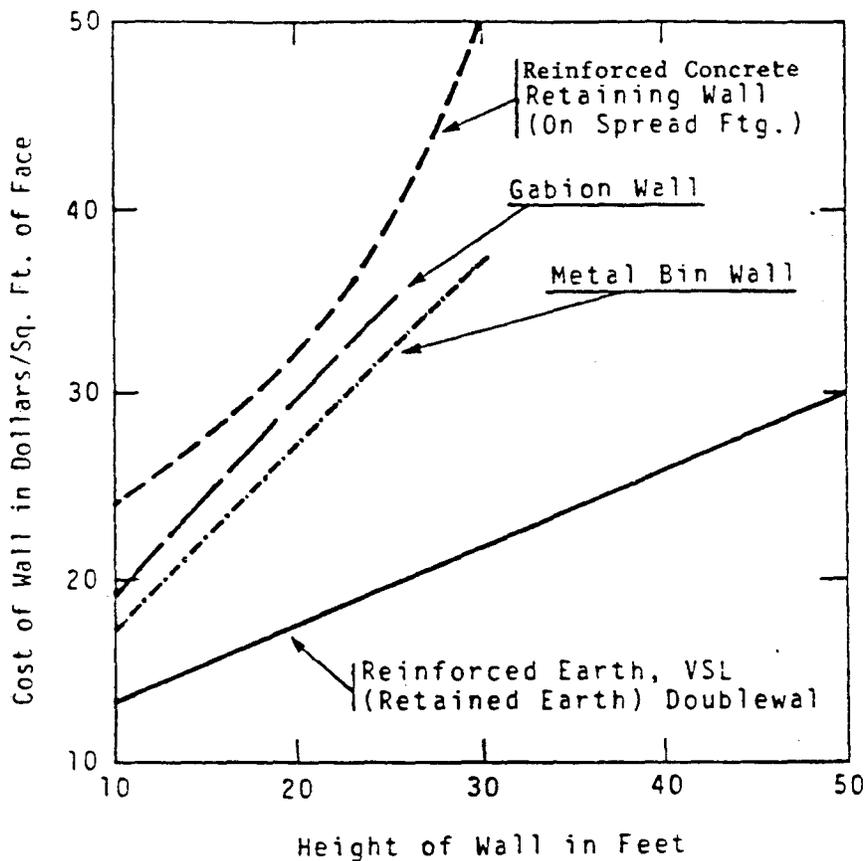
As can be seen from this breakdown, increasing the reinforcement to provide an additional factor of safety may not significantly increase the total cost.

1.3 TYPES OF SYSTEMS

Reinforced soil systems and multianchored soil systems are the two main classes of systems using distributed inclusions.

a. Types of Reinforced Systems

Reinforced soil systems can be described by the reinforcement geometry, the stress transfer mechanism, the reinforcement



NOTES

- (1) Costs shown are for wall materials and erection.
- (2) Backfill and structural excavation costs are not included, except for Gabion and metal bin walls, where the costs include the cost of backfill placed inside the Gabion baskets and metal bins.
- (3) For each individual project, backfill, structure and architectural treatment costs should be added to costs from the chart to make an overall cost comparison of wall types.
- (4) Cost variations between RE, VSL Reinforced Earth, and Doublewall do not appear sufficient to justify separate cost curves for estimating purposes.
- (5) Costs shown are intended for preliminary estimating and cost comparison purpose only.
- (6) Costs shown are based on a combination of recent bid experience in FHWA Region 10.
- (7) Chart was compiled by Ron Chassie, FHWA Region 10 Geotechnical Engineer, Portland, OR.

Figure 8. Cost comparison of six wall types, 1981.⁽⁶⁾

Table 2. Comparison of reinforced soil systems ⁽⁷⁾

REINFORCEMENT TYPE	SOIL GEOMETRY 30° 60° 90°	RECOMMENDED SOIL TYPE Clay Silt Sand Gravel .002 .02 .2 2mm	STRESS TRANSFER MECHANISM		REINFORCEMENT MATERIAL		EXTENSIBILITY		PROPRIETY SYSTEM / PRODUCT NAMES
			Surface Friction	Passive Resistance	Metal	Non-Metal	Extensible	Inextensible	
STRIP	Smooth		•		•				Reinforced Earth
	Ribbed		•		•				Reinforced Earth Paraweb
GRID			•	•	•				VSL, MSE, CAS, RSE, and Welded Wire Wall Maccaferri Gabion
SHEET			•		•				Tensor, Mirafi, Tenax, Reinforced Earth Grid, Conwed
BENT ROD			•	•	•				Geotextiles
FIBER			•		•				Anchored Earth
FLEXIBLE, SMALL DIAMETER NAILS		IN SITU SOILS	•		•				
		IN SITU SOILS		•	•				
RIGID, LARGE DIAMETER PILES		IN SITU SOILS		•	•				

⁽⁷⁾Based on stress transfer between soil reinforcement. Other Criteria may preclude use of soils for specific applications.

material, the extensibility of the reinforcement material, and the method of soil placement as shown in table 2.

Reinforcement Geometry

Three types of reinforcement geometry can be considered:

- Linear unidirectional. Strips, including steel strips (Reinforced Earth), Paraweb plastic strips, and any custom-made fabric strips such as polyester fabric strips originally used by Vidal, rods, cables, and nails.
- Composite unidirectional. Grid strips or bar mats [such as used in VSL Retained Earth, Mechanically Stabilized Embankments (MSE) and Georgia Stabilized Embankment (GASE)].
- Planar bidirectional. Continuous sheets of geosynthetics, welded wire mesh, and woven wire mesh.

Stress Transfer Mechanism

Stresses are transferred between soil and reinforcement by friction (figure 9a) and/or passive resistance (figure 9b), depending on reinforcement geometry:

- Friction. Stresses are transferred from soil to reinforcement by shear along the interface. This is the dominant mechanism with linear and planar reinforcements (strips, rods, cables, nails, fabrics, and geotextiles sheets).
- Passive resistance. Stresses are transferred from soil to reinforcement by bearing between the transverse elements against the soil. This is the dominant mechanism for reinforcement containing a large number of transverse elements of composite inclusions such as bar mats, grids, and wire mesh.

Reinforcement Material

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

- Metallic reinforcements. Consist of mild steel or aluminum.
- Nonmetallic reinforcements. Generally polymeric materials consisting of polypropylene, polyethylene, or polyester polymers.

The performance and durability considerations for these two classes of reinforcement vary considerably and are detailed in chapter 2.

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.

Soil Placement

There are two classes of soil placement:

- Placed soil reinforced systems. Layers of imported or previously excavated soil are placed and compacted, alternating with reinforcement layers.
- In-situ reinforced systems. Inclusions such as nails or piles are placed in an otherwise undisturbed natural soil.

Proprietary Systems

A list and a short description of proprietary systems for the reinforcement of placed soils were previously presented in table 1. Detailed descriptions can be found in volume II, Summary of Research and Systems Information, and in NCHRP Report 290.⁽²⁾

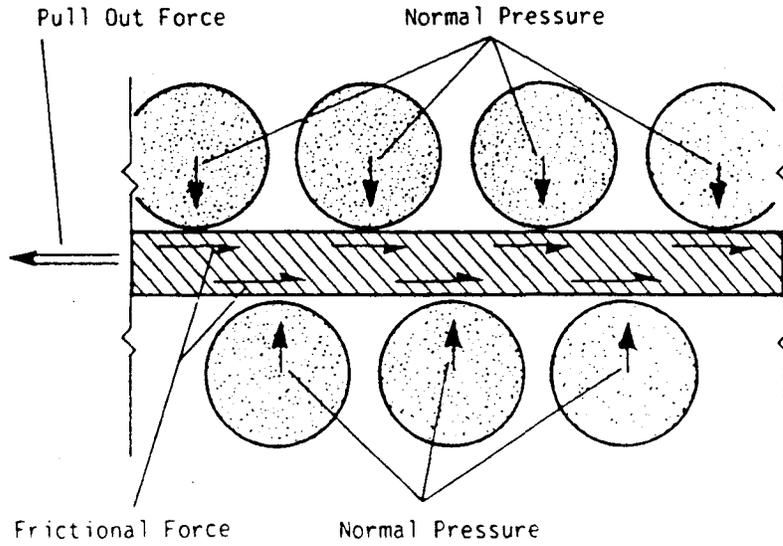
b. Types of Multianchored Systems

Multianchored systems contain a large number of anchors distributed in a regular manner throughout the soil mass. Multianchored systems consist of three types of elements: facing, anchors (or tendons), and anchorage elements. In multianchored systems, the facing retains the fill, whereas in reinforced soil systems the facing has only a localized role in preventing surface erosion and sloughing. In anchored systems resistance to lateral pressure from the fill is provided by soil passive resistance against anchor element movement. Interaction between anchors (tendons) and fill material is usually negligible and consequently the soil mass is usually not reinforced, only retained.

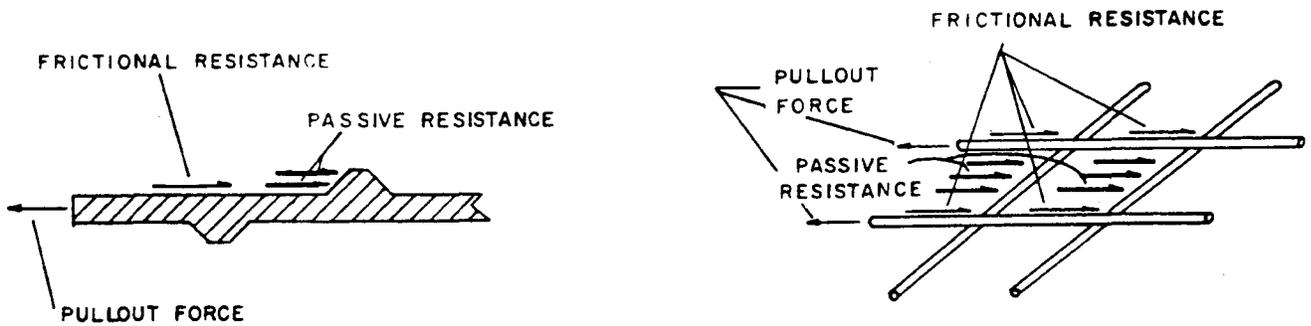
A variety of anchorage elements are used to provide passive resistance at the end of the anchor:

- A concrete plate or beam deadman.
- A special shape (key, corkscrew, zigzag) at the end of the anchor.

A list of proprietary multianchored systems is given in table 3.



A) FRICTIONAL STRESS TRANSFER BETWEEN SOIL AND REINFORCEMENT



B) SOIL PASSIVE (BEARING) RESISTANCE ON REINFORCEMENT SURFACES

Figure 9. Stress transfer mechanisms for soil reinforcement.

c. Systems Considered in this Manual

Systems considered in this manual include:

- Reinforced Placed Soil (Fill) Systems. Reinforced Earth, VSL Retained Earth System, Mechanically Stabilized Embankment (MSE), Georgia Stabilized Embankment (GASE), Hilfiker Retaining Wall ("Welded Wire" Wall), Reinforced Soil Embankment (RSE), Maccaferri Gabion Faced Woven Wire Mesh, Tensar System, geogrid and geotextile reinforced fill systems.

Table 3. Examples of multianchored systems

<u>Proprietary Name</u>	<u>Anchor</u>	<u>Anchorage Element</u>	<u>Facing</u>
American Geo-Tech	Steel Tendons	Concrete Beams	Rectangular Concrete Panels
Tension Retaining Earth System (TRES)	Steel Tendons	Concrete Blocks	Hexagonal Concrete Panels
Ladder Wall ("Mur Echelle")	Steel Tendons	Concrete Blocks	Concrete Panels or Continuous Wall
Actimur	Steel Tendons	Steel Plates	Sheet Piles
Micro-anchorages	Prestressed Steel Tendons	Concrete Frictional Blocks	Continuous thin reinforced concrete wall
Anchored Earth	Steel Bars	Zigzag or Triangular End of Steel Bars	Rectangular Concrete Panels

- In-Situ Reinforced Systems. Soil nailing.
- Multianchored Systems. American Geo-Tech System, Tension Retaining Earth System (TRES), Ladder Wall (Mur Echelle), Actimur, Micro-anchorages, Anchored Earth (all are proprietary names).

All the above systems are described in detail in volume II, Summary of Research and Systems Information.

Systems that are not part of this study but are described in volume II under the title "Alternative Systems" are:

- Cantilever and Counterfort Walls. Traditional reinforced concrete walls.

- Gravity Walls. Traditional gravity walls, crib-walls and Peller walls, Criblock walls, bin walls, Doublewall, Evergreen Walls, gabion walls, and Stresswalls.
- Anchored Walls. Grouted anchor walls, deadman anchored walls, tie-back walls.
- Microreinforced Systems. Texsol, Mini-grids.

1.4 GENERAL PRINCIPLES OF MECHANICAL STABILIZATION OF SOIL

a. Inclusions

Characterization of Inclusions by Size

Inclusions can be put in two broad classes: microinclusions and macroinclusions.

Microinclusions are small elements such as fibers, yarns, and microgrids located very close to each other. As a result, a relatively large proportion of the soil particles are in contact with an inclusion.

Macroinclusions are large elements such as strips, grids, and fabrics whose spacing is large compared to the size of soil particles.

In this manual, only macroinclusions are discussed. Figure 10 provides several examples of macroinclusions.

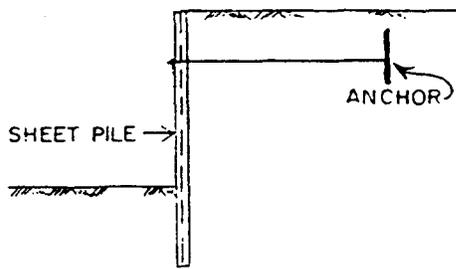
Characterization of Inclusions by Load Transfer Mechanism

Inclusion may act in two ways, as anchors and as reinforcements.

In anchorage, stresses are transferred between soil and inclusion at the ends of the inclusion (figures 10a and 10c). Both ends of each tensile inclusion are attached to an anchorage element such as plate or block. (In the case of a retaining structure, one of the anchorage elements is the face of the wall.) The anchorage element transmits compressive and shear stress to the soil.⁽⁸⁾

As reinforcement, stresses are transferred between soil and inclusion along each inclusion. Several mechanisms are involved in such stress transfer. They are discussed in section b.

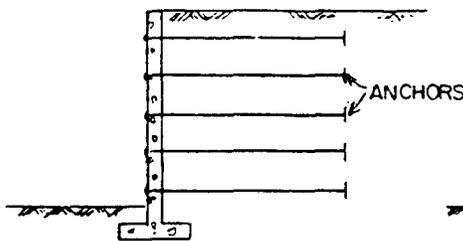
The essential difference between anchorage and reinforcement is the location and distribution of stress transfer: at the ends of the inclusion (anchorage) or along the inclusion (reinforcement). Anchorage improves the behavior of the structure without improving the soil itself, and reinforcement improves the behavior of the soil (and, consequently, the structure). When inclusions are used as reinforcement, it is usually possible to define a "reinforced soil material" as discussed in section b.



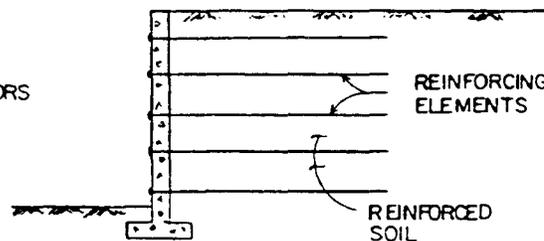
(a) anchored sheet pile wall



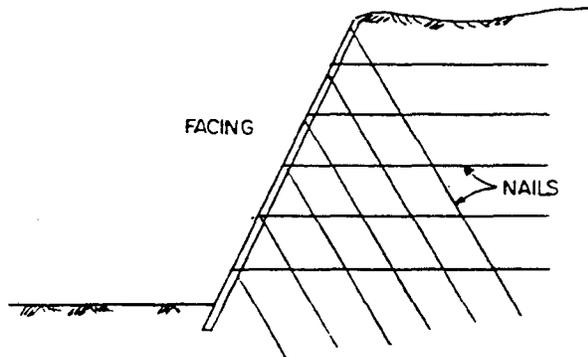
(b) embankment on geotextile on soft soil



(c) multianchored soil structure



(d) reinforced placed soil



(e) reinforced in-situ soils (nailing)

Examples a and b are of localized inclusions:

Examples c, d, and e are of distributed inclusions:

Figure 10. Types of anchored and reinforced soil systems.

Characterization of Inclusions by Distribution

Inclusions can be either localized or distributed. In other words, they can be either in small number and placed in special locations, or in large number and placed uniformly throughout the entire considered soil mass. Examples of structures with localized inclusions are sheet pile walls with a few anchors (figure 10a), and embankments on a geotextile resting on a soft foundation (figure 10b). Examples of structures with distributed inclusions are multianchored soil structures (figure 10c), reinforced soil structures (figure 10d), and nailed soil structures (figure 10e). A soil mass with distributed inclusions tends to act as a coherent unit. The distinction between localized and distributed inclusions has a major impact on design. In this manual, only distributed inclusions are considered.

b. Reinforced Soil Concept

The Reinforced Soil Material

A "reinforced soil material" has the following characteristics:⁽⁹⁾

- Load transfer between soil and inclusion take place continuously along the inclusion, i.e., the load transfer mechanism should be by "reinforcement", not "anchorage" (see section a.).
- Inclusions be distributed throughout the soil mass with a certain degree of regularity, i.e., they should be distributed and not localized.

If a soil mass is reinforced with horizontal parallel layers of steel or geosynthetic strips, the "reinforced soil material" is anisotropic, with higher tensile strength and modulus in the direction of reinforcement than perpendicular to it.

Representative Sample of Reinforced Soil

It is often difficult to obtain and test representative samples of a "reinforced soil material". To be representative, a sample must be at least several times the size of the spacing between inclusions:

- In the case of microinclusions, such as filaments or microgrids, this requirement is easily met by samples whose size is manageable.
- In the case of macroinclusions, such as steel strips or geosynthetic layers, sample size must be at least 5 ft (1.5 m), since spacing between inclusions is at least 6 in (0.15 m) and sometimes much more. From a practical standpoint, it is not easy to conduct tests on samples of that size. This is one of the reasons analysis and design typically is done using discrete elements (soil and reinforcement, separately).

Stress Transfer Mechanisms

Stresses are transferred between soil and reinforcement by two mechanisms: friction and passive resistance (see figure 9).

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 layers, and soil nails.

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 geogrids, bar mat, and wire mesh type reinforcement. Nails placed across a slip surface to stabilize a slope and the transverse ridges on "ribbed" strip type reinforcement also provide some passive resistance.

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

Mode of Action of the Reinforcement

The primary function of the 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. Nails used across a potential slip surface to stabilize a slope also carry shear and bending.

1.5 PRACTICAL CONSIDERATIONS IN DESIGN

a. Design Practice and Construction Considerations

Thorough and careful design and the involvement of the designer during construction is required for several reasons:

- The systems are relatively new and many designers still have little experience with soil reinforcement.
- Although design procedures are now well established in the case of typical structures, the sensitivity of design parameters to nontypical situations is not fully known.
- With the upcoming expiration of the original Reinforced Earth patents, the number of competitive systems is expected to increase. The differences between the various systems in terms of design, construction procedures, and performance must be considered. Responsibility for different areas, i.e., design, materials, installation must be established for new systems.
- Many contractors are not familiar with the available systems or even with the concept of soil reinforcement.
- The amount of expert assistance available during construction may vary significantly. Therefore, the contracting agencies or their engineer should be involved in the preparation of material and construction specifications, and should monitor the construction to ensure that those specifications are enforced.

b. Design Approach

Ideal Design Approach

A stabilized soil structure (i.e., a reinforced or a multianchored soil structure) should be designed using three types of analysis: an analysis at working stresses, a limit equilibrium analysis and a deformation (or displacement) response analysis.

Analysis at Working Stresses

An analysis at working stresses allows the engineer to:

- Select reinforcement location and check that stresses in the stabilized soil mass are compatible with the properties of the soil and inclusions.
- Evaluate local stability at the level of each reinforcement and predict progressive failure.
- Estimate vertical and lateral displacements.

Limit Equilibrium Analysis

A limit equilibrium analysis allows the engineer to check the overall stability of the structure. Three types of stability must be considered, external, internal, and combined:

- The 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.
- The 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.

Deformation Response Analysis

A deformation response analysis allows for an evaluation of the anticipated performance of the structure with respect to anticipated horizontal displacement. In addition, the influence of variations in the type and density of reinforcement on the performance of the structure can be evaluated. Deformation analyses are the most difficult and least certain of the three types of analysis. In many cases, they are done only approximately or it is simply assumed that adjusted factors of safety against external or internal stability failure will ensure that deformations will be within tolerable limits. A conventional settlement analysis is also required.

c. Current Design Approach

NCHRP 290 describes most of the current design procedures.⁽⁸⁾ These procedures vary from one system to another, with the only uniform feature being that analyses for horizontal displacement are rarely done. In some cases, only one of the above-described analyses is performed. In other cases, a combination of analyses is performed. In this manual, the use of a generalized method is advocated for design of a system in a retaining structure project.

d. Recommended Practical Design Approach

The three step approach recommended in this manual combines the ideal approach described above and current practice. The three steps are:

1. Evaluation of external stability.
2. Evaluation of internal stability.
 - local
 - global
3. Displacement evaluation for construction control.

Analyses for internal stability, external stability, and displacements are presented in chapters 3 through 6 for each type of stabilized soil structure considered. The considerations on internal stability in the remainder of this chapter do not apply to multianchored soil systems, which are discussed in chapter 6.

e. Internal Stability

Internal stability, as well as overall stability (i.e., part of the external stability) of mechanically stabilized structures, is calculated using a slip surface analysis. Whereas overall stability is determined by classical slope stability methods, the internal stability analysis requires additional concepts and methods, which have features that are briefly described in the following paragraphs.

Modes of Internal Failure

Internal failure of a mechanically stabilized soil structure 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.

Internal Stability Analysis of Reinforced Soil Walls

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 line has been assumed to be approximately bilinear in the case of inextensible reinforcements (figure 11a), approximately linear in the case of extensible reinforcements (figure 11b), and passes through the toe of the wall in both cases.

A reasonable approximation for the maximum total load to be carried by a reinforcement is given by the maximum horizontal stress in the soil times the tributary wall area. This is illustrated in figure 12.

Slip Surface that Crosses Reinforcements

For reinforced embankment slopes and some reinforced soil walls, the most critical slip surface may cross one or more layers of reinforcements. A special slope stability analysis method which takes into account the effect of inclusions intercepted by the slip surface must be used in these 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.

A reinforcement intercepted by a slip surface has two beneficial effects:

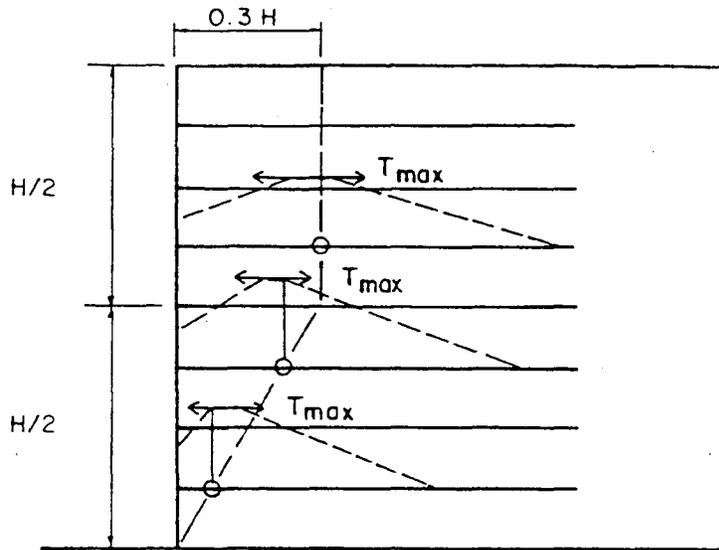
- For circular failure surfaces, the tangential component of the tensile force in the inclusion increases the resisting moment as compared to the unreinforced case.
- The normal stress along the slip surface, in the vicinity of the inclusions, is increased. This effect is called the confinement effect. It leads to an increase of the resisting forces through a local increase in the shear strength of the soil in the vicinity of the reinforcement.

In general, any shape of slip surface can be considered: plane, circle, multilinear, etc. Reinforced slope stability analysis methods have been published and computer programs are available which allow for limit equilibrium analysis of stabilized soil masses. (7, 9, 10, 11, 12)

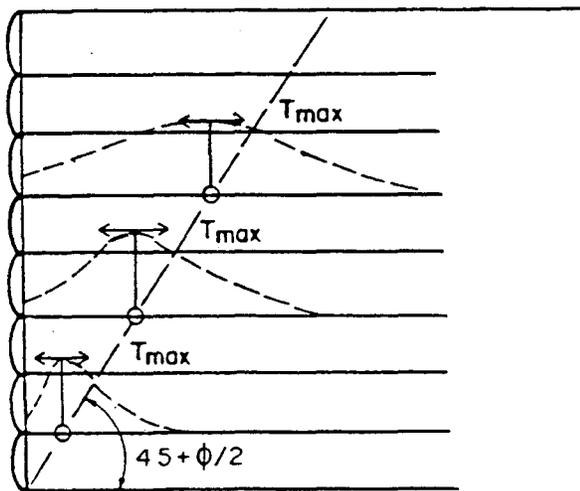
Factor of Safety

There is an associated factor of safety for each potential internal failure mode:

- $(FS)_F$: Safety factor on the shear strength of the fill material or the natural ground in the stabilized mass.
- $(FS)_{BR}$: Safety factor on the breakage resistance of the reinforcements.
- $(FS)_{PO}$: Safety factor on the pullout resistance of the reinforcements.

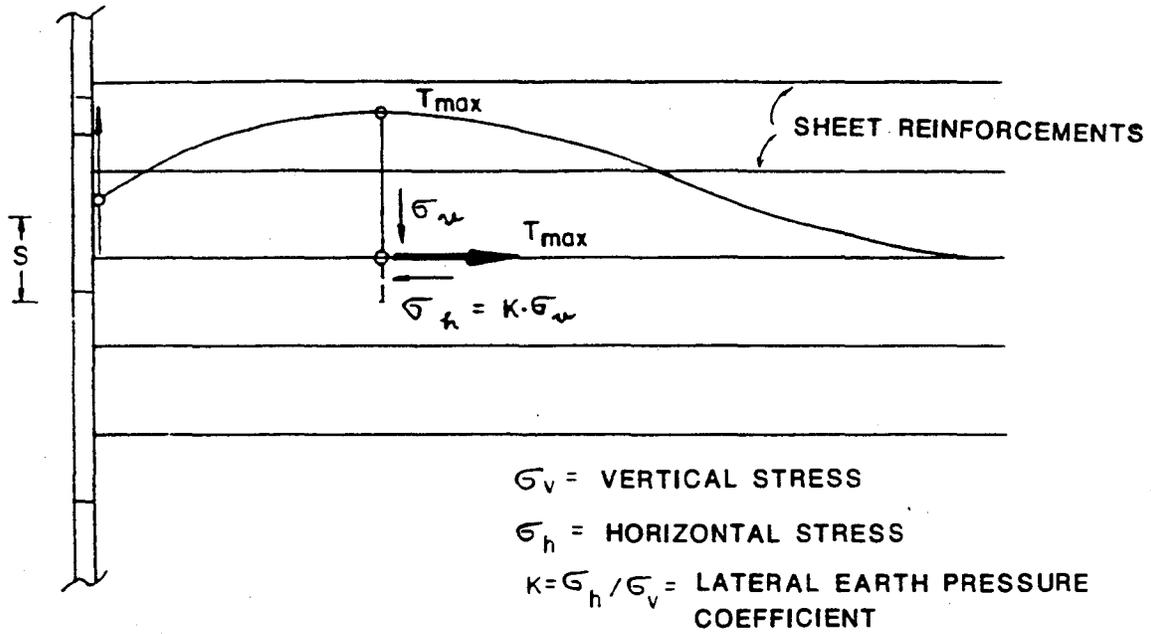


(a) Soil walls with inextensible reinforcements.



(b) Soil walls with extensible reinforcements.

Figure 11. Locus of maximum tensile force lines and assumed failure surface.



NO SHEAR FORCE IN THE FACING AND BETWEEN THE SLICES:

SHEET RELIEF: $T_{max} = K \sigma_v \cdot S =$ MAXIMUM TENSILE FORCE IN THE REINFORCEMENT PER UNIT WIDTH OF WALL.

STRIP RELIEF:

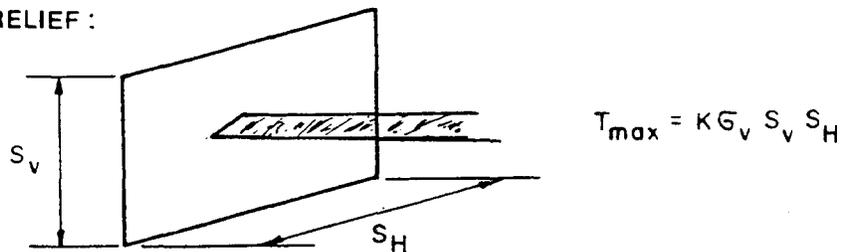


Figure 12. Calculation of the maximum tensile forces in the reinforcement layers.

These safety factors can be determined at any reinforcement layer.

This is not the case using a reinforced slope stability analysis method. For each potential failure surface, a global factor of safety, FS_g , is obtained. Generally, the allowable reinforcement resistances are given, and the design of the stabilized soil structure is conducted in order to obtain a minimum value of the global factor of safety.

Recommended factors of safety for internal stability are presented in the design chapters 3 through 6.

f. External Stability

External failure of the reinforced soil mass is generally assumed to be possible by:

- Sliding of the stabilized soil mass over the foundation soil.
- Bearing capacity failure of the foundation soil.
- Overturning of the stabilized soil mass.
- Slip surfaces failure entirely outside the stabilized soil mass.

Factors of safety for external stability are based on classical analysis of reinforced concrete and gravity wall type systems and are discussed further in chapters 3 through 6.

1.6 DESIGN OF COMPLEX STABILIZED STRUCTURES

The basic design methods presented in this manual consider simple stabilized structures with horizontal reinforcement layers having approximately the same length (figure 2) throughout the full height of the structure. Although most stabilized structures fall into this category, more complex structures are sometimes built or considered at the design stage, including:

- Structures with inclusion layers of different lengths.
- Structures with inclusion layers of different inclinations.
- Structures with inclusion layers of different strengths.
- Structures with multiple facings (or "stacked" wall designs).
- Structures supporting a sloping soil surcharge.
- Composite structures such as a Reinforced Earth wall constructed above a slope stabilized by soil nailing.

Design guidance for these structures is provided in chapter 3 following the basic design method. It is important to realize that the applicability of usual methods may be questionable in some cases. For example:

- Traditional soil mechanics methods used for evaluating external stability of the stabilized structure may not be applicable to complex structures because such structures may be far from being a rigid body, or because their shape may make it difficult to use traditional methods.
- Semiempirical methods developed in the past 2 decades to evaluate the internal stability of stabilized soil structures may not be appropriate for complex structures because these methods have been established using measurements made on simple reinforced soil structures.

Consequently, in addition to the recommended design approach in section 1.5.d., a global limit equilibrium analysis as outlined in that section should be considered for the design of complex structures. The limit equilibrium analysis should consider three types of slip surfaces: (1) slip surfaces entirely inside the structure; (2) slip surfaces entirely outside the structure; and (3) slip surfaces partially inside and partially outside the structure.

The following provides a review of special design considerations for the specific complex structures listed above, and design guidance for each system is provided in chapter 3.

Structures with Reinforcement Layers of Different Lengths

The design methods presented in this manual can be used with (or can lead to) structures where inclusion layers have different lengths provided the length difference between successive layers is not too large. If, for some reason, significant length differences between inclusions are considered, a limit equilibrium analysis must be conducted.

At the conceptual design stage, it may be tempting to consider a cross section of the type shown in figure 13, with inclusion length increasing as depth increases. Such a design must be approached with caution to take into account:

- Potential sliding at all reinforcement levels.
- Increased tensile forces in the lower level reinforcements.
- Increased lateral deflections in the upper zones.
- Inadequate pullout resistance near the top against future surcharge loads.

Structures with Multiple Facings (Stacked Wall Structures)

Some complex structures may consist of several stabilized soil masses resulting in a structure with multiple facings (figure 14a). In this case, two design approaches should be used for the internal stability analyses and the most conservative solution should be selected:

- First Approach. Design structure "1" (figure 14a) considering structure "2" as its foundation. Then design structure "2" considering structure "3" as its foundation, and structure "1" as surcharge, etc.
- Second Approach. Design an equivalent structure (figure 14b), as described in chapter 3.

These approaches are only for internal stability analysis. For both cases, a global limit equilibrium analysis is also necessary. Judgment is often needed for complex geometrics and analysis modeling techniques.

Composite Structures

Composite structures resulting from the combination of one or more reinforced soil masses and one or more alternative systems should be designed using several approaches, similar to those indicated above for stacked wall structures. Each section should be analyzed separately and stacked as described in Approach 1 for stacked wall structures. In addition, an equivalent system should be evaluated using each separate design approach. The most conservative design should be selected. It is impossible to give general guidelines for the selection of the design approaches, due to the wide variety of possible composite structures.

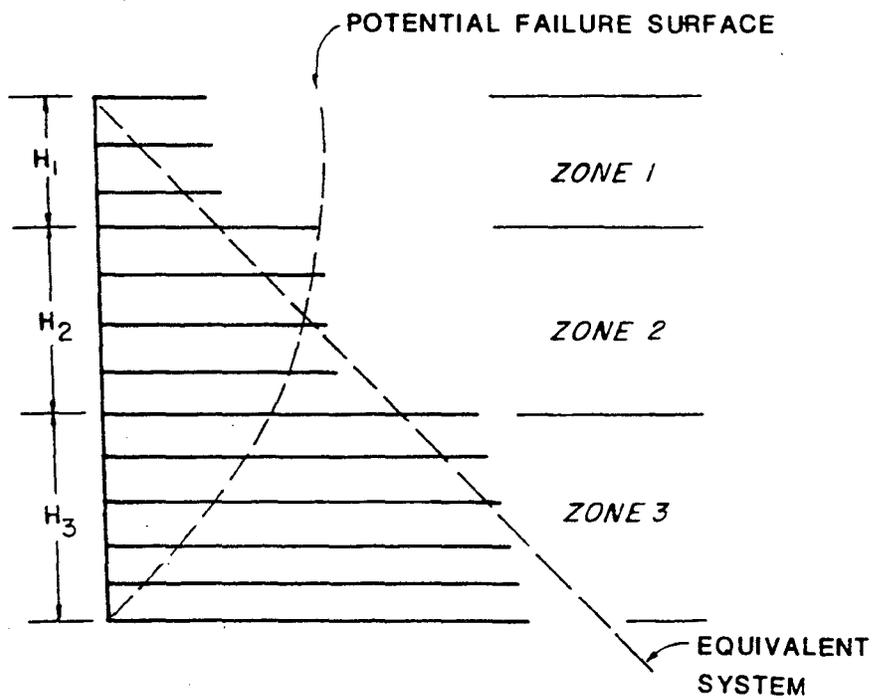
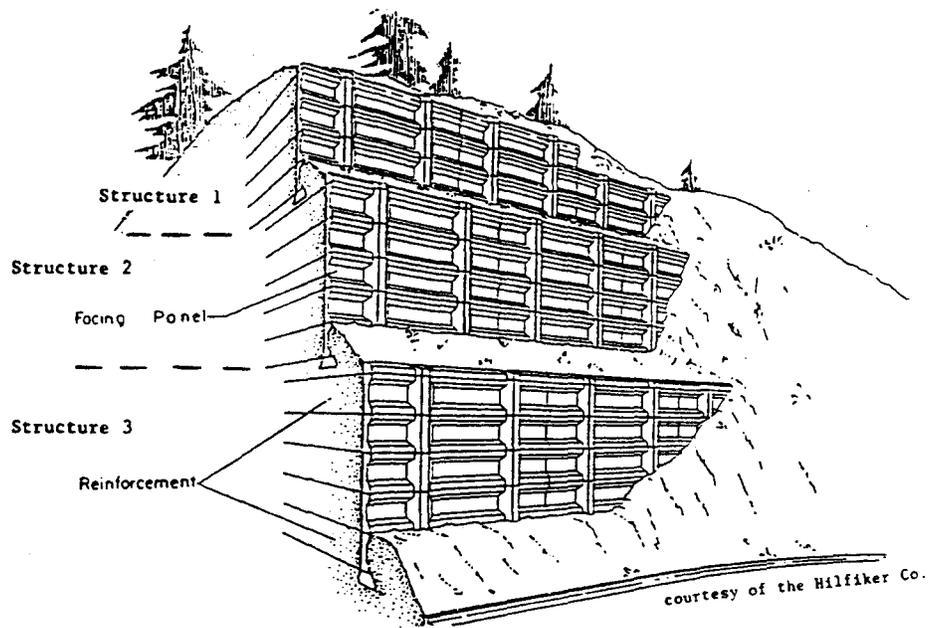
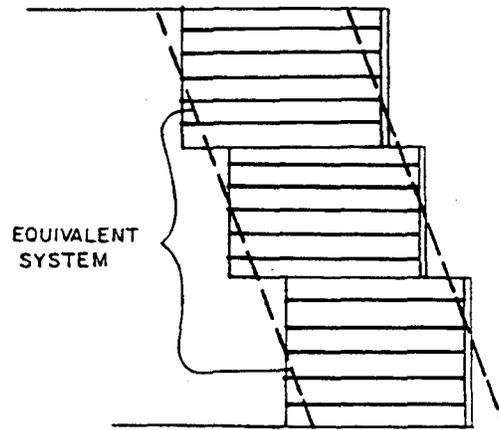


Figure 13. Poor conceptual design with increasing reinforcement lengths with depth.



(a) Actual structure.



(b) Equivalent structure for design.

Figure 14. Complex structures with multiple wall facing.

CHAPTER 2

PROJECT AND MATERIALS EVALUATION

2.1 INTRODUCTION

Parameters controlling the design of reinforced soil structures are discussed in this chapter. Subsurface exploration requirements necessary to define the site conditions are reviewed. Also described are the requirements for each component of the reinforcing system, including:

- Facings
- Reinforcement Materials
- Reinforced Fill Material.

Finally, soil-reinforcement interaction evaluation is reviewed.

2.2 SOIL AND SITE EXPLORATION

The feasibility of using a reinforced soil retaining structure 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.

It is particularly necessary to analyze slope failure mechanisms before repairing a slope to evaluate the applicability of the soil reinforcement technique in order to ensure stability after the retaining structure is in place. The conditions that prevail during the excavation of the slide material to obtain necessary space for the reinforced soil structure must be thoroughly explored.

Subsurface investigations are required not only in the area of the construction but also in neighboring areas which may affect the stability of the excavations before the reinforced soil structure is installed. The subsurface exploration program should be oriented not only towards obtaining all the information which could influence the design and the stability of the final structure, but also to the conditions which prevail throughout the construction of the reinforced soil structure.

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

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

a. 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 which may provide information on subsurface conditions.
- The extent, nature, and locations of existing or proposed below grade utilities and substructures which may have an impact on the exploration or subsequent construction.
- Available right-of-way.
- Areas of potential instability such as deep deposits of organic soils, slide debris, areas of 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 in this regard is available from the U.S. Geological Survey, the Soil Conservation Service, the U.S. Department of Agriculture, and Planning Boards or local county offices.

b. Subsurface Exploration

The subsurface exploration program generally consists of soil soundings, borings, test pits, and indirect methods including geophysical exploration techniques such as seismic refraction, electrical resistivity, or other special tests. The type and extent of the exploration should be decided after review of the preliminary data obtained from the field reconnaissance, consultation with a geotechnical engineer, or an engineering

geologist. In any event, the exploration must be sufficient to evaluate the geologic and subsurface profile in the area of construction.

The following minimum guidelines are suggested for the subsurface exploration:

- Soil borings should be performed at intervals of:
 - 100 ft (30 m) along the alignment of the wall.
 - 150 ft (45 m) along the back of the reinforced section of the wall.
 - 150 ft (45 m) in the area in front of the wall.

The width of the reinforced soil wall or zone of soil stabilization by soil nailing, etc. may be assumed as 0.8 times the anticipated height. For wall heights in excess of 50 ft (15 m), the back borings should be spaced at 100 ft (30 m) intervals, and, in addition, another row of borings should be performed along the midpoint between the face of the wall and the back of the reinforcement, at intervals of about 150 ft (45 m).

- The boring depth should be controlled by the general subsurface conditions. Where bedrock is encountered within a reasonable depth, rock cores should be obtained for a length of about 10 ft (3 m). This coring will be useful to distinguish between solid rock and boulders. Deeper coring may be necessary to better characterize rock slopes behind new retaining structures. In areas of soil profile, the borings should extend at least to a depth equal to twice the height of the wall. If subsoil conditions within this depth are found to be weak and unsuitable for the anticipated pressures from the wall height, or for providing an adequate medium for anchorage of soil nails in case of an in-situ reinforced soil, then the borings must be extended until reasonable soils are encountered.
- In each boring, soil samples should be obtained at 5 ft (1.5 m) depth intervals and at changes in strata for visual identification, classification, and laboratory testing. Methods of sampling may follow ASTM designation D-1586 or D-1587 (Standard Penetration Tests 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 get 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 provide data on the strengths and density of granular soils. In some situations, it may be desirable to perform in-situ tests using a dilatometer, pressuremeter, or similar means to determine soil modulus values.
- Adequate bulk samples of available soils should be obtained and evaluated as indicated in the following testing section to determine the suitability of the soil for use as backfill in the reinforced soil wall. 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.

c. Testing

Each soil sample should be visually examined and appropriate tests performed to allow the soils to be classified according to the Unified Soil Classification System (ASTM D-2488-69). A series of index property tests are sometimes performed which further aid in classification of the materials into categories and 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. The 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 reinforced soil walls and in-situ soil nailing projects. At sites where relatively weak and compressible cohesive soils are encountered below the foundations of the reinforced soil 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.

All samples of rock recovered in the field exploration should be examined in the laboratory to make an engineering classification including rock type, joint spacing and orientation, stratification, location of fissures, joints and discontinuities and strength. Representative cores should be tested for compressive strength. Any joint fill materials recovered from the cores should be tested to evaluate their effect on potential failure along the weakened planes. Detailed field investigation by an engineering geologist is advisable if rock stability is important at the site.

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-term and short-term conditions. Triaxial tests are also appropriate for this purpose. The compaction behavior of potential backfill materials should be investigated by performing laboratory compaction tests according to AASHTO standards.

Properties to indicate the potential corrosiveness of the backfill material must be measured. Tests include:

- pH.
- Electrical resistivity.
- Salt content including sulfate, sulfides, and chlorides.
- Oxidation agents such as soils containing Fe_2SO_4 , calcareous soils, and acid sulfate soils.

The test results will provide necessary information for planning corrosion protection measures and help in the selection of reinforcement elements with adequate durability. Recommended test methods are covered under specific reinforcement sections.

2.3 FACING SYSTEMS

The types of facing elements used in the different reinforced soil systems control their aesthetics since 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 drainage paths. The type of facing influences settlement tolerances. In multianchored structures, the facing is a major structural element. Major facing types are:

- Segmental precast concrete panels such as used in Reinforced Earth, the Georgia Stabilized Embankment System, the Mechanically Stabilized Embankment System, the Retained Earth System (VSL), the Reinforced Soil Embankment (Hilfiker), Tensar GeoWall, the American Geo-Tech System, the Stress Wall Systems, the TRES System, the WEBSOL system, the Tensar System, and the York System of the Department of Environment, United Kingdom. Soil nailing systems may also use precast facing panels.
- Cast-in-place concrete, shotcrete or full height precast panels - This type of facing is available in the Hilfiker and Tensar systems. Shotcrete is the most frequently used system for permanent soil nailed retaining structures.

- **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 are still used in structures where difficult access or difficult handling requires lighter facing elements. Pre-formed metallic facings are also used in some soil nailing systems.
- **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 and Tensar retaining wall system. Welded wire grid facing is also commonly used with soil nailing in fragmented rocks or intermediate soils (chalk, marl, shales).
- **Gabion Facing** - Gabions (rock-filled wire baskets) can be used as facing with reinforcing strips consisting of welded wire mesh, welded bar mats, polymer geogrids, or the double-twisted woven mesh used for gabions placed between the gabion baskets.
- **Fabric 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.
- **Plastic Grids** - A plastic grid used for the reinforcement of the soil 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 polymer and a pleasing appearance.
- **Postconstruction Facing** - For wrapped faced walls, whether geotextiles, geogrids, or wire mesh, a facing can be attached after construction of the wall by shotcreting, guniting, or attaching prefabricated facing panels made of concrete, wood, or other materials. Shotcrete is the most frequently used system for permanent soil nailed retaining structures.

Precast elements can be cast in several shapes and provided with facing textures to match environmental requirements and to blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure. In addition, the use of separate panels provide the flexibility to absorb differential movements, both vertically and horizontally, without undesirable cracking which could occur in a rigid 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 can, of course, be countered by providing shotcrete or 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) which 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 environment in the countryside. These facings as well as geosynthetic wrapped facings are especially advantageous for construction of temporary or other short-term design life structures.

2.4 REINFORCING MATERIALS

The following information on the reinforcement materials is needed for the design: geometric characteristics, strength and stiffness properties, durability, and soil reinforcement interaction properties. The two most commonly used reinforcement materials, steel and geosynthetics, are considered in this section.

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.
- Sheets and Geosynthetic Grids: A layer of sheet or geosynthetic grid is characterized by the width of the sheet or grid component and the center-to-center horizontal distance between the sheets or grid components. The cross-sectional area is not needed

since the strength of a sheet and a grid are expressed by a tensile force per unit width rather than by a 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. It is defined as follows:

$$R_c = b/s_h \quad (1)$$

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

In this section, the basis for determining the allowable tensile forces in the reinforcement per unit width of reinforcement, T_a , is given.

The following strength properties are required:

- The yield strength and modulus of the reinforcement.
- The long-term allowable design tensile capacity of the reinforcement, which may be dependent on the design life of the reinforced soil wall.

The long-term design strength of extensible reinforcement usually cannot be determined for a specific project due to the length of time required for testing. Therefore, such data generally has to be provided by other means or reduction factors must be used to account for potential creep, construction damage, and aging effects. In the case of metallic reinforcements, an allowance must be made for corrosion. This is done by increasing the metal cross section area to account for the estimated corrosion loss.

Steel Reinforcement

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

$$T_a = \frac{\sigma_a A_c}{b} \quad (2)$$

where: σ_a = allowable tensile stress = $0.55\sigma_y$
 σ_y = yield stress of steel¹

¹Task Force 27 recommends $0.47\sigma_y$ for welded wire mesh type reinforcements due to welded connections. However, for these

types of reinforcement, only the longitudinal membranes are used to determine T_a , and a reduction due to welded connections is not considered necessary. The strength of the junction is primarily a pullout consideration and requirements will be reviewed in the pullout section of this chapter.

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 (see section 2.5).

The quantities needed for determination of A_c for steel strips and grids are shown in figure 15. The use of hardened and otherwise low strain steels may increase the potential for catastrophic failure: therefore, an increased factor of safety may be warranted with such materials.

Geosynthetic Reinforcement

Selection of T_a for geosynthetic is more complex than for steel. The tensile properties of geosynthetics are affected by creep, construction damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer vary widely, and the details of polymer behavior are generally unfamiliar to civil engineers.

Ideally, T_a should be determined by thorough consideration of allowable elongation, creep potential and possible strength degradation using the method presented by Bonaparte and Berg.⁽¹²⁾ This method is complex and requires extensive long-term strength testing of the geosynthetic product.

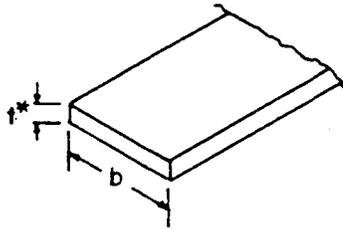
In the absence of sufficient test data, T_a can be calculated by the following simplified expression:

$$T_a = \frac{T_{ULT} (CRF)}{FD \cdot FC \cdot FS} \leq T_s \quad (3)$$

where:

- T_{ULT} = Ultimate (or yield tensile strength) from wide strip tensile strength tests (ASTM D-4595)
- T_s = long-term tension capacity of the geosynthetic at a selected design strain (usually 5% or less)
- FD = Durability factor of safety. (It is dependent on the susceptibility of the geosynthetic to attack by microorganisms and chemicals, thermal oxidation, and environmental stress)

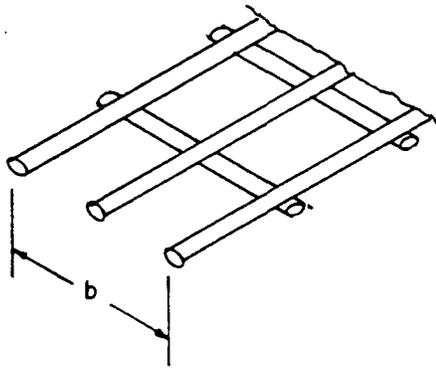
STEEL STRIPS:



$$A_c = b \cdot t^*$$

WHERE t^* = THICKNESS CORRECTED FOR
CORROSION LOSS

STEEL GRIDS



$$A_c = \text{NO. OF BARS} \cdot \pi \frac{d^{*2}}{4}$$

WHERE d^* = DIAMETER OF BAR OR WIRE
CORRECTED FOR CORROSION
LOSS

Figure 15. Calculation of A_c for steel reinforcement.

cracking and can range from 1.1 to 2.0. In the absence of product specific durability information, use 2.0. See section 2.5.b. for additional information on geosynthetic durability).

FC = Construction damage factor of safety. It can range from 1.1 to 3.0. In the absence of product specific construction damage test use 3.0. See section 2.5.b. for further information on controlling construction damage).

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, vertically faced structures, FS should be a minimum of 1.5.

CRF = Creep Reduction Factor ($CRF = T_1/T_{ult}$, where T_1 is the creep limit strength obtained from creep test results). If the CFR value for the specific reinforcement is not available, Task Force 27 provides the following recommendations:

Polymer Type	Creep Reduction Factors ²
Polyester	0.4
Polypropylene	0.2
Polyamide	0.35
Polyethylene	0.2

2.5 DURABILITY OF REINFORCEMENT SYSTEMS

The required service life of reinforced soil structures may exceed 75 to 100 years for permanent structures, at the end of which the structure should still be safe. The service life of a reinforced soil structure depends on the life (durability) of the reinforcing elements and to some extent on that of the facing.

Where metallic reinforcement is used, 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 also provide corrosion protection, provided the coating is not significantly damaged during construction. Epoxy coating can be used for corrosion protection, but it is also susceptible to construction damage which can significantly reduce

²Additional reduction should be made for applications in high temperature environments (temperatures greater than 90°F in the region of the reinforcement, e.g. at facing connection, in hot climates).

its effectiveness. When PVC or epoxy coatings are used, the maximum particle size of the backfill should be restricted to 3/4 in or less to reduce the potential for construction damage. Soil nails for permanent applications are generally protected from corrosion by the grout used during placement and by electrostatically applied resin bond epoxy. In aggressive environments, as with tiebacks, full encapsulation is usually recommended. Several State highway departments routinely use epoxy coated reinforcing elements. In cases where other metals, such as aluminum alloys or stainless steel have been used, corrosion, unexpectedly has been a severe problem.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking. In addition, it is susceptible to construction damage.

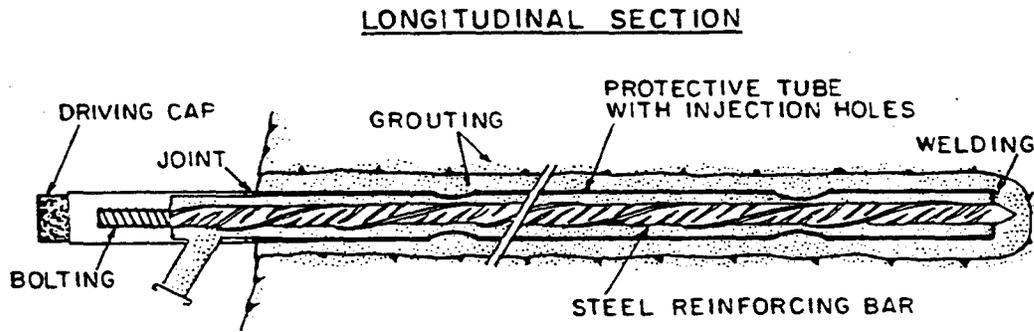
a. Metallic Reinforcement

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⁽¹³⁾. Based on these studies, deterioration of steel strips, mesh, bars and mats can be estimated and accounted for by using increased metal thickness.

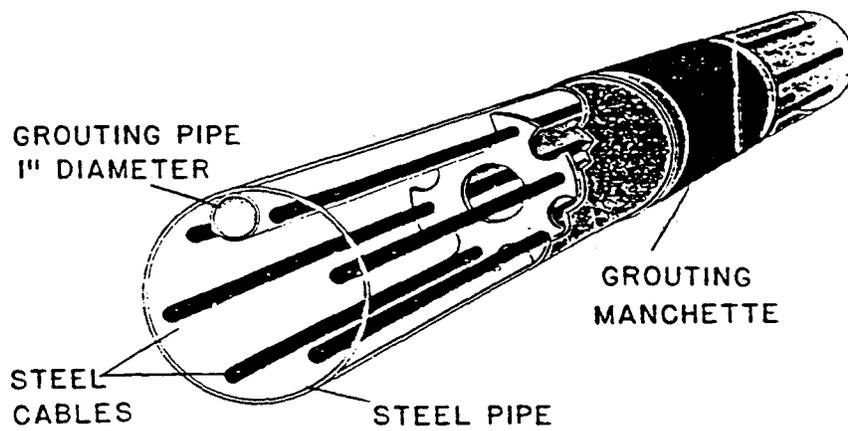
The majority of mechanically stabilized earth walls constructed to date have used galvanized steel and backfill materials with low corrosive potential. The zinc coating provides a sacrificial anode which corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion. After the zinc is oxidized, corrosion of the base metal starts. Stainless steel reinforcements have been used on projects to avoid corrosion problems.

Soil nails for permanent applications are usually corrosion protected by a minimum grout cover of 1.5 in (38 mm) along the total length. Secondary protection should be provided by electrostatically applied resin bond epoxy with a minimum thickness of about 18 mils (457 μ m). Full encapsulation is recommended for an aggressive (high saline or alkaline) environment. Encapsulation is accomplished by grouting the nail into a uniformly corrugated plastic or steel tube (figure 16a) to provide double protection. Prefabricated corrosion protected nails have been developed by the French contractor Intrafor-Cofor (figure 16b) using prestressed steel cables in compression tubes to maintain the grout under compression and prevent microcracking.

The corrosion of buried metals depends on the presence of dissolved salts in the soil, pH, porosity, and degree of saturation. Highest corrosion rates are produced by a high content of dissolved salts, a high chloride concentration, a high



A) "TBHA" nail patented and developed by SOLRENFOR for permanent structures.



B) Prestressed multireinforced nail "INTRAPAC" developed by INTRAFOR-COFOR, France.

Figure 16. Corrosion protection of soil nailing.

sulfate content, and acidic or alkaline pH conditions in the soil. Corrosion is also accelerated by electric currents produced by contact between dissimilar metals, stray currents, infiltration of chlorides and other salts after construction, stress level, oxygenization, and changes in water quality. To account for these factors, different corrosion rates are assumed for normal high quality backfill soils, saline soils, and in seawater environments.

Proposed FHWA Corrosion Rate Estimation

(Metallic Reinforcements)⁽¹³⁾

As previously stated, it is necessary for reinforcing elements to be designed with an additional metal area which corresponds to the corrosion losses anticipated to occur during the service life of the structure. The design reinforcement cross section should then be based on the area "A_c", defined by the relationship, $A_c = A_n - A_s$, where A_s is the sacrificial section lost to corrosion and A_n is the original reinforcement cross sectional area. The project design should be based on allowable material stress at the completion of the minimum service life. For permanent structures, suggested service lives are 75 years for routine applications and 100 years for abutments, structures directly supporting railroads, roadways, and other critical structures.

Corrosion rate predictions are very difficult and uncertain. A review of existing data indicates disagreement among several studies.⁽¹³⁾ The corrosion rates presented below are suitable for conservative design. These rates assume a mildly corrosive backfill material having the controlled electrochemical properties limits which are discussed later in this section under electrochemical properties.

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 a reinforced soil system 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. Because of limited data and the fact that several reinforcement layers may be within the chloride rich zone, higher rates of metal loss should be anticipated.

For permanent structures exposed to deicing salts, it is assumed that the upper 7.5 ft (2.29 m) of the reinforced backfill (as measured from the roadway surface) are affected by these higher rates. For walls directly supporting roadways where deicing salts

are used, 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.

For permanent bridge abutment applications, the permeation of salt-laden runoff through the expansion joints could result in a chloride rich environment near the face panel connections for a significant percentage of the wall height. For this condition, higher corrosion rates than assumed in the design could occur.

One method to minimize this problem is to control this seepage through the use of an impermeable membrane and drainage system. Figure 17 illustrates the type of detail typically used. This additional safety precaution is recommended for all permanent abutment applications.

Limitations of Design Procedure

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

- Structures exposed to a marine or other chloride rich environment.
- Structures exposed to stray currents such as from nearby underground power lines and structures supporting or located adjacent to electrical railways.
- Structures constructed with reinforced backfill materials which fall outside the electrochemical property criteria presented in this chapter.
- Reinforced soil wall systems where the reinforcing elements are not electrically isolated from any metal in the facing elements. (This includes systems where the reinforcement material is used to form the facing.)
- The use of metal reinforcing elements which are not galvanized with at least 2.0 oz/ft^2 (610 g/m^2) coating.

Each of these situations creates a special set of conditions which should be specifically analyzed by a corrosion specialist. Alternatively, noncorrosive reinforcing elements should be considered.

Tests to Determine Corrosion Rates and Electrochemical Soil Properties

The corrosion rate previously indicated is based on correlations between electrochemical backfill properties and measured rates of corrosion. Three available methods to determine corrosion rates are box tests, electrochemical cells, and measurements taken on

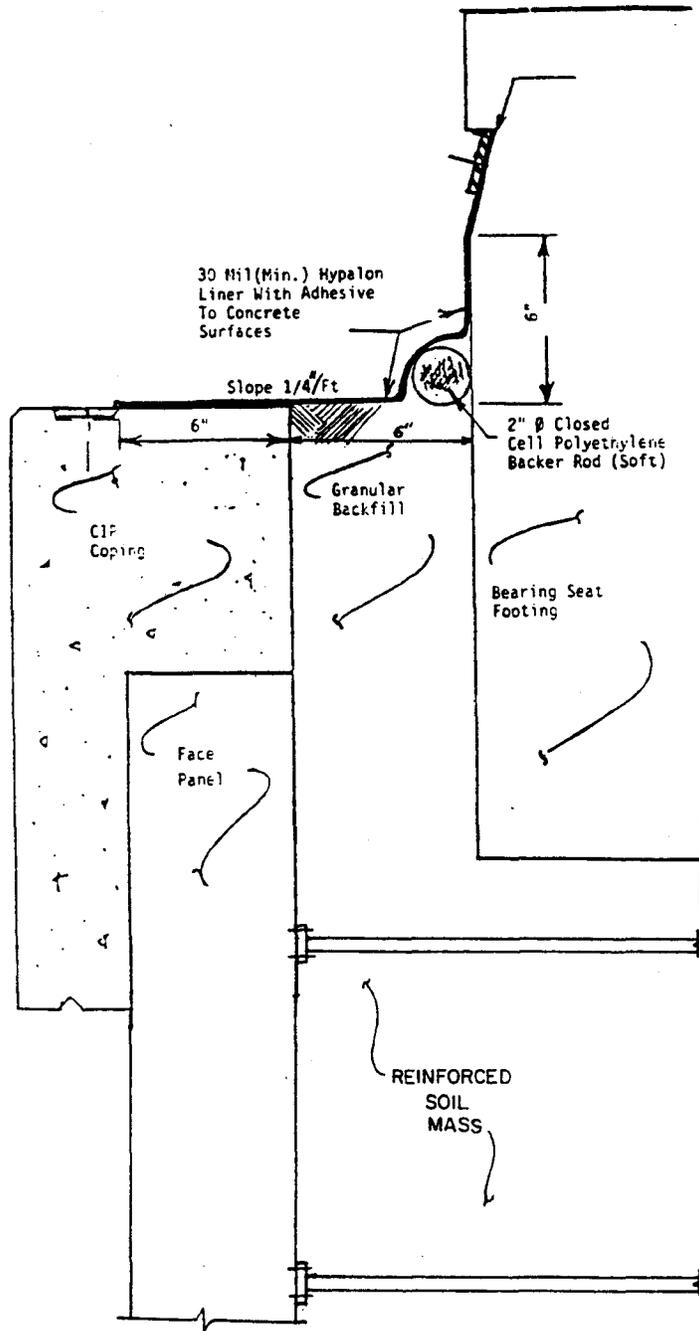


Figure 17. Abutment seat detail to prevent salt water runoff from discharging into reinforced earth mass.

actual structures. Measurements on actual reinforced soil structures provide the most reliable data. Each method is briefly discussed below:

- **Box Tests** - Specimens of metal are placed into soils with known properties, and metal weight loss is determined as a function of time. Test results may vary due to small sample size and test duration. Correlation to actual design models is generally lacking.
- **Electrochemical Cell Tests** - The principle of this test is that the current generated during the corrosion process is related to the amount of metal dissolved per unit time. These tests are adequate for parametric studies but are not suitable for predicting actual corrosion rates.
- **Measurements on Reinforcements** - This type of testing provides the best information on corrosion rates and should always be considered for the design of critical structures. Observations have been made on 46 full-sized Reinforced Earth structures over the last 15 years.⁽¹³⁾ Measurements can be by removal and evaluation of test coupons on a periodic basis or preferable yet much more complicated by measurements of "real time" corrosion rates on full-size reinforcing elements. A method of real time measurements is currently being used in an FHWA sponsored research study, "Durability/Corrosion of Reinforced Soil Structures".

Data on the long-term corrosion performance of soil nails are very limited because the technology has only recently been implemented in permanent structures. Therefore, recommendations for durability evaluation and corrosion protection are essentially based on relevant field experience with permanent ground anchors which has been reviewed and updated for soil nailing under a separate FHWA contract.⁽¹⁴⁾

Reinforced backfill corrosivity is defined in terms of resistivity, pH, chloride, and sulfate content. The corrosion rates presented in the previous section are consistent with backfill soils exhibiting certain minimum or maximum values of these properties which are generally associated with soils classified as "moderately to mildly corrosive".

- **Resistivity** - The following qualitative relationship is generally accepted:

<u>Aggressiveness</u>	<u>Resistivity ohm-cm</u>
very corrosive	< 700
corrosive	700 - 2,000
moderately corrosive	2,000 - 5,000
mildly corrosive	5,000 - 10,000
noncorrosive	> 10,000

- pH - Soils that are acidic (pH less than 4.5) or alkaline (pH greater than 9) are generally associated with high corrosion rates in carbon steel. In addition, galvanization is strongly attacked in highly acidic or alkaline soils.
- Water Content - Maximum corrosion rates generally occur at saturations of 60 to 75 percent. This range of saturation roughly corresponds to the moisture content range required for controlled fill placement.
- Soluble Salts - Chloride and sulfate accelerate corrosion by disrupting the formation of protective layers (i.e., zinc's corrosion by-product protects carbon steel).

The following recommendations for reinforced soil backfill electrochemical properties define a corrosive environment to provide additional safety against exceeding the predicted corrosion rates presented earlier.⁽¹³⁾

Table 4. Recommended electrochemical properties of suitable backfill for using metallic reinforcement

Property	Criteria	Test Method
resistivity	> 3,000 ohm-cm	California DOT No. 643
pH	> 5 - < 10	California DOT No. 643
chlorides	< 200 ppm	California DOT No. 422
sulfates	< 1000 ppm	California DOT No. 417

Test procedures should agree with FHWA Publication RD89-186. For routine applications, unless other durability considerations control the performance of the reinforcement system, (i.e. stray currents, marine environment) the use of galvanization in accordance with the requirements of ASTM A-123 is recommended (2.0 oz/ft² [610 gm/m²] of exposed surface area; thickness = 86 μm). Use of epoxy coatings for routine corrosion environments provides no greater degree of design confidence than galvanization. Plain, uncoated steel, without galvanization, has been used to a limited extent for permanent applications. However, based on the current lack of sufficient experimental data on steel corrosion rates, uncoated steel reinforcement is not recommended.

Until further findings from the FHWA corrosion research program become available, the California procedures are recommended. Current AASHTO and ASTM method are not considered to provide adequate sensitivity or repeatability.

Epoxy Coatings

Fusion bonded epoxy coatings have increasingly been used by the construction industry to mitigate the effects of corrosion. A number of mechanically stabilized earth structures have recently been constructed with epoxy coated tensile reinforcements, and

their use appears to be increasing. The majority of these installations have been for special applications where there has been either heavy application of deicing salts or where lower quality reinforced backfill (based on electrochemical properties) had to be used. Recommendations for soil nails generally specify that secondary protection be provided by electrostatically applied resin bond epoxy.

The coatings suppress the anodic and cathodic reactions leading to electrochemical corrosion. They hinder the passage of electrons or ions between the anodic locations on the metal and the electrolyte surrounding the outer surface of the coating. The ability of a particular coating to prevent electrochemical reactions (corrosion) is governed by:

- Permeability properties.
- Diffusion properties.
- Osmotic flow processes.

In addition, a coating must be properly applied and be of appropriate thickness to perform its function. Appropriate application techniques can be assured by following suitable specifications. Strip shaped elements may be coated in accordance with AASHTO M-284 (epoxy coated reinforcing bars). The American Society for Testing and Materials (ASTM) standard for epoxy coated welded wire fabric (grid reinforcement) is A884-88. The latest version of the proposed standard includes considerations related to coating thickness, holidays, and coating adhesion.

To aid in determination of appropriate coating thickness, Terre Arme International sponsored an extensive research study in which corrosion resistance, laboratory abrasion resistance and the extent of coating damage during construction were measured. Damage was evaluated after each of the following project phases: handling during storage, placement of reinforcements within the backfill and construction traffic. For all tests, the extent of damage was considerable where coating thickness was less than 10 mils (254 μm). Results of this study suggest the 5 to 12 mil (127 to 305 μm) range provided by AASHTO M-284 is not satisfactory for permanent reinforced soil applications.

The FHWA currently recommends an 18 mil (457 μm) thickness for all permanent reinforced soil structures for the following reasons: the critical nature and long service life of these installations, the need to minimize (if not eliminate) "holidays" (voids) along the reinforcing element surface, results of the Terre Arme research study, and the current lack of long term performance data on the integrity of coatings in underground conditions.⁽¹³⁾ It is important to recognize that no provisions for sacrificial steel are currently made when using epoxies. Hence, the protective coatings must function as intended for the entire life of the structure. For design life of epoxy coatings, refer to FHWA Publication RD89-186.

Recent research indicates a high susceptibility to construction damage when epoxy coated reinforcements are used in coarse (plus 3/4 inch [19 mm]) aggregate which could completely negate its effectiveness.

PVC Coatings

PVC coatings are often used over galvanized woven wire mesh to provide an additional level of corrosion protection. Although there are records demonstrating the effectiveness of such treatments in retarding corrosion of gabion structures, as with epoxy coatings, there is some question as to the susceptibility of the PVC coating to construction damage. For use of PVC coated reinforcements, it is recommended that the maximum aggregate size should be limited to 3/4 in (19 mm) and field trials be performed to determine the potential for construction damage.

b. Durability of Polymeric Reinforcement

Nonmetallic materials used for soil reinforcement, such as the polymers used for geotextiles and geogrids, are less susceptible to corrosion. They have variable resistance to chemical attack, seawater, and biological activity. Degradation most commonly occurs from mechanical damage, loss of strength due to creep, and deterioration from exposure to ultraviolet light.

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 traffic directly on the material.

Table 5 provides relationships for the severity of loading imposed on the geotextiles to various construction conditions. The severity of these loading conditions can be related to the strength requirements for geotextiles that are anticipated to survive those conditions. For example, on a project where coarse angular gravel fill is to be placed at an 8 in (203 mm) compacted lift thickness using medium weight dozers, as a minimum, a geosynthetic with moderate to high strength should be used to reduce the potential for damage. A moderate strength geosynthetic has a wide width strength (ASTM D4595) on the order of 75 to 100 lb/in (13.1 to 17.5 kN/m).

Even when using high strength geosynthetics, some damage to the reinforcement may still occur. Any damage due to construction operations, sometimes called "site damage", will decrease its strength. Preliminary evidence indicates that under severe loading tensile strength reduction of up to 60 percent can occur. It is recommended that the strength of the geosynthetic be decreased by a factor of 1.1 to 3, to account for possible construction damage, depending on the construction conditions and experience. In the absence of any other information, the

Table 5. Relationship of construction elements to severity of loading imposed on geotextile in roadway construction.

VARIABLE	SEVERITY- CATEGORY		
	LOW	MODERATE	HIGH TO VERY HIGH
Equipment	Light weight dozer (8 psi)	Medium weight dozer; light wheeled equipment (8-40 psi)	Heavy weight dozer; loaded dump truck (>40 psi)
Subgrade Condition	Cleared	Partially cleared	Not cleared
Aggregate	Rounded sandy gravel	Coarse angular gravel	Cobbles, blasted rock
Lift Thickness (in.)	18	12	6

allowable strength of the geosynthetic should be decreased by 50 percent (or double the required design strength).

In all cases, the contractor should demonstrate that the proposed construction techniques will not severely damage the reinforcement.

The polymer formulation and resin additive package of the geosynthetic must be compatible with the chemistry of the backfill and the potential for the environment within the backfill to change with time. The backfill should be checked for such items as high and low pH, chlorides, organics, and oxidation agents such as soils which contain Fe_2SO_4 , calcareous soils, and acid sulfate soils which may result in deterioration of the geosynthetic with time. Other possible detrimental environmental factors include chemical solvents, diesel, and other fuels, active slag fills, and industrial wastes.

Because of varying polymer 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. As such, the manufacturer of the geosynthetic should supply the results of exposure studies on the specific product including, but not limited to, strength reduction due to aging of the microstructure, chemical attack, microbiological attack, environmental stress cracking, hydrolysis, and any possible synergism between individual factors. AASHTO-AGC-ARTBA Task Force 27 on Ground Modification Systems tentatively recommends that the allowable strength of the geosynthetic should include a safety factor FD of 2.0 unless such information is provided, and in all cases it should be decreased by a minimum of 10 percent. Durability of geosynthetics is the subject of important ongoing FHWA research.

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, then 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. Ultraviolet stability should be evaluated on a product-specific basis.

2.6 REINFORCED FILL MATERIALS

Most soil reinforcing systems specify high quality backfill in terms of durability, drainage and friction consisting of well graded, granular materials. Many of the soil reinforcement systems depend on friction between the reinforcing elements and

the soil. In such cases, generally a material with high friction characteristics is specified and required. Some systems of soil reinforcement rely on passive pressure on reinforcing elements and in those cases, the quality of backfill is still critical. These requirements generally eliminate soils with high clay contents.

Other systems of soil reinforcement, such as the Anchored Earth system (still in the experimental stage), the American Geo-Tech System, the Mechanically Stabilized Embankment system, and Tensar GeoWalls have less rigid specifications for the backfill materials. In the latter cases, cohesive backfills have been reported to perform satisfactorily, but only limited performance data are available. From a reinforcement point of view alone, much lower quality backfills could be used for all wall and slope systems than are used at present; 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 handling, placing and compaction advantages in using granular soils which tend to speed up construction.

The vendors of proprietary reinforcement systems have their own criteria for the backfill. Nonetheless, detailed specifications should be provided by the contracting agency and they should only be different from the recommendations herein only if appropriate justification can be made. It is pertinent to give gradations and soundness test results of the material available locally and from possible borrow sources in the vicinity so that these can be considered for possible use in the reinforced soil structure.

The following gradation and soundness limits are given in the FHWA specifications for mechanically stabilized earth walls with metallic reinforcement as recommended by AASHTO-AGC-ARTBA, Joint Committee Task Force 27. The following specification could also be used for construction of walls using geosynthetics in the absence of any other information concerning the applicability of a geosynthetic with available fill.

- Select Granular Fill Material for the Reinforced Zone. All backfill material used in the structure volume 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>U.S. Sieve Size</u>	<u>Percent Passing</u>
4 in (102 mm) ⁽³⁾	100
No. 40 (0.425 mm)	0-60
No. 200 (0.75 mm)	0-15

Plasticity Index (PI) shall not exceed 6.

⁽³⁾ As a result of recent research on construction survivability of geosynthetics and epoxy coated material, it is recommended that the maximum particle size for these materials be reduced to 3/4 in (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.⁽¹⁵⁾

(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 an equivalent sodium sulfate value) of less than 30 percent after four cycles.

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 above and below optimum. The compaction requirements of backfill are different in close proximity of the wall facing (within 5 to 6 ft, 1.5 to 2 m) of the wall face. Lighter compaction equipment is utilized 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 'pea' gravel is recommended close to the face of the wall to provide adequate strength and tolerable settlement in this zone. For the backfill adjacent to abutments, soundness requirements should be more stringent.

Special attention must also be focused on design details such as internal and external drainage. Specific details for drainage construction are given in chapter 7 for different reinforcement systems.

Lower quality fill materials could be considered for the construction of reinforced embankment slopes, but should be limited to moderate frictional materials ($\phi > 25^\circ$) with low cohesion (PI < 20).

2.7 IN-SITU SOILS SUITABLE FOR SOIL NAILING

Assessment of the suitability of the subsurface soil (or rock) to provide short- and long-term pullout capacity of soil nails

requires determination of the shear strength and creep characteristics of the soil.

In rocks, the overall strength is controlled by the presence and location of fissures, joints, or other discontinuities. Highly fractured rocks with open joints and cavernous limestone are difficult to grout and should preferably be avoided.

Soil nails are not cost effective in loose granular soils with SPT blow count number (N) lower than 10 or relative density lower than 30 percent. Nailing becomes practically unfeasible in cohesionless soils with a uniformity coefficient less than 2. This is because, in such cases, nailing requires stabilization of the cut face prior to excavation by grouting or slurry wall construction.

In fine grained cohesive soils, long-term pullout performance of the nails is a critical design criterion. Similar to ground anchors, soil nails will generally not be used in soft cohesive soils with undrained shear strength lower than 0.5 tsf (50 kPa) or soils susceptible to creep. A number of national codes (German Standards and French Recommendations) index the creep susceptibility to the Atterberg limits and natural moisture content of the soil. They preclude the use of permanent ground anchors in organic soils, and plastic clayey soils with liquid limit (LL) greater than 50 and liquidity index (LI) greater than 0.2 [or consistency index (I_c) less than 0.9]. Soils with a plasticity index (PI) greater than 20 must also be carefully assessed for creep. At present, in light of the limited experience with soil nailing in clayey soils, the applicability criteria developed for ground anchors are recommended for feasibility evaluation of nailed soil structures.

2.8 SOIL REINFORCEMENT INTERACTION

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.

As discussed under section 1.4, 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 versus linear or planar elements, thickness of in-plane or out-of-plane transverse elements, and aperture dimension to grain size ratio). 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 and the soil type.

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 (see section 2.6). Therefore, creep is primarily an issue of the type of reinforcement. Table 6 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 cohesive) soils.

b. Estimate of the Reinforcement Pullout Capacity in Embankments and Retaining Walls

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.

In this manual, a normalized definition is recommended. The pullout resistance, P_r , of the reinforcement per unit width of reinforcement is given by:

$$P_r = F^* \cdot \alpha \cdot \sigma'_v \cdot L_e \cdot C \quad (4)$$

- where: $L_e \cdot C$ = the total surface area per unit width of the reinforcement in the resistivity 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; $C=\pi$ for nails
 F^* = the pullout resistance (or friction-bearing-interaction) factor

Table 6. Basic aspects of reinforcement pullout performance in granular and low cohesive soils.

Reinforced Soil System	Generic Reinforcement Type	Major Load Transfer Mechanism	Displacement to Pull-Out	Long Term Performance
Mechanically Stabilized Embankments	Inextensible strips	Frictional		
	smooth	L.D.	0.05 in	Noncreeping
	ribbed	H.D.	0.5 in	
	Extensible composite plastic strips	Frictional	Dependent on reinforcement extensibility	Dependent on reinforcement structure and polymer creep
	Extensible sheets	Frictional (interlocking)	Dependent on reinforcement extensibility	Dependent on reinforcement structure and polymer creep
	geotextiles	L.D.	(1 to 4 in)	
	Inextensible grids			
	bar mats	Passive H.D.	0.5 to 0.8 in	Noncreeping
	welded wire meshes	Frictional + passive H.D.	0.5 to 0.8 in	Noncreeping
	Extensible grids			
geogrids	Frictional + passive H.D.	Dependent on extensibility (1 to 2 in)	Dependent on reinforcement structure and polymer creep	
woven meshes	Frictional + passive H.D.	1 to 2 inch	Noncreeping	
Anchors	Passive	0.2 to 0.4 in	Noncreeping	
Ground Reinforcement	Nails (ground reinforcement)	Frictional H.D.	0.08 to 0.12 in	Noncreeping

Note: L.D. - low dilatancy effect H.D. - high dilatancy effect 1 in = 25.4 mm

α = a scale effect correction factor
 σ'_v = the effective vertical stress at the soil-reinforcement interfaces

The pullout resistance factor F^* can most accurately be obtained from pullout tests performed in the specific backfill to be used on the project. 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:

$$\begin{aligned}
 F^* &= \text{Passive Resistance} + \text{Frictional Resistance} \\
 \text{or, } F^* &= F_q \cdot \alpha_p + K \cdot \mu^* \cdot \alpha_f \quad (5)
 \end{aligned}$$

where: F_q = the embedment (or surcharge) bearing capacity factor
 α_p = a structural geometric factor for passive resistance
 K = a ratio of the actual normal stress to the effective vertical stress; it is influenced by the geometry of the reinforcement
 μ^* = an apparent friction coefficient for the specific reinforcement
 α_f = a structural geometric factor for frictional resistance

The pullout capacity parameters for equation 5 are summarized in table 7 for the soil reinforcement systems considered in this manual. The scale effect correction factor α indicates the nonlinearity of the $P - L$ relationship. Due to the extensibility, the application of a pullout force on the reinforcement results in a decreasing shear displacement distribution over the length of the reinforcement. The interface shear stress is therefore not uniformly mobilized along the total length of the reinforcement. The average shear stress τ_{av} mobilized at the peak pullout load depends upon the reinforcement elongation during pullout which in turn depends upon the extensibility of the reinforcement materials and the reinforcement length. The scale effect correction factor α can be defined as:

$$\alpha = \frac{\tau_{av}}{\tau_p} = \frac{\tan \rho_m}{\tan \rho_{p \cdot a \cdot k}}$$

where: τ_{av} and τ_p are, respectively, the average and ultimate interface lateral shear stresses mobilized along the reinforcement.

ρ_m and $\rho_{p \cdot a \cdot k}$ are, respectively, the average and peak interface friction angle mobilized along the reinforcement.

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 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.

A summary of the procedures for performing and evaluating tests to obtain pullout design parameters F^* and α along with a theoretical discussion of empirical methods is included in chapter 2 of volume II, Summary of Research and Systems Information. Also covered in volume II are analytical procedures for evaluating displacement and creep potential from pullout tests.

Table 7. Summary of pullout capacity design parameters

Interaction Mechanisms

Reinforcement Type	Trade Name	Friction			Passive		S _{opt} /t	
		μ*	K	α _f	F _q	α _β		α
Inextensible strips ribbed	Reinforced Earth	tan φ ≤ μ* ≤ μ ₀	1	1	N.A.	N.A.	1	N.A.
	smooth	tan ρ _{ds}	1	1	N.A.	N.A.	1	N.A.
Extensible plastic strips	Paraweb	tan ρ _{ds}	1	1	N.A.	N.A.	<1*	N.A.
	Geotextiles	tan ρ _{ds}	1	1	N.A.	N.A.	<1*	N.A.
Inextensible grids	VSL	N.A.	N.A.	N.A.	15(F _q < 30 Fig. 37, Vol. II (or Eq. 9, Vol. II))	$\frac{1}{2} \cdot \frac{t}{S_x}$	1	$\frac{F_q}{2 \tan \phi}$
	Meshes	tan ρ	0.75	(π/2) · (D/S _y)	37 (Eq. 9, Vol. II)	$\frac{1}{2} \cdot \frac{t}{S_x}$	1	-25
Extensible grids	Geogrids and woven wire mesh	tan ρ	1	α _s	-20 Fig. 36, Vol. II	$\frac{f_b}{2} \cdot \frac{t}{S_x}$	<1 ^(a)	$\frac{f_b}{q} \cdot \frac{t}{2 \tan \phi}$
	Anchors	N.A.	N.A.	N.A.	Eqs. 11 and 12, Vol. II	$\frac{1}{2} \cdot \frac{t}{L}$	1	---
Geotech TREES	N.A.	N.A.	N.A.	Eq. 9, Vol. II	$\frac{1}{2} \cdot \frac{t}{L}$	1	---	

Note: More specific pullout information for each reinforcement type is contained in Volume II, Chapter 2.
*In the absence of pullout information to evaluate α, use α = 0.6.

Table 7. Summary of pullout capacity design parameters (continued).

D	= Rod diameter
F*	= Pullout resistance factor
f_b	= The fraction of the transverse member on which bearing can be fully developed
F_q	= Embedment (or surcharge) bearing capacity factor
K	= Ratio of actual normal stress to effective vertical stress
L	= The length from the face of the wall to the anchor
S_{opt}	= Optimal spacing of transverse elements
S_x	= Longitudinal spacing between transverse elements
S_y	= Lateral spacing between longitudinal elements
t	= Thickness of the bearing member
α	= Scale effect correction factor
α_p	= Structural geometric factor for passive resistance
α_f	= Structural geometric factor for frictional resistance
α_s	= Fraction of the solid surface area of the grid
ϕ	= Angle of internal friction
μ^*	= Apparent friction coefficient for the specific reinforcement
μ_o	= An estimate of the apparent friction coefficient based on the uniformity coefficient C_u of the soil
ρ	= Interface friction angle mobilized along the reinforcement
ρ_{ds}	= Interface friction angle obtained from direct shear test
τ	= Shear stress
τ_{ds}	= Shear strength of the soil obtained from direct shear test

CHAPTER 3

DESIGN OF REINFORCED FILL WALLS

3.1 INTRODUCTION AND BASIS

a. Purpose, Scope, and Organization

This chapter contains general and simplified design guidelines common to all reinforced soil wall systems. It is limited to reinforced soil walls having a near-vertical face (i.e., face inclination of 70° to 90°) and horizontal rows of the same length and type of reinforcement that retain a homogeneous backfill. Evaluation of complex structures is reviewed. Structures with face inclinations of less than 70° are covered in section 3.8.c and in chapter 4.

The approach presented herein provides a unified evaluation method for any system so that the suitability of a given design can be determined for a specific project. Thus, the engineer should be able to perform a preliminary design to determine the acceptability of reinforced soil for a specific project, perform a rapid check of designs provided by others, and design simple systems. It is not intended to replace proprietary system designs. The design methods presented in this chapter are based on current experience, which is limited to:

- Structures up to 100 ft (30 m) in height for inextensible steel reinforcement.
- Structures up to 50 ft (15 m) in height for extensible polymer reinforcement.
- Vertical reinforcement spacings ranging from 0.5 to 3 ft (150 mm to 910 mm).
- Granular backfill (see chapter 2).
- Segmented and flexible facing systems.
- Structures with adequate drainage to eliminate hydrostatic water pressure (see chapter 2).

The design systems with complex features should be referred to experts as such designs may require sophisticated methods that are beyond the scope of this manual. For example, the influence of full height rigid panels on the design methods is not fully understood at this time. Other examples include compound failure analysis of composite structures and structures with multiple facings.

The chapter is organized to provide practical, step-by-step design guidance.

- Section 3.1 establishes the basis of the method.

- Section 3.2 gives an outline of all the design steps needed to design a reinforced soil wall.
 - Sections 3.3 to 3.6 give the methods needed to analyze the external stability and the local internal stability.
 - Section 3.7 presents a design example in detail.
 - Section 3.8 gives general guidelines for the design of more complex reinforced soil walls.
 - Finally, contained in volume II - Summary of Research and Systems Information are some of the most significant results of recent research carried out on the subject. Background on the external and internal stability analyses used in the design guidelines compared to current practice is also included.
- b. Fundamental Mechanisms and Behavior of Reinforced Fill Walls

The design/analysis guidelines presented in this chapter, along with most other current design procedures for reinforced fill walls, are empirically modified versions of conventional procedures for concrete gravity walls. Because the theoretical framework for gravity wall design has proved to be successful for reinforced soil walls and because readily implementable methods based on more rigorous approaches have not yet been developed, reinforced soil designers have focused on modifications and extensions to conventional theory based on observed field and model behavior. Therefore, the design method in this manual, as well as most other current methods, has a large empirical component. An example of this is that values of horizontal and vertical soil stresses used in the design methods are generally only accurate in an average sense, while the values of reinforcement tensions that are used are in good agreement with those that have been measured in actual structures.

Extensible and Inextensible Reinforcements

The horizontal reinforcements in a reinforced soil wall act by restraining lateral displacement of the reinforced fill. The extensibility of the reinforcements compared to the deformability of the fill is an essential feature of the behavior of the wall, as it controls the state of horizontal stress in the reinforced soil mass. Inextensible reinforcement creates a relatively unyielding mass, such that the state of horizontal stress approaches an at-rest, K_0 , condition, while with extensible reinforcement, the fill can yield laterally so that an active state, K_A , condition can be reached throughout the reinforced soil mass.

Table 2 in chapter 1 indicates the extensibility of different systems. Extensibility depends on the material used for reinforcement (metal versus nonmetal), its geometric form, and the influence of confinement. Guidelines for evaluating the insoil extensibility of the reinforcement are provided in volume II.

Reinforcements constructed with linear metal elements (such as metal strips and bar mats) are inextensible; i.e., the reinforcements can rupture before the soil reaches a failure state. Low modulus geotextiles (such as nonwovens) are extensible. Other materials such as geogrids, wire woven at oblique angles (e.g., gabion materials), and woven geotextiles are in between truly extensible and inextensible materials. However, as these materials can deform substantially before failure, i.e., the soil reaches a failure state before reinforcement ruptures, they are generally assumed as extensible for design purposes.

Other features of the wall that affect the behavior include the density of the reinforcement, the type of facing and the rigidity of the connections between the reinforcement and the facing.

Reinforcement Tension

The variation of the tensile forces along the reinforcement and the location of the maximum force has been established both experimentally, through instrumented models and full-scale structures, and theoretically, using numerical analysis.

As shown in figure 18a, the maximum tensile force in the reinforcement is generally located some distance behind the facing. In order to create a maximum force at that location, the shear stresses exerted by the fill on the reinforcement must be in opposite directions on the two sides of the peak force as shown.

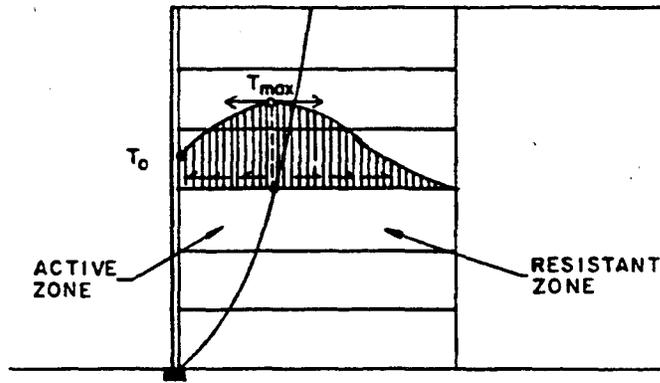
The locus of the points of maximum tensile force, called the maximum tensile forces line, thus separates the reinforced fill into two zones:

- An active zone between the facing and the maximum tensile forces line, where the shear stresses on the reinforcements are directed towards the wall face.
- A resistant zone behind the maximum tensile forces line, where the shear stresses on the reinforcement are directed away from the wall face.

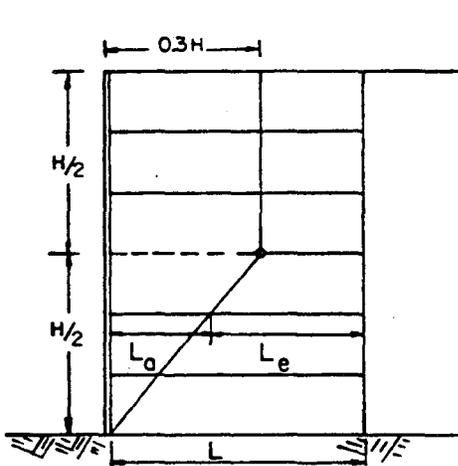
The global effect is that the tensile force generated in the reinforcement by the soil in the active zone is transferred through the reinforcement back to the soil in the resistant zone.

The location of the maximum tensile forces line is influenced by the extensibility of the reinforcement as well as the overall stiffness of the facing. Figures 18b and 18c show the limiting locations of the maximum tensile forces line in walls with inextensible and extensible reinforcements:

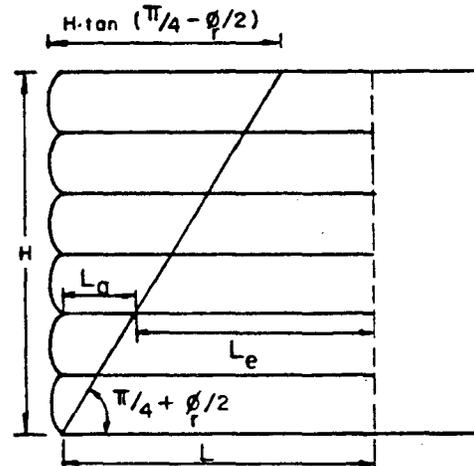
- With inextensible reinforcements (figure 18b), the maximum tensile forces line can be modeled by a bilinear failure surface which is vertical in the upper part of the wall. The state of stress is assumed to be



(a) general case



(b) inextensible reinforcements



(c) extensible reinforcements

Figure 18. Tensile forces in the reinforcements and schematic maximum tensile force line.

at rest at the top and decreases to the active state in the lower part of the wall (the at rest state at the top of the wall has been attributed to both construction stresses and the restraint provided by the reinforcements against lateral yielding).

- With extensible reinforcements (figure 18c), the maximum tensile forces line coincides with the Coulomb or Rankine active failure plane, and the stresses in the fill correspond to the active earth pressure condition.

The location of the maximum tensile forces line may also vary due to external factors such as the shape of the structure and surcharge conditions.

Modes of Failure of a Reinforced Soil Wall

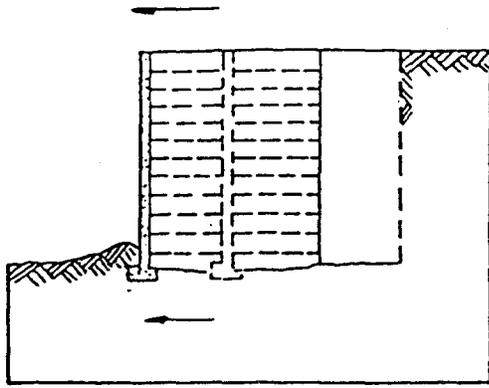
Reinforced soil wall design consists of determining the geometric and reinforcement requirements to prevent internal and external failure.

- Internal failure. As indicated previously in section 1.5.e, there are two modes of internal failure:
 - By breakage or excessive elongation of reinforcements.
 - By reinforcement pullout.

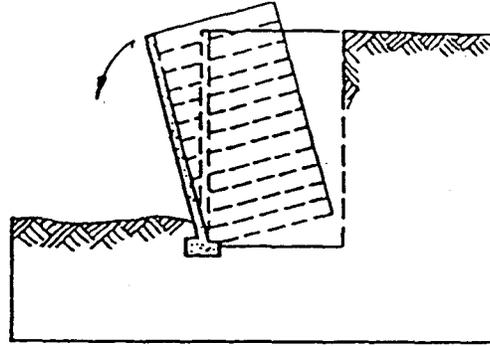
Each mode of failure can be analyzed using the maximum tensile forces line. This line is assumed to be the most critical potential slip surface. The length of reinforcement extending beyond this line will thus be the available pullout length.

- External failure. As with classical unreinforced retaining structures, four potential external failure mechanisms are usually considered for reinforced soil structures, as shown in figure 19. They include:
 - Sliding on the base.
 - Overturning.
 - Bearing capacity failure.
 - Deep seated stability failure (rotational slip-surface or slip along a plane of weakness).

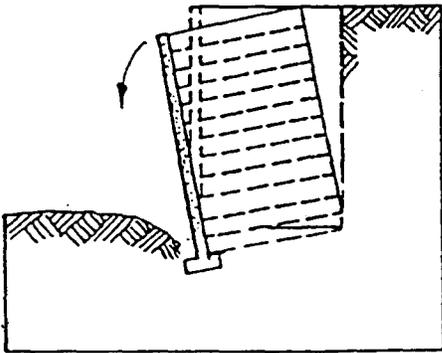
Due to the flexibility and satisfactory field performance of reinforced soil walls, the adopted values for the factors of safety for external failure are lower than those used for reinforced concrete cantilever or gravity walls. For example, the factor of safety for overall bearing capacity is 2 rather than the conventional value of about 3, which is used for more rigid structures.



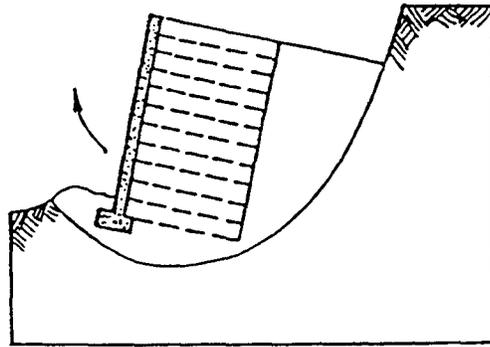
(a) sliding



(b) overturning



(c) bearing capacity



(d) rotational slip - surface failure

Figure 19. Potential external failure mechanisms of a reinforced soil wall.

Likewise, the flexibility of reinforced soil walls would make the potential for overturning failure highly unlikely. However, overturning criteria aid in controlling lateral deformation by limiting tilting and, as such should always be satisfied.

For simple structures with rectangular geometry, relatively uniform reinforcement spacing and a near-vertical face, it is sufficient to separately evaluate internal and external failures. However, in the case of complex geometrical reinforced soil wall designs, it is generally not possible to separate the internal failure from the external failure, because the most critical slip surface can be partly in the wall and partly outside. (Guidelines for evaluating complex structures are covered in Section 3.8.)

Influence of Foundation and Retained Backfill Settlement

The influence of the compressibility of the foundation on the tensile forces in the reinforcements and on the horizontal deformations at wall face are reviewed in the Reinforced Fill Walls section of volume II. It appears that a compressible foundation soil slightly increases the tensile forces in the lower reinforcement layers in the case of inextensible reinforcements, as well as the global lateral displacement of the wall. As long as the settlement is uniform for practical design, this influence can be neglected. However, differential settlement increasing from the back to the front of the wall will result in a more significant increase in the lower level reinforcement tension and stresses at the facing connections for inextensible reinforcement.

The relative settlement of the retained fill with respect to the reinforced fill influences the inclination of the thrust at the back of the reinforced soil wall. If the same fill material is used for the reinforced fill and for the retained fill and if the same soil foundation supports both the reinforced soil wall and the retained fill, the thrust may be inclined at an angle λ for inextensible reinforcement as will be discussed in section 3.3. If either condition is not true, then the direction and the order of magnitude of the relative settlement has to be evaluated in order to adjust the λ value. For simplification, $\lambda = 0$ should be used in all cases where the settlement of fill within the reinforced fill section is anticipated to be greater than in the retained fill.

Differential settlement of the wall face with respect to the reinforced fill will lead to an increase in the reinforcement stress near the wall face. This stress increase will be largest for reinforcement rigidly connected to the facing because bending stresses will develop. If the potential for such conditions exist, special flexible connections should be considered or a flexible face wall constructed and the permanent facing attached after the settlement has taken place. Special connections need to be thoroughly evaluated and carefully designed for compatibility with the reinforcement and facing system and for constructability in cooperation with the manufacturer of the specific system.

c. Recent Research

The design methods in this chapter have been based on conservative procedures developed over the last 20 years, which have been substantiated and modified as necessary by recent research that has been performed as part of the study to prepare this manual, as well as other on-going research. The influence of that research on various aspects of the design methods are included in volume II - Summary of Research and Systems Information.

In summary, the research found that external design could be modified by inclining the thrust at the back of the wall, at least for inextensible reinforcement. Insufficient data was available to justify this approach for extensible reinforcement. This modification will allow for shorter base widths in the reinforced zone. For internal stability, a simplified approach was developed around the stiffness of the reinforced zone. The approach allows the influence of extensibility and density of reinforcement to be directly analyzed while decreasing the complexity of some of the previous models in terms of the distribution of stress in the reinforced zone. Finally, a first order approximation method of the anticipated total lateral deformation in the wall during construction was developed empirically based on the extensibility of the reinforcement and the reinforcement length to height ratio. A simple method with a good experimental base was not previously available. These procedures were incorporated into the following step by step design approach.

3.2 DESIGN STEPS

The following is a step-by-step outline for the design of reinforced soil walls of rectangular geometry and a near-vertical face (see table 8 for definitions of all terms). Further details of the procedure can be found in sections 3.3 through 3.5. Modification to these procedures for design of complex structures are reviewed in Section 3.8. The design steps are:

- Step 1: Establish design limits, scope of the project, and external loads (figure 20):
- a. External wall height, H_o .
 - b. Wall face batter, θ .
 - c. Total length of wall and variations in wall height along the length.
 - d. Slope angle β of soil surface.
 - e. External loads and their locations:
 - . Uniform surcharge load q .
 - . Concentrated surcharge loads, P_v , P_h .
 - . Traffic barriers.
 - . Seismic loading.
 - f. Type of facing and connections:
 - . Wrapped.
 - . Segmental concrete panels.
 - . Full height concrete panels.

Table 8. List of notations.

a_n	-	Maximum horizontal acceleration
A	-	Area
A_c	-	Cross sectional area of steel reinforcement minus estimated corrosion losses (see section 2.4)
b	-	The width of a reinforcing element
b_f	-	The width of a footing
c	-	Cohesion in terms of total stress
c'	-	Effective cohesion
c_f	-	Cohesion of foundation soil
c_u	-	Undrained shear strength
c_v	-	Coefficient of consolidation
C	-	Perimeter of reinforcing strip or bar
C_c	-	Compression index
C_r	-	Recompression index
d	-	Depth, diameter
D	-	Wall embedment
D_1, D_2	-	Effective width of applied stress with depth
D_r	-	Relative Density
E	-	Young's modulus
e	-	Eccentricity
F^*	-	Pullout resistance factor
FS	-	Factor of safety
FS_{bc}	-	Factor of safety with respect to bearing capacity
FS_o	-	Factor of safety with respect to overturning
FS_{po}	-	Factor of safety with respect to pullout
g	-	Acceleration due to gravity
h_s	-	Height of surcharge
H, H_o	-	Wall or slope height
H'	-	Wall or slope height modified to include uniform or sloping surcharge
J	-	Modulus of geosynthetic reinforcement
K	-	Stress ratio

Table 8. List of notations (continued).

K_A	=	Active earth coefficient of the retained backfill
K_{ab}	=	Lateral earth pressure coefficient based on Coulomb theory and peak angle of internal friction
K_o	=	Coefficient of earth pressure for at-rest condition
l	=	Length of footing
L	=	Length of reinforcement
L_a	=	Length of reinforcement in the active zone
L_e	=	Embedded length of reinforcement to resist pullout
L_t	=	Length of reinforcement required for internal stability
M	=	Moment, mass
M_d	=	Driving moment
M_{TR}	=	Resisting moment due to vertical component of thrust
M_{WR}	=	Resisting moment due to weight of mass above base
n	=	Number of reinforcement layers
N_c	=	Bearing capacity factor
N_γ	=	Bearing capacity factor
P_a	=	Resultant of active earth pressure
P_{aE}	=	Dynamic horizontal thrust
P_b	=	Resultant of active earth pressure due to the retained backfill
P_h	=	Concentrated horizontal surcharge load
P_I	=	Horizontal inertial force
P_q	=	Resultant of active earth pressure due to the uniform surcharge
P_r	=	Available pullout resistance
P_v	=	Concentrated vertical surcharge load
q	=	Surcharge load
q_s	=	Allowable bearing capacity
q_{ult}	=	Ultimate bearing capacity
R	=	Resultant force

Table 8. List of notations (continued).

R_c	=	Reinforcement coverage ratio
R_v	=	Resisting force (Meyerhof's approach)
R_e	=	Reduction coefficient
S_h, S_H	=	Center to center horizontal spacing of reinforcement strips or grids
S_m	=	Minimum vertical spacing of reinforcement
S_M	=	Maximum vertical spacing of reinforcement
S_R	=	Stiffness factor of reinforcement
S_v	=	Vertical spacing between the horizontal geogrid layers
T	=	Tension of the reinforcement
T_a	=	Allowable tension per unit width of reinforcement
T_m, T_{max}	=	Maximum tensile force in the reinforcement per unit length along the wall.
T_{m1}	=	First component of maximum tensile force
T_{m2}	=	Second component of maximum tensile force due to inertia
T_o	=	Tensile force at the connection of the reinforcement to the facing
T_{ult}	=	Ultimate tensile strength of a geosynthetic
V	=	Sum of vertical forces on reinforced fill
w_{opt}	=	Optimum water content
W	=	Vertical force due to the weight of the fill
W'	=	Weight of surcharge
x	=	A dimension or coordinate
z, z'	=	Depth below a reference level
Z	=	Depth to reinforcement level
Z_{ave}	=	Distance from ground surface to midpoint of bar in the resisting zone

Table 8. List of notations (continued).

α	-	Maximum ground acceleration coefficient; scaling factor
α_n	-	Maximum wall acceleration coefficient at centroid for seismic loading
β	-	Slope of soil surface
γ	-	Unit weight
γ_b	-	Unit weight of backfill
γ_{dmax}	-	Maximum dry unit weight
γ_r	-	Unit weight of reinforced zone
Δ	-	Change in some parameter or quantity
ϕ	-	Angle of internal friction
ϕ_b	-	Peak angle of internal friction for drained condition of retained backfill
ϕ_f	-	Angle of internal friction of foundation soil
ϕ_r	-	Angle of internal friction of reinforced backfill
$\phi_{s,r}, \rho_{s,r}$	-	Angle of internal friction between soil and reinforcement
ϕ'	-	Effective angle of internal friction
ϕ_u	-	Angle of internal friction for undrained condition
ϵ	-	Strain
λ	-	Inclination of earth pressure resultant relative to the horizontal, when retained soil is also horizontal
λ_p	-	Inclination of earth pressure resultant relative to the horizontal, when retained soil is at slope β
μ	-	Friction coefficient along the sliding plane, which depends on the location plane, i.e., $\tan \phi_r$ or $\tan \phi_f$
θ	-	Face batter of reinforced wall section
σ_h	-	Horizontal stress
σ_p	-	Preconsolidation stress

Table 8. List of notations (continued).

σ_v	=	Vertical stress
τ	=	Shear resistance
ϵ	=	Geometric scaling factor
δ	=	Displacement; geometrical coefficient
δ_R	=	Relative displacement

Note: The following subscripts are used to denote the specific soil region:

- r, for the fill material in the reinforced soil section.
- b, for the backfill material, i.e., the fill material located between the reinforced soil section and the natural soil.
- s, for the natural soil.
- f, for the foundation soil.

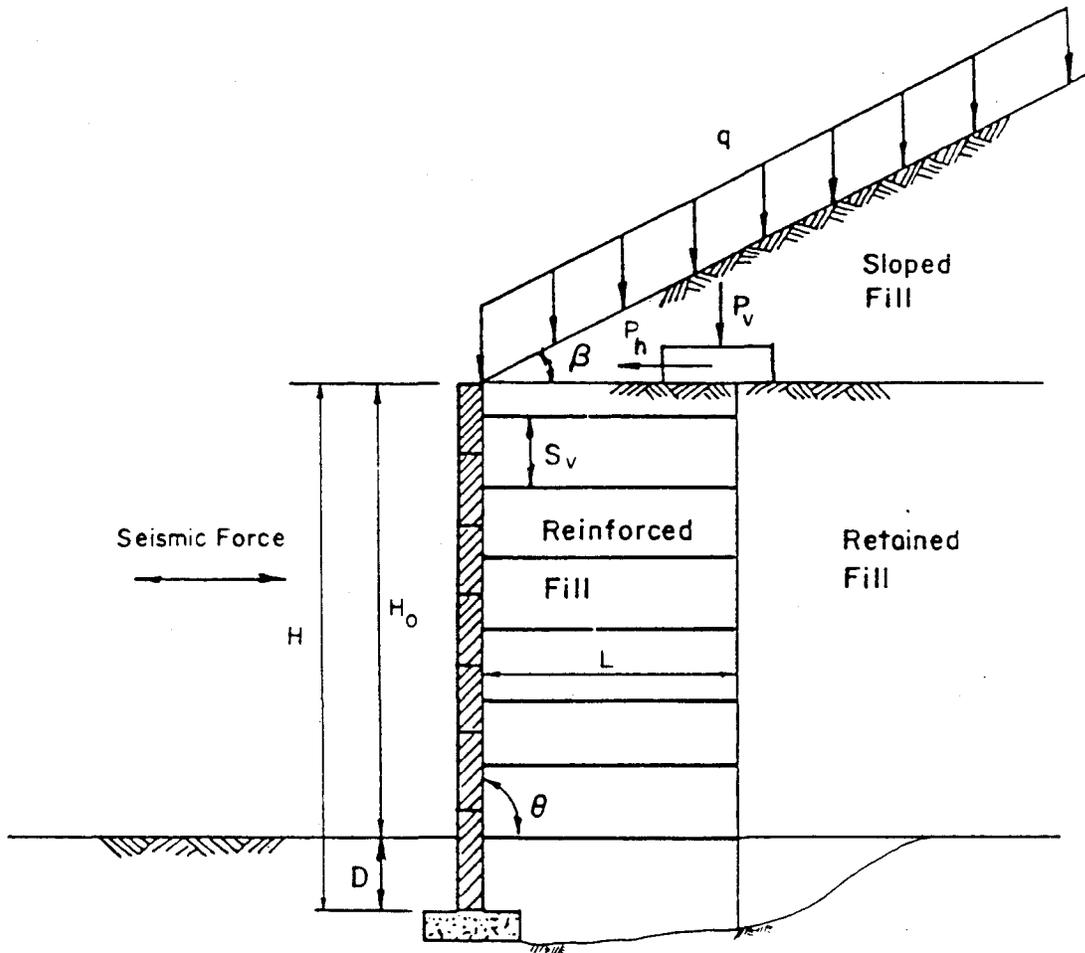


Figure 20. Geometric and loading characteristics of a reinforced soil wall.

- g. Vertical spacing of reinforcements S_v :
 - Usually controlled by facing connection locations.
 - Based on construction requirements.
 - Based on reinforcement strength.
- h. Design and service life periods.
- i. Environmental conditions such as frost action, drainage, seepage, rainfall runoff, chemical nature of backfill, scour (for seawall applications) and seepage water that will influence design requirements.

Step 2: Determine engineering properties of foundation soil (see section 2.2 for exploration and testing recommendations):

- a. The soil profile below the base of the wall; exploration depth should be at least twice the height of the wall or to refusal. Borings should be spaced at least every 100 to 150 ft (30 to 45 m) along the alignment of the wall.
- b. Strength parameters of the foundation soil (c_{uf} , ϕ_{fu} , c_f' , and ϕ_f'), unit weight (γ_f), and consolidation parameters (C_c , C_r , C_v , and σ'_p) for each foundation stratum.
- c. Location of groundwater table. Check need for drainage behind and beneath the wall.

Step 3: Determine backfill properties of both reinforced section and retained backfill (see section 2.6 for recommended fill requirements):

- a. Water content, gradation, and plasticity.
- b. Compaction characteristics, dry unit weight γ_d and optimum water content w_{opt} (95% of AASHTO T-99 is usually used) or relative density D_r .
- c. Peak angle of internal friction ϕ_r from drained direct shear tests for reinforced zone material and ϕ_b from either drained direct shear or triaxial tests for retained backfill. Note cohesion c is neglected (i.e., $c = 0$).
- d. pH, chlorides, sulfides, sulfates, and other agents that may effect aging of polymer reinforcements. (For chemical and biological characteristics of the backfill that may affect durability of the reinforcements, see section 2.5).

Step 4: Establish design factors of safety and construction criteria. Recommended minimums are listed below; local codes and specific project requirements may require greater values:

- a. External stability:
 - Sliding: F.S. ≥ 1.5 .
 - Bearing capacity: F.S. ≥ 2.0 .

- Overturning: F.S. \geq 2.0.
 - Deep seated (overall) stability: F.S. \geq 1.5.
 - Settlement: maximum allowable total and differential, based on performance requirements of the project.
 - Horizontal displacement: determine tolerable total and differential based on face batter θ and construction requirements; typically,
 - \leq 3/4 in for each 10 ft (6.2 mm per m) of wall height for precast wall facing.
 - \leq 2 in for each 10 ft (16.7 mm per m) of wall height for flexible wrapped face walls.
 - Seismic stability: F.S. \geq 75 percent of all static F.S.
- b. Internal stability
- Rupture strength: determine allowable tension in reinforcement T_a - see section 2.4.b (do not forget the reinforcement/facing connection tensile strength).
 - Pullout resistance: F.S. \geq 1.5 (minimum embedment length L_e is 3 ft [0.91 m]).
 - Durability: take into account the design life of the project in final determination of T_a .
 - Seismic stability: F.S. \geq 1.1 against pullout.

Step 5: Determine preliminary wall dimensions (see figure 20 and section 3.3):

- a. Wall embedment, depth D.
- b. Preliminary material spacing of reinforcement layers S_v .
- c. Preliminary reinforcement length L.

Step 6: Develop the lateral earth pressure diagram at the back of the wall (back of the reinforced zone) and the distribution of the vertical stress at the base (see section 3.3.d). Take surcharge loads into account (uniform and concentrated loads).

Step 7: Check external wall stability (see section 3.4):

- a. Sliding resistance.
- b. Bearing capacity.
- c. Overturning of the wall.
- d. Deep seated (overall) stability.
- e. Compound failure.
- f. Stability of excavation, if required for the wall construction.
- g. Seismic stability.

Adjust preliminary reinforcement length as necessary.

- Step 8: Estimate the settlement of the reinforced soil wall using conventional settlement analyses.
- Step 9: Calculate the maximum horizontal stress at each level of reinforcement(s) (see section 3.5.a):
- a. Determine, at each reinforcement level, the distribution of the vertical stresses due to the weight of the reinforced fill and the uniform surcharge.
 - b. Determine, at each reinforcement level, the additional vertical stress due to any concentrated surcharges.
 - c. Calculate the horizontal stresses σ_h using the appropriate K coefficient.
- Step 10: Check internal static stability and determine reinforcement requirements for each reinforcement layer (see sections 3.5.b through 3.5.e):
- a. Check safety against reinforcement rupture
 - (1) Check that $T_a \geq (\sigma_h S_v S_h)/b = \sigma_h S_v/R_c = T_{max}^{max}/R_c$, where R_c is the reinforcement coverage, b/S_h .
 - (2) For sheet reinforcements: if $T_a < T_{max}^{max}$ then reduce S_v and/or increase T_a by use of a stronger reinforcement.
 - (3) For discrete reinforcements (grids and strips): if $T_a < T_{max}^{max}/R_c$ then decrease S_v , decrease S_h , increase b and/or increase T_a by use of stronger reinforcement.
 - b. Check the strength of connections at the facing (see section 3.5.b).
 - c.
 - (1) Determine the length of reinforcement required to develop pullout resistance beyond the active zone (stability with respect to pullout).
 - (2) Check that the required reinforcement length is equal or less than the length resulting from external analysis. If it is greater, then the width of the reinforcement zone must be increased.
 - d. Review strength requirements and alternative reinforcement spacings for economical design.
- Step 11: Check internal seismic stability (see section 3.5.e):
- a. Calculate the maximum acceleration $\alpha_m g$ in the wall.
 - b. Determine the inertia force in the resisting zone.
 - c. Calculate in each reinforcement layer the dynamic force increment.
 - d. Check the stability with respect to breakage and pullout.

Step 12: Evaluate anticipated lateral displacement (see section 3.6).

Step 13: Prepare specifications (see chapter 9).

3.3 PRELIMINARY CALCULATIONS (Steps 1 to 6)

Steps 1 through 4 can be carried out on the basis of the information already given. Further discussion of steps 5 and 6 is given below.

a. Wall Embedment Depth

Minimum embedment depth D at the front of the wall (figure 20) recommended by AASHTO-AGC-ARTBA Task Force 27 is as follows:

<u>Slope in Front of Wall</u>	<u>Minimum D to Top Of Leveling Pad</u>
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 1.5 ft (0.46 m).

b. Determine Vertical Spacing Requirements

A predetermined vertical spacing of the reinforcement is required for evaluating the required reinforcement strength. The spacing requirements can be given, as would be the case in a review of specific designs provided by others, or determined from fabrication and construction requirements including type of facing, facing connection spacings, and lift thickness required for fill placement.

For preliminary feasibility evaluation, typical facing and connection arrangements for specific systems are contained in the Description of Systems section of volume II. The type of facing and location of connections should be verified with the manufacturer to make sure the available information is in line with current materials.

For wrapped faced walls with sheet type reinforcement, the vertical spacing should be a multiple of the compacted lift thickness required for the fill (typically 8 in to 12 in [20 to 30 cm]). For spacings greater than 2 ft (0.61 m), intermediate layers that extend a minimum of 3 to 4 ft (0.91 to 1.2 m) into the backfill are recommended to prevent excessive bulging of the face between the layers. For convenience, an initial uniform spacing

of 1 or 2 feet (30 or 61 cm) could be selected. Following the internal stability analysis, alternative spacings can easily be evaluated by analyzing the reinforcement strength requirements at different wall levels and modifying the spacing accordingly or by changing the strength of the reinforcement to match the spacing requirements.

c. Preliminary Reinforcement Length

Determination of the reinforcement length L is an iterative process, taking into account external stability and internal pullout resistance (steps 7, 10, 11).

For the first trial section to be analyzed, assume that the width of the reinforced soil wall and therefore the length of reinforcements, is the following:

$$L = 0.5 (H_0 + D) = 0.5 H \geq 6 \text{ ft (1.83 m)} \quad (6)$$

Traditionally the minimum length of reinforcement has been empirically limited to $0.7 H$. Current research as reviewed in volume II indicates that walls on firm foundations which meet all external stability requirements can be safely constructed using lengths as short as $0.5 H$.

d. Lateral Earth Pressures and Vertical Stresses for External Stability

As illustrated in figure 21, the lateral earth pressure at the back of the reinforced soil wall due to the retained fill increases linearly from the top.

As noted in section 3.1 and further explained in volume II, for relatively stiff, inextensibility reinforced systems, the lateral pressure (or thrust) has been found to be inclined downward, relative to the horizontal by an inclination angle λ .^(16, 17) This concept is similar to the coefficient δ used for design of conventional reinforced concrete walls to account for the inclination of the lateral pressure resulting from the relative downward movement of the soil at the back of the wall. The inclination of the lateral pressure has not been confirmed for extensible reinforcement.

For a wall with a horizontal surface, the inclination angle λ of the earth pressure relative to the horizontal is taken as:

$$\lambda = \left[1.2 - \frac{L}{H} \right] \phi_b \quad \text{when reinforcements are inextensible}^{(16)} \quad (7a)$$

$$\lambda = 0 \quad \text{when reinforcements are extensible} \quad (7b)$$

P_a , the thrust at the back of the wall, is equal to $0.5 \times K_{ab}(\lambda) \gamma H^2$ plus any influence from any surcharge loads acting on the retained backfill with:

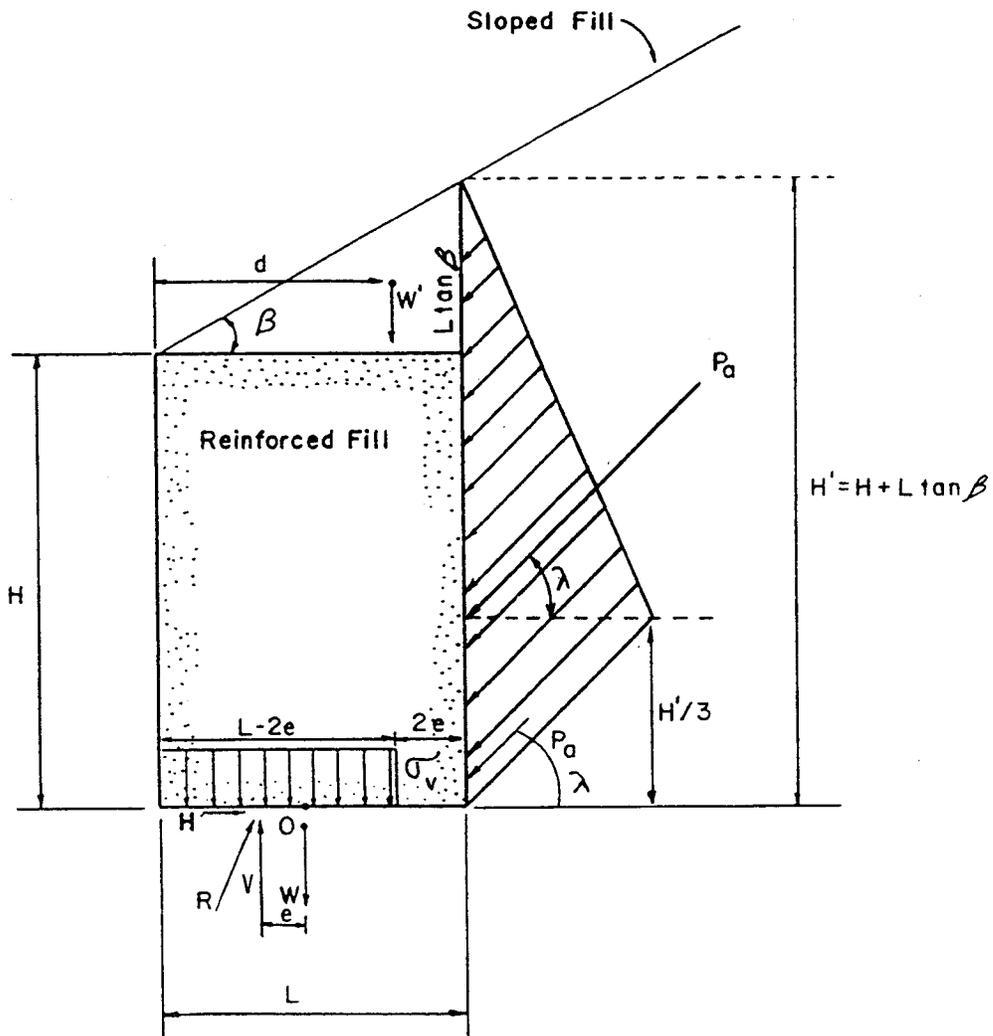


Figure 21. Calculation of vertical stress σ_v at any level (reinforced soil wall with sloping surcharge).

$$K_{a,b} = \tan^2 (45^\circ - \phi_b/2) \quad \text{if } \lambda = 0 \text{ and } \beta = 0 \quad (8)$$

$K_{a,b}$ is based on Coulomb's lateral earth pressure coefficient and can be obtained from table 9 as a function of ϕ_b and λ , if $\lambda \neq 0$

In the case of a wall retaining an infinite slope inclined at the angle β , take:

$$\lambda_\beta = \phi_b [1 - (1 - \beta/\phi_b)(L/H - 0.2)] \text{ for inextensible reinforcement} \quad (9a)$$

$$\lambda_\beta = \beta \text{ for extensible reinforcement} \quad (9b)$$

$K_{a(\lambda, \beta)}$ = active earth pressure coefficient, calculated using table 9 or the following equation:

$$K_a = \left[\frac{\sin(\theta - \phi_b) / \sin \theta}{\sqrt{\sin(\theta + \lambda)} + \sqrt{\sin(\phi_b + \lambda) \sin(\phi_b - \beta) / \sin(\theta - \beta)}} \right]^2 \quad (10)$$

Figure 21 also shows the vertical stresses at the base of the wall defined by H' . It should be noted that the weight of any wall facing is neglected in the calculations. Calculation steps are:

a. Determine λ .

b. Calculate $P_a = \frac{1}{2} K_{a(\lambda, \beta)} \gamma_b H'^2$. (11)

c. Calculate eccentricity, e , of the resulting force on the base by considering moment equilibrium of the mass of the reinforced soil section; i.e. $\Sigma M_o = 0$. Noting that V in figure 21 must equal the sum of the vertical forces on the reinforced fill, this condition yields:

$$e = \frac{P_a (\cos \lambda)(H'/3) - P_a (\sin \lambda)(L/2) - W' (d-L/2)}{\gamma_r HL + W' + P_a \sin \lambda} \quad (12)$$

d. Calculate the equivalent uniform vertical stress on the base, σ_v :

$$\sigma_v = \frac{\gamma_r HL + W' + P_a \sin \lambda}{L - 2e} \quad (13)$$

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 21. (18)

Table 9. Coefficient of lateral earth pressure K_a as a function of surface slope angle β and inclination angle λ (for vertical walls).

ϕ	λ	30°	31°	32°	33°	34°	35°	36°	37°	38°	39°	40°
		$\phi = 0^\circ$	0.333	0.320	0.307	0.295	0.283	0.271	0.259	0.246	0.238	0.227
5°	0.319	0.306	0.294	0.283	0.271	0.260	0.249	0.239	0.229	0.219	0.210	
10°	0.308	0.296	0.285	0.274	0.263	0.253	0.243	0.233	0.223	0.214	0.204	
15°	0.301	0.290	0.279	0.268	0.258	0.248	0.238	0.228	0.219	0.209	0.201	
20°	0.297	0.286	0.275	0.265	0.255	0.245	0.235	0.226	0.217	0.208	0.199	
25°	0.296	0.285	0.274	0.264	0.254	0.244	0.234	0.225	0.216	0.207	0.199	
30°	0.297	0.286	0.276	0.265	0.255	0.246	0.236	0.227	0.218	0.209	0.201	
34°	0.300	0.289	0.278	0.268	0.258	0.249	0.239	0.230	0.221	0.212	0.204	

β	λ	0°	5°	10°	15°	20°	25°	30°	34°
		$\phi = 34^\circ$	0.283	0.297	0.313	0.334	0.361	0.400	0.468
5°	0.271	0.285	0.302	0.323	0.350	0.390	0.459	0.690	
10°	0.263	0.277	0.294	0.315	0.343	0.383	0.455	0.698	
15°	0.258	0.272	0.289	0.311	0.339	0.381	0.455	0.711	
20°	0.255	0.269	0.287	0.309	0.338	0.381	0.458	0.731	
25°	0.254	0.269	0.287	0.309	0.340	0.385	0.466	0.758	
30°	0.256	0.271	0.290	0.313	0.345	0.392	0.477	0.784	
34°	0.258	0.274	0.294	0.318	0.351	0.400	0.490	0.829	

- e. Add the influence of surcharge and concentrated loads to σ_v .

3.4 EXTERNAL STABILITY

a. Sliding Along the Base

It is required that:

$$FS_{\text{sliding}} = \frac{\Sigma \text{ horizontal resisting forces}}{\Sigma \text{ horizontal sliding forces}} \geq 1.5 \quad (14)$$

where the resisting force is the lesser of the shear resistance along the base or of a weak layer near the base of the reinforced soil wall and the sliding force is the horizontal component of the thrust on the vertical plane at the back of the wall (see figures 21 and 22).

Figure 22 shows the calculation of a reinforced soil wall with extensible reinforcement, $\lambda = 0$, retaining a horizontal backfill, and supporting a uniform surcharge load. Note that any passive resistance at the toe due to embedment is ignored due to the potential for that soil to be removed through natural or manmade processes (e.g. erosion, utility installation, etc.). The shear strength of the facing system is also conservatively neglected.

Additional surcharge loads may include wheel load and traffic barrier induced sliding forces. Calculation of these forces should be based on AASHTO design code.⁽¹⁾ The calculation steps for a reinforced soil wall with a sloping surcharge are:

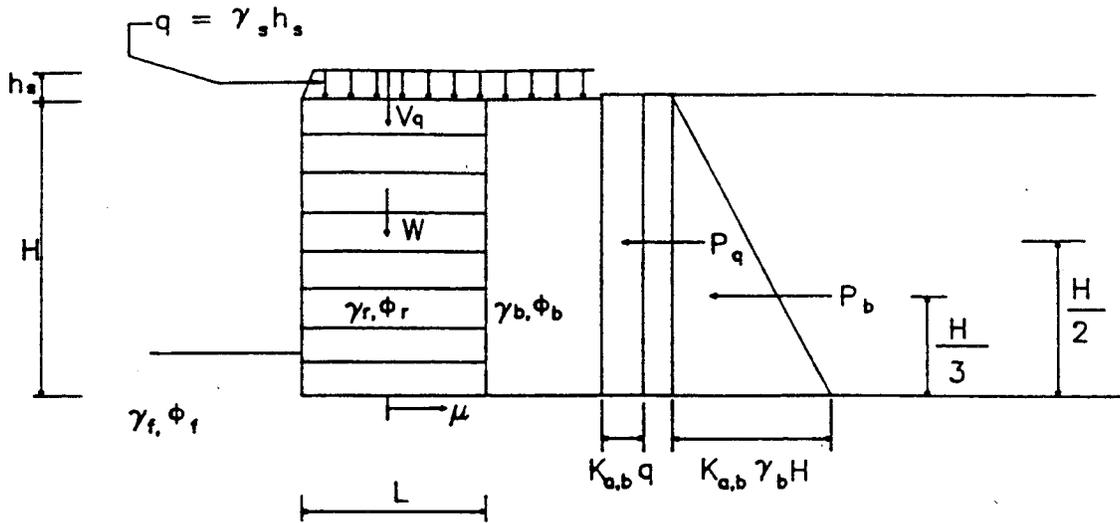
a. Calculate thrust $P_a = K_a(\phi, \lambda, \beta) \left(\frac{1}{2} \gamma_b H'^2 + q H' \right)$ (15)

where, $H' = H + L \tan \beta$ (16)

b. Calculate the sliding force: $P_b = P_a \cos \lambda_\beta$.⁽¹⁾ (17)

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

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



To Calculate FS_{SLIDING} :

$$V_q = \gamma_s h_s L$$

$$W = \gamma_r HL$$

$$P_b = 0.5K_{a,b}\gamma_b H^2$$

μ = Minimum of
 $\tan \phi_r$ or $\tan \phi_b$ or $\tan \rho$

$$K_{a,b} = \tan^2(\pi/4 - \phi_b/2)$$

$$P_q = K_{a,b}\gamma_s h_s H$$

$$FS_{\text{SLIDING}} = \frac{\Sigma \text{ Horizontal Resisting Forces}}{\Sigma \text{ Horizontal Sliding Forces}} = \frac{(V_q + W) \mu}{P_b + P_q}$$

$$= \frac{[(\gamma_s h_s + \gamma_r H) \mu] L}{K_{a,b} H (0.5\gamma_b H + \gamma_s h_s)} \geq 1.5$$

Figure 22. External sliding stability of a reinforced soil wall with extensible reinforcement and a uniform surcharge load.

2. Sliding along the reinforced backfill (ϕ_r).
3. 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 might be assumed on the basis of $F \cdot \alpha$ values used for pullout resistance determinations.

- d. Calculate the resisting force per unit length of wall:

$$P_R = (W + W' + P_a \sin \lambda_\beta) \cdot \mu \quad (18)$$

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

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

- e. Calculate the factor of safety with respect to sliding and check if it is greater than the required value.

- f. If not:

- Increase the reinforcement length, L and repeat the calculations.
- Decrease slope angle β .
- Re-evaluate required FS.

b. Overturning

Owing to the flexibility of reinforced soil structures, it is unlikely that a block overturning failure could occur. Nonetheless, an adequate factor of safety against this classical failure mode will limit excessive outward tilting and distortion of a suitably designed wall.

Overturning stability is analyzed by considering rotation of the wall about its toe. It is required that:

$$[FS]_0 = \text{resisting moments/driving moments} \geq 2.$$

The resisting moments result from the weight of the reinforced fill, the vertical component of the thrust, and the surcharge applied on the reinforced fill (dead load only). The driving moments result from the horizontal component of the thrust exerted by the retained fill on the reinforced fill and the surcharge applied on the retained fill (dead load and live load).

Figure 23 illustrates the calculation of the external overturning stability of a reinforced soil wall with extensible reinforcement, $\lambda = 0$, retaining a horizontal backfill with a uniform surcharge load. As in the case of sliding stability, the beneficial effect of embedment is neglected.

Calculation steps for a reinforced soil wall with a sloping backfill are:

- a. Calculate the driving moment of the thrust P_a , acting on the H' height (figure 21):

$$M_D = P_a (\cos \lambda_p) (H'/3) \quad (19)$$

- b. Calculate the resisting moment due to the weight M_{WR} of all the mass above the base:

$$M_{WR} = W'd + W \cdot L/2 \quad (20)$$

- c. Calculate the resisting moment due to the vertical component of the thrust:

$$M_{TR} = P_a (\sin \lambda_p) L \quad (21)$$

- d. Calculate the factor of safety with respect to overturning:

$$FS_o = \frac{W'd + W (L/2) + P_a (\sin \lambda_p) L}{P_a (\cos \lambda_p) (H'/3)} \quad (22)$$

and check that it is greater than the required value.

- e. If not, increase the reinforcement length, L .
- f. Calculate the eccentricity, e , of the resulting force at the base of the wall (section 3.3.d) and check that eccentricity does not exceed $L/6$. If $e > L/6$, increase the reinforcement length.

c. Bearing Capacity Failure

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 with respect to the ultimate bearing capacity:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS_{bc}} = \frac{q_{ult}}{2} \quad (23)$$

Calculation steps are the following for a reinforced soil wall with a sloping surcharge:

- a. Calculate the eccentricity e of the resulting force at the base of the wall (section 3.3.d).
- b. Calculate the vertical stress σ_v at the base assuming Meyerhof distribution (section 3.3.d):

$$\sigma_v = \frac{W' + W + P_a \sin \lambda}{L - 2e} \quad (24)$$

- c. Determine the ultimate bearing capacity q_{ult} using classical soil mechanics methods, e.g.:

$$q_{ult} = c_f N_c + 0.5 (L - 2e) \gamma_f N_\gamma \quad (25)$$

(N_c and N_γ are dimensionless bearing capacity coefficients and can be obtained from most soil mechanics textbooks.)

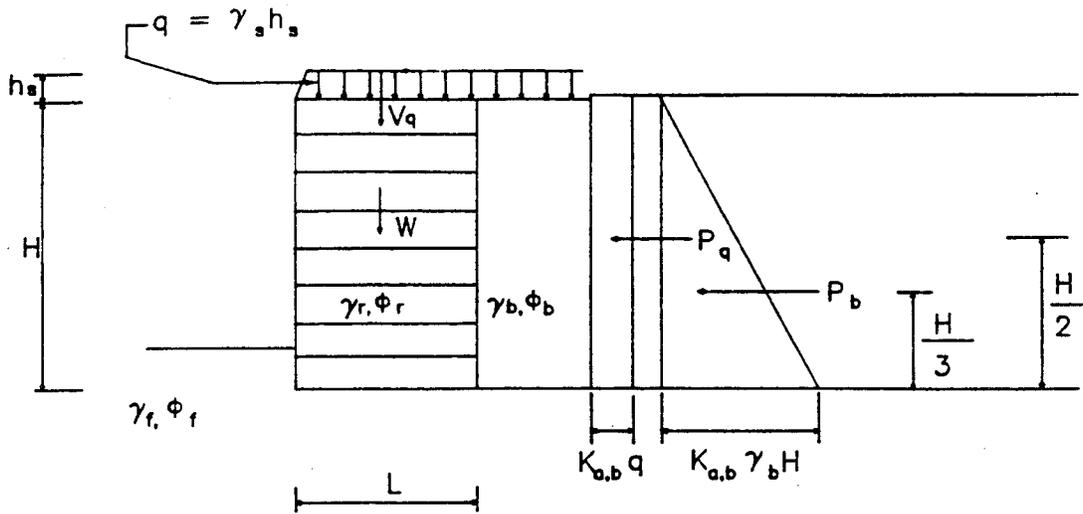
q_{ult} is reduced when the ground at the base of the wall slopes away from the structure. Methods outlined in standard texts such as NAVFAC DM-7 should be followed when such ground effects exist. Again, the beneficial effect of wall embedment is neglected.

- d. Check that: $\sigma_v \leq q_a = q_{ult}/2$.
- e. As indicated in step b and step c, 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.

Figure 24 illustrates the calculation of the bearing capacity of a wall with extensible reinforcement ($\lambda = 0$) retaining a level backfill and supporting a uniform surcharge.

d. 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 most of them. The reinforced soil wall is considered as a rigid body and only failure surfaces completely outside a



To Calculate FS _{OVERTURNING} :

$$V_q = \gamma_s h_s L$$

$$W = \gamma_r HL$$

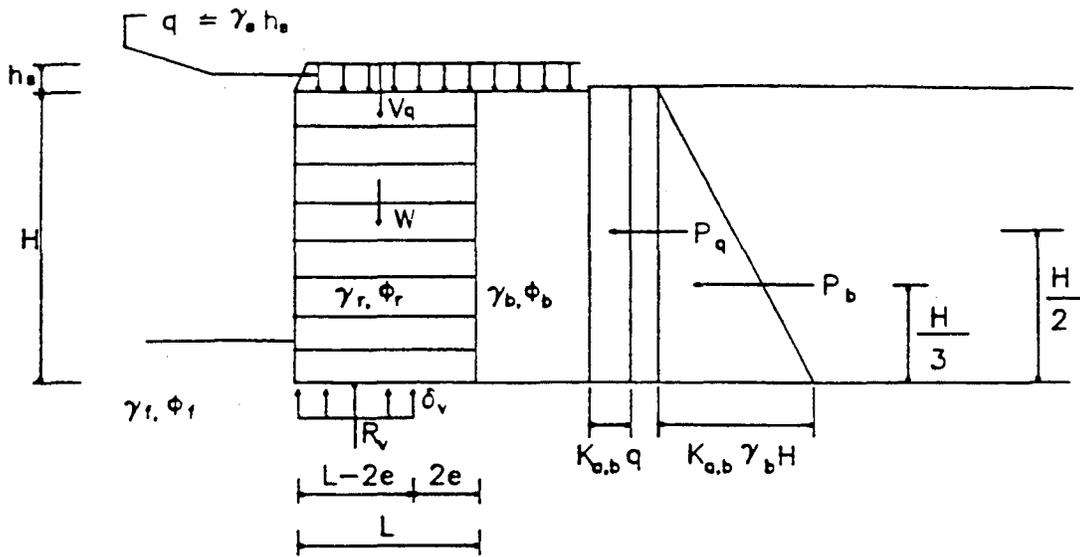
$$P_b = 0.5K_{a,b}\gamma_b H^2$$

$$K_{a,b} = \tan^2(45 - \phi_b/2)$$

$$P_q = K_{a,b}\gamma_s h_s H$$

$$\begin{aligned}
 FS_{\text{OVERTURNING}} &= \frac{\sum \text{ Moments Resisting}}{\sum \text{ Moments Overturning}} = \frac{(V_q + W)(L/2)}{P_b(H/3) + P_q(H/2)} \\
 &= \frac{3L^2(\gamma_s h_s + \gamma_r H)}{H^2 K_{a,b}(\gamma_b H + 3\gamma_s h_s)} \geq 2.0
 \end{aligned}$$

Figure 23. Extensible overturning stability of a reinforced soil wall with extensible reinforcements ($\lambda = 0$) and a uniform surcharge load.



To calculate the bearing capacity :

$$V_q = \gamma_s h_s L$$

$$W = \gamma_r HL$$

$$P_b = 0.5K_{a,b} \gamma_b H^2$$

$$P_q = K_{a,b} \gamma_s h_s H$$

$$K_{a,b} = \tan^2(45 - \phi_b/2)$$

q_o = allowable bearing capacity of the soil.

1) The eccentricity , e , of the resultant loads :

$$e = \frac{\sum \text{Driving Moments}}{\sum \text{Resisting Forces}} = \frac{P_b (H/3) + P_q (H/2)}{W + V_q}$$

$$= \frac{K_{a,b} H^2 (\gamma_b H + 3 \gamma_s h_s)}{6L (\gamma_r H + \gamma_s h_s)} \leq L/6$$

2) The magnitude of the vertical stress, $\sigma_{v \max}$:

$$\sigma_{v \max} = \frac{V_q + W}{L-2e} = \frac{\gamma_r H + \gamma_s h_s}{1-2e/L} \leq q_o$$

where : $q_o = q_{ult} / 2$

Figure 24. Bearing capacity for external stability of a reinforced soil wall with extensible reinforcement and a uniform surcharge load.

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 as discussed in section 3.8.

If the minimum safety factor is less than the required value, increase the reinforcement length or improve the foundation soil.

e. Seismic Loading

During an earthquake, the retained fill exerts a dynamic horizontal thrust, P_{AE} , on the reinforced soil wall, in addition to the static thrust. Moreover, the reinforced soil mass is subjected to a horizontal inertia force $P_{IR} = Ma_m$, where M is the mass of the reinforced wall section 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 25 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 factors.

The seismic external stability evaluation is performed according to the following steps:

- a. Select a peak horizontal ground acceleration $\alpha_o g$ based on the design earthquake.
- b. Calculate the maximum acceleration $a_m = \alpha_m g$, developed in the wall according to the formula:

$$\alpha_m = (1.45 - \alpha_o) \alpha_o \quad (26)$$

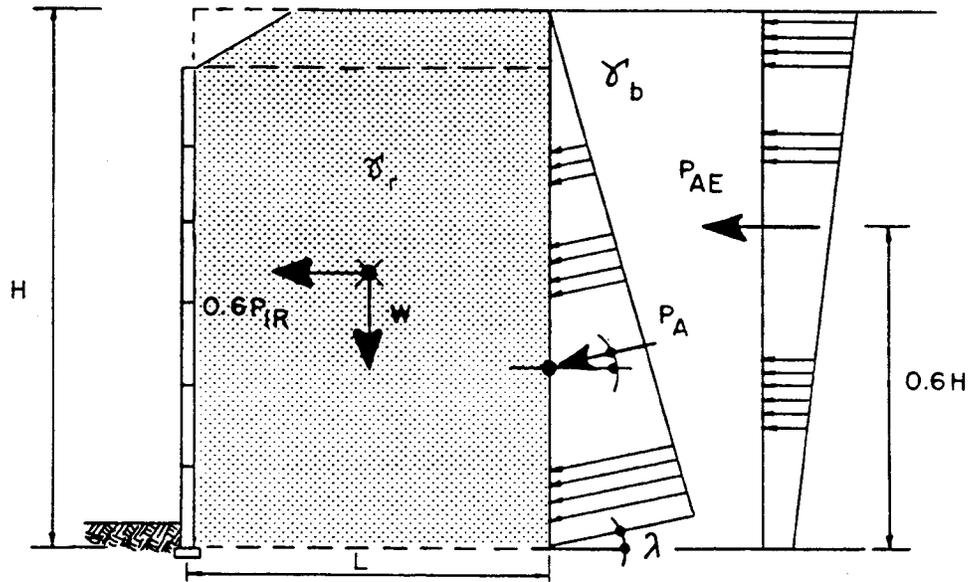
where: α_o = max. ground acceleration coefficient
 α_m = max. wall acceleration coefficient at centroid.

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

$$P_{IR} = \alpha_m \gamma_r H L$$

$$P_{AE} = 0.375 \alpha_m \gamma_b H^2$$

- d. Add to the static forces P_b and P_q acting on the structure, seismic thrust P_{AE} and 50 percent of P_{IR} respectively. The reduced P_{IR} is used since these two forces are unlikely to peak simultaneously.



$$P_{IR} = M\alpha$$

$$\approx 0.5 \alpha_m \gamma_r H^2$$

$$P_{AE} = 0.375 \alpha_m \gamma_b H^2$$

MINIMUM ALLOWABLE SAFETY FACTORS FOR SEISMIC = 75% STATIC

Figure 25. Seismic external stability of a reinforced soil wall.

- e. Evaluate sliding and overturning stability as detailed in sections 3.4.a and 3.4.b.
- f. Check that the corresponding safety factors are equal or greater than 75 percent of the static safety factors.

Relatively large earthquake shaking (i.e. $\alpha \geq 0.4$) could result in significant permanent lateral and vertical wall deformations. In seismically active areas where such strong shaking could exist, a specialist should be retained to evaluate the anticipated deformation response of the structure.

f. Settlement Estimate

Conventional settlement analyses for shallow foundations should be carried out to ensure that immediate, consolidation, and secondary settlement of the wall are less than the performance requirements of the project. Both total and differential settlements should be considered. If foundation settlement is excessive, then the design must include improvement of the foundation soils.

3.5 INTERNAL LOCAL STABILITY

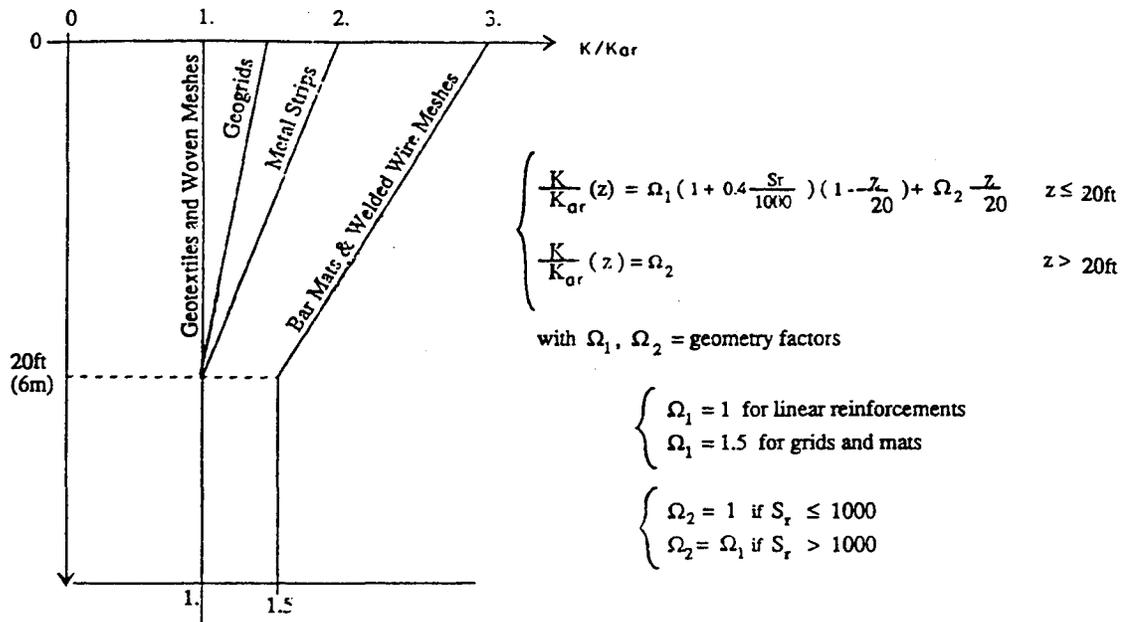
a. Calculation of Maximum Tensile Forces in the Reinforcement Layers

The first step in checking internal stability is to calculate the maximum tensile forces T_{max} developed along the potential failure line in the reinforcements. The method of calculation and the evaluation parameters are traditionally the main variations between different system design methods. As discussed in volume II, the research performed for this study indicates that the maximum tensile force is primarily related to the stiffness of the reinforced soil mass which is controlled by the extensibility and density of reinforcement. Based on the research, a conservative relationship between the global reinforced soil stiffness and S_r , the geometry of the reinforcements and the horizontal stress has been developed as shown in figure 26. The influence of the geometry can be adequately taken into account by the factors Ω_1 and Ω_2 . The resulting K/K_r ratio decreases from the top to a constant value below 20 ft (6m).

The figure was prepared by back analysis of the lateral stress ratio K from available fill data. The lines shown on the figure correspond to usual values representative of the specific reinforcement systems which are known to give satisfactory results. This provides a simplified evaluation method for all cohesionless reinforced fill walls. Future data will most likely lead to modifications in figure 26, including narrower ranges of stiffness values for specific conditions.

Calculation steps are the following:

- a. Calculate at each reinforcement level the horizontal stresses σ_h along the potential failure line discussed in section 3.1.b from the weight of the retained fill $\gamma_r Z$ plus, if present, uniform surcharge loads q concentrated surcharge loads $\Delta\sigma_v$ and $\Delta\sigma_h$.



where: K_{ar} = active lateral earth pressure coefficient
= $\tan^2 (45 + \phi_r/2)$ for horizontal surface

$$= \cos \beta \left\{ \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi_r}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi_r}} \right\} \text{ for sloped surface at angle } \beta$$

S_r = global reinforcement stiffness factor in units of F/L^2

$$S_r = \frac{EA'}{(H/n)} \text{ for inextensible reinforcement}$$

E = modulus of reinforcement in units of F/L^2
 A' = average area of the reinforcement per unit width of wall

$$= \frac{b \times t}{S_h} = R_c t \text{ for strip reinforcement (see figure 15)}$$

$$= \frac{A_c}{S_b} = \frac{A_c}{b} R_c \text{ for bar mat and steel grids (see figure 15)}$$

H/n = average vertical spacing based on the number of layers n over height H

$$S_r = \frac{J \cdot R_c}{(H/n)} \text{ for geosynthetic reinforcement}$$

J = modulus of geosynthetic in units of F/L usually determined from wide width test (ASTM D-4595) as secant modulus at 5% strain

$$= (T \text{ at } 5\% \epsilon) / 0.05$$

Figure 26. Variation of the stress ratio K with depth in an inextensibly reinforced soil wall

$$\sigma_h = K (\gamma_r Z + q + \Delta\sigma_v) + \Delta\sigma_h \quad (29)$$

where: $K = K(z)$ based on ϕ , as shown in figure 26.

K is based on the stiffness of the reinforced section which is defined by the stiffness factor S_R . For preliminary calculations when the reinforcement type is unknown, $S_R = 1,500$ k/ft/ft can be assumed for inextensible reinforcement and $S_R = 50$ k/ft/ft can be assumed for extensible reinforcement. The final design should always be checked based on the stiffness factor for the actual reinforcement and spacing to be used.

$\Delta\sigma_v$ is the increment of vertical stress due to concentrated vertical loads using a 2V:1H pyramidal distribution as shown in figure 27a.

$\Delta\sigma_h$ is the increment of horizontal stress due to horizontal concentrated surcharges, if any, and calculated as shown in figure 27b. Dynamic loads for traffic barriers should be included based on AASHTO specification. (f)

For sloping soil surfaces above the reinforced soil wall section, either the actual surcharge can be replaced by a uniform surcharge σ_v equal to $0.5 \gamma h_s$ where h_s is the height of the slope at the back of the wall or by calculation of K based on the slope angle β with the least conservative influence normally selected.

- b. Calculate the maximum tension T_{max} per unit length along the wall in each reinforcement layer:

$$T_{max} = S_v \cdot \sigma_h \quad (30)$$

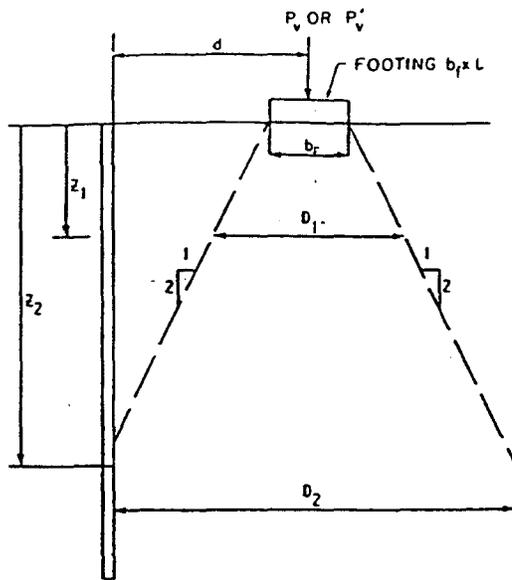
Calculation of T_{max} allows the determination of reinforcement size^{max} at each number n of discrete reinforcements (metal strips, bar mats, geogrids, etc.) per unit width of wall face or the tensile capacity required of sheet type reinforcement (welded wire mesh, geosynthetic) to be used (see next section).

b. Internal Stability with Respect to Breakage

Stability with respect to breakage of the reinforcements requires that:

$$T_{max} \leq T_a R_c \quad (31)$$

where R_c is the coverage ratio b/S_H , b is 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 (section 2.4):



For strip load $\Delta \sigma_v = \frac{P_v}{D}$

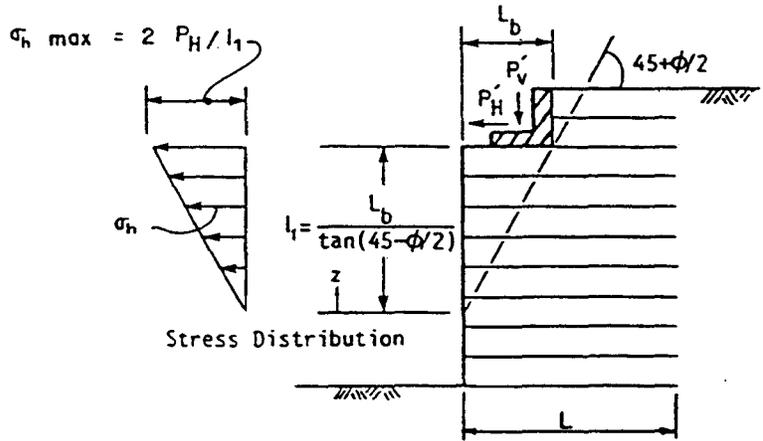
For isolated footing load $\Delta \sigma_v = \frac{P'_v}{D(L+Z)}$

For point load $\Delta \sigma_v = \frac{P'_v}{D \times D}$

$D_1 = b_f + \frac{2Z_1}{2} = b_f + Z_1$
 $D_2 = b_f + \frac{2Z_2}{2} + d$

where: D = effective width of applied load with depth.
 l = length of footing

a. Distribution of Stress From Concentrated Vertical Load P_v .



b. Distribution of Stress From Concentrated Horizontal Load P_H .

where: L_b = the effective width of the applied load at the point of application.

Figure 27. Schematic illustration of concentrated load dispersal.

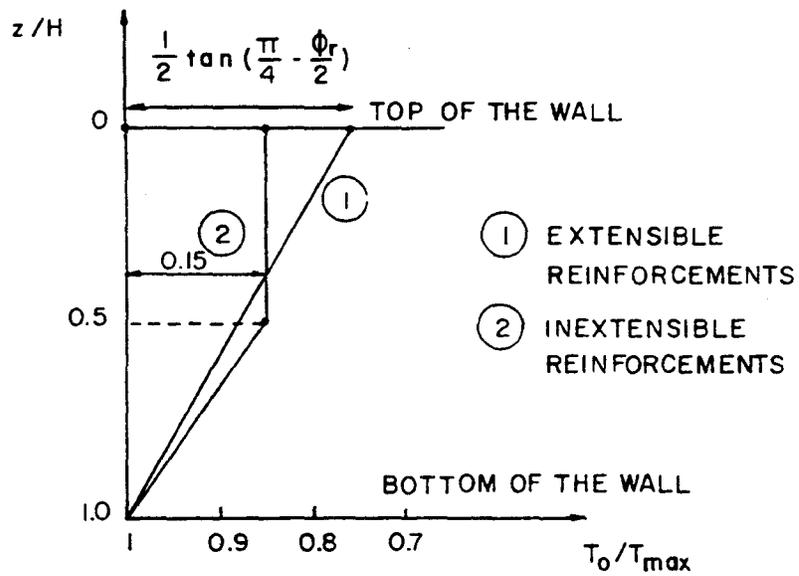


Figure 28. Determination of the tensile force T_0 in the reinforcement at the connection with the facing (inextensible and extensible reinforcements).

At the connection of the reinforcements with the facing, check that tensile force T , determined as indicated in figure 28 is not greater than the allowable tensile strength of the connection. The connection strength will depend on the structural characteristics of the facing system used.

c. Internal Stability with Respect to Pullout Failure

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

$$T_{max} \leq \frac{1}{FS_{po}} P_r R_c \quad (32)$$

$$T_{max} \leq \frac{1}{FS_{po}} (F^* \alpha \gamma_r z' L_e C) R_c \quad (33)$$

where:

- FS_{po} = safety factor against pullout = 1.5
- P_r = available pullout resistance for a particular type of reinforcement (see section 2.8)
- C = 2 for strip, grid, and sheet type reinforcement and π for circular bar reinforcements
- F^* = the pullout resistance factor (section 2.8, table 7)
- α = scale effect correction factor (section 2.8, table 7)
- $\gamma_r z'$ = the overburden pressure, including distributed surcharges
- 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, as described in section 3.7.b.

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

$$L_e \geq \frac{1.5 T_{max}}{C F^* \alpha \gamma_r z' R_c} \quad (34)$$

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.

In the case of a reinforced soil wall with a sloping surcharge, the overburden pressure ($\gamma_r z'$) varies with the distance from the face, and the maximum pullout resistance, P_r , has to be calculated according to:

$$P_r = \int_{L-L_0}^L [CF' \gamma_r z' (x) dx] \quad (35)$$

Solution of this equation gives:

$$P_r = CF' \gamma_r Z_{av} L \quad (36)$$

in which Z_{av} is the distance from the ground surface to the midpoint of the bar in the resisting zone.

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

$$L_t = L_a + L_0 \quad (37)$$

where: L_a is obtained from figure 18 for simple structures not supporting concentrated external loads such as bridge abutments.

For the total height of a reinforced soil wall with extensible reinforcement.

$$L_a = (H - Z) \tan (45 - \phi'_r/2) \quad (38)$$

where: Z is the depth to the reinforcement level

For a wall with inextensible reinforcement from the base up to $H/2$:

$$L_a = 0.6 (H-Z) \quad (39)$$

For the upper half of a wall with inextensible reinforcements:

$$L_a = 0.3H \quad (40)$$

See section 3.8 for determination of L_a for complex structures.

Usually, for construction ease, the final length is chosen as uniform based on the maximum length requirements. 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 as discussed further in section 3.7.e.

The majority of reinforced soil walls constructed to date have used $0.7H$ as a minimum reinforcement length requirement. Research including monitoring of structures has indicated that shorter lengths can be used provided internal and external stability requirements have been satisfied. However, it is important, especially when using lengths less than $0.7H$, that a thorough evaluation of the fill, backfill, and foundation properties be

performed prior to acceptance of the design. It should also be noted that lateral wall displacement increases with decreasing length as discussed in section 3.6. Final lengths should be carefully selected based on performance requirements.

d. Strength and Spacing Variations

This section provides guidance in considering variations in the reinforcement strength and/or spacing over the height of the wall for economical reasons or space limitations. However, increasing the complexity of the geometry will also increase the required attention to detail during construction and careful inspection is required to avoid construction mistakes.

Use of a constant reinforcement density 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.

There are generally two practical ways to do this:

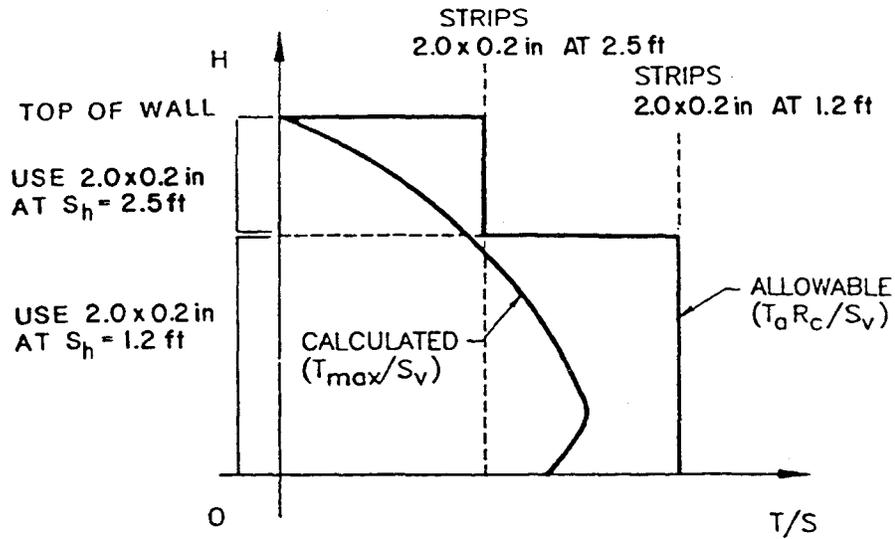
- In the case of reinforcements consisting of strips, grids, or mats, which are used with precast concrete facing panels, the vertical spacing is maintained constant and the reinforcement density is increased with depth by increasing the number and/or the size of the reinforcements.

For instance, in Reinforced Earth walls, the horizontal spacing of the 2 in x 0.2 in (50 mm x 4 mm) strips is usually 2.5 ft (0.75 m), although the horizontal reinforcement spacing can be decreased as shown in figure 29a by using special facing panels for the lower levels in high walls.

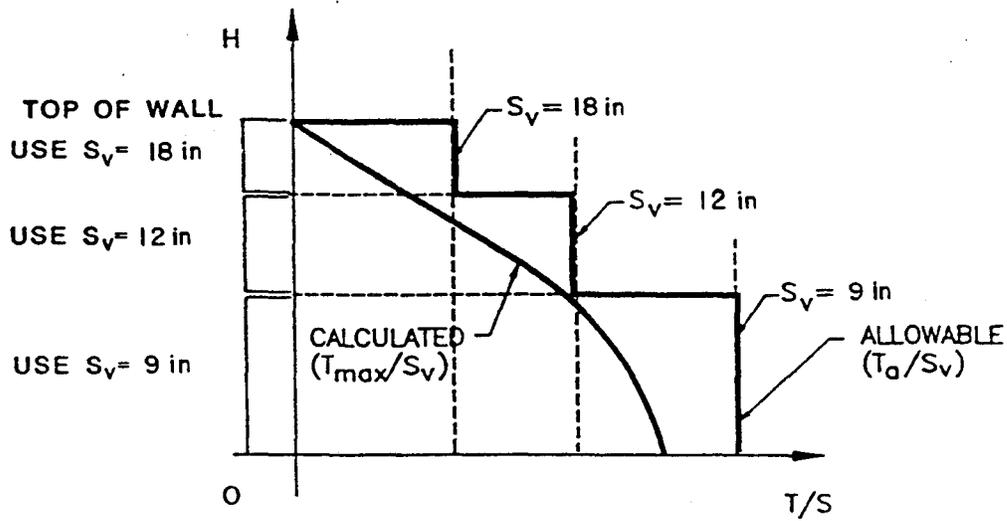
- In the case of planar reinforcements, generally made of geotextiles or geogrids, the most common way of varying the reinforcement density T_v/S_v is to change the vertical spacing S_v , especially if wrapped facing is used, because it easily accommodates spacing variations. The range of acceptable spacings is governed by consideration of placement and compaction of the backfill for the minimum value ($S_m \sim 6$ in [15 cm]) and by local stability during construction for the maximum value ($S_M \sim 2.0$ ft [61 cm]).

As indicated on figure 29, the allowable reinforcement density T_v/S_v is increased with depth. At any level it must be equal or greater than the required reinforcement density.

The spacing plots in figure 29, provide a simple method to visualize the effects of changing reinforcement density. The vertical axis represents elevation within the wall and the horizontal axis can be thought of as horizontal stress to be restrained by the reinforcement. Vertical lines on the plot represent maximum horizontal stresses permitted to be carried by a specific reinforcement density. Reinforcement density is a function of cross-sectional area of reinforcing elements,



A) LINEAR REINFORCEMENT DENSITY VERSUS HEIGHT:
INEXTENSIBLE REINFORCEMENT



B) PLANAR REINFORCEMENT DENSITY VERSUS HEIGHT:
EXTENSIBLE REINFORCEMENT

Figure 29. Examples of determination of equal reinforcement density zones.

allowable reinforcement material stress, and horizontal and vertical reinforcement spacing.

In each case, the reinforcement density is calculated as indicated in section 3.5 and plotted versus the height. Then, the discrete values of the allowable reinforcement density corresponding either to strength increments (linear reinforcements) or to vertical spacing increments (planar reinforcements) are calculated, and finally the heights separating zones of equal reinforcement density are determined.

e. Internal Stability with Respect to Seismic Loading

As indicated in section 3.4.e, a seismic loading induces an internal inertia force P_{IR} acting horizontally on the active zone in addition to the existing static forces.

This force will lead to incremental dynamic increase 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 reinforcement rupture and considered acceptable relative to pullout resistance. Calculation steps for internal stability analyses with respect to seismic loading are as follows (figure 30 for inextensible reinforcement and figure 31 for extensible reinforcement).

- a. Calculate the maximum acceleration $\alpha_m g$ in the wall and the force P_{IA} acting on the reinforced soil mass above level z:

$$P_{IA} = \alpha_m W_A \quad (41)$$

$$\alpha_m = (1.45 - \alpha)\alpha \quad (42)$$

where: M_A is the mass of the active zone above level z

- b. Calculate the total horizontal static stress in the reinforced fill and consequently the first component T_{m1} of the maximum tensile force (figures 30 or 31) as follows:

- Calculate horizontal stress σ_h using K coefficient (values given in section 3.5.a)

$$\sigma_h = K\sigma_v = K(\gamma_r H + q + \Delta\sigma_v) \quad (29)$$

- Calculate the maximum tensile force component T_{m1} in each reinforcement:

$$T_{m1} = S_v \sigma_h \quad (43)$$

- c. Calculate the dynamic increment T_{m2} directly induced by the inertia force P_{IA} in the reinforcements.

This is done by distributing P_{IA} in the different reinforcements proportionally to their "resistant area"

(1) DETERMINE σ_v

$$(2) \sigma_h = K \sigma_v = K(\gamma_r + Z + q + \Delta\sigma_v) + \Delta\sigma_h$$

$$(3) T_{m1} = \frac{K \delta H S_v}{n} \quad (\text{n STRIPS PER UNIT WIDTH OF WALL FACE})$$

(4) DETERMINE DYNAMIC INCREMENT

$$T_{m2} = P_I \sum \frac{n b \cdot L_e}{n_i \cdot b_i \cdot L_{ei}}$$

$$T_m = T_{m1} + T_{m2}$$

$$T_o = 85\% \text{ TO } 100\% T_m$$

ACCORDING TO FIGURE 28

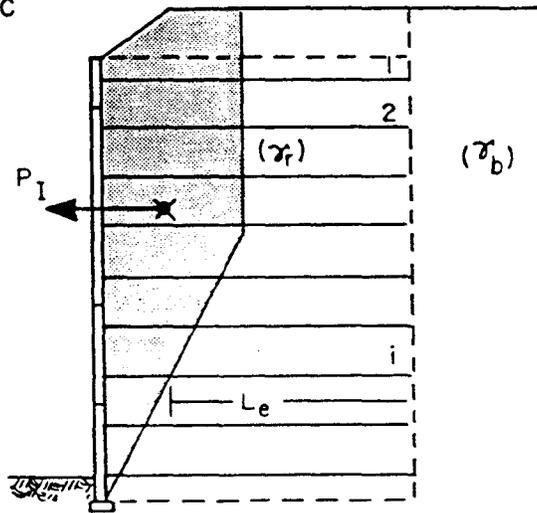


Figure 30. Internal seismic stability of a reinforced soil wall (inextensible reinforcement).

MAXIMUM ACCELERATION
 IN THE WALL $\alpha_m = (1.45 - \alpha_0) \cdot \alpha_0$

1st STEP : DETERMINE T_{m1}

$$T_{m1} = \frac{K \cdot W \cdot S_v}{n} \text{ per unit width of wall face}$$

2nd STEP : DYNAMIC INCREMENT

$$T_{m2} = P_{IA} \frac{(R_c \cdot L_e)_i}{\sum (R_c \cdot L_e)_i}$$

$$T = T_{m1} + T_{m2}$$

$T_o = 85\% \text{ to } 100\% T_m$
 according to figure 28

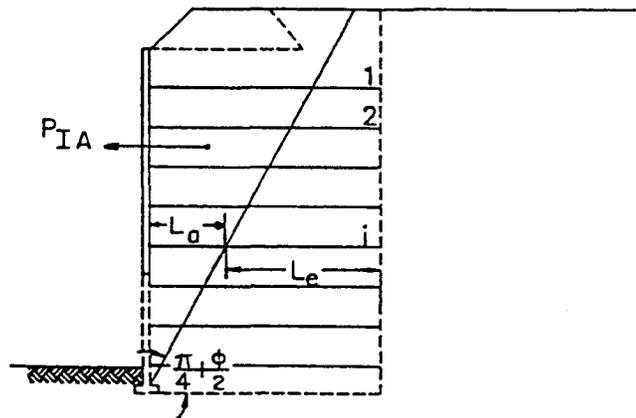


Figure 31. Internal seismic stability of a reinforced soil wall (extensible reinforcement).

$L_e \cdot b$ (L_e : embedment length of reinforcement in the resistant zone; b : reinforcement width). This leads to:

$$T_{n2} = P_r \frac{(R_c \cdot L_e)_i}{\sum (R_c \cdot L_e)_i} \quad (44)$$

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

d. Use the total maximum tensile force:

$$T_{max} = T_{n1} + T_{n2} \quad (45)$$

for checking the stability with respect to breakage and to pullout of the reinforcement, according to sections 3.5.b and 3.5.c, but with seismic safety factors of only 75 percent of the minimum allowable static safety factors values.

This leads to:

$$\text{Breakage failure: } T_{max} \leq \frac{1}{0.75} \cdot T_a \cdot R_c$$

$$\text{Pullout failure: } T_{max} \leq \frac{P_r R_c}{0.75 FS_{p0}} \quad (47)$$

$$T_{max} \leq \frac{2 \cdot F_d^* \alpha}{0.75 \times 1.5} \cdot \gamma z' \cdot L_e R_c \quad (48)$$

where: F_d^* (dynamic) = $0.8 F^*$ (static)
 α is the reinforcement scale effect correction factor from table 7

The recommended design method with respect to seismic loading was developed for inextensible 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. In addition, since there is a substantial factor of safety in the design tension for potential creep of extensible reinforcement under long-term static loads. An additional factor of safety against a dynamic overload is provided. Therefore, the inextensible reinforcement analysis should be safe for extensible reinforcement.

3.6 LATERAL WALL DISPLACEMENT EVALUATION

There is no standard method to evaluate the overall lateral displacement of reinforced soil walls. Loading of the reinforced soil section and associated lateral deformation will primarily occur during construction with the exception of post construction surcharge loads. Post construction movement could also occur due to settlement of the structure.

The major factors influencing lateral displacements during construction include compaction intensity, reinforcement to soil stiffness ratio (i.e., the area of reinforcement and deformability as compared to the modulus and area of the reinforced soil section), reinforcement length, slack in reinforcement-to-facing connections, and deformability of the facing system.

The total lateral displacement of simple structures on firm foundations that is anticipated during construction can be estimated from figure 32, based on the length of reinforcement L to height of the wall H ratio and the extensibility of the reinforcement. This figure was empirically developed using data from actual structures and computer simulation models. It provides a first order lateral deformation estimate that could be used to establish appropriate face batter and to evaluate anticipated horizontal alignments.

It should be noted that as L/H decreases, the lateral deformation increases. This is important when determining the suitability of the final reinforcement length. For example, going from a length of 0.7H to 0.5H could essentially double the lateral deformation anticipated during construction.

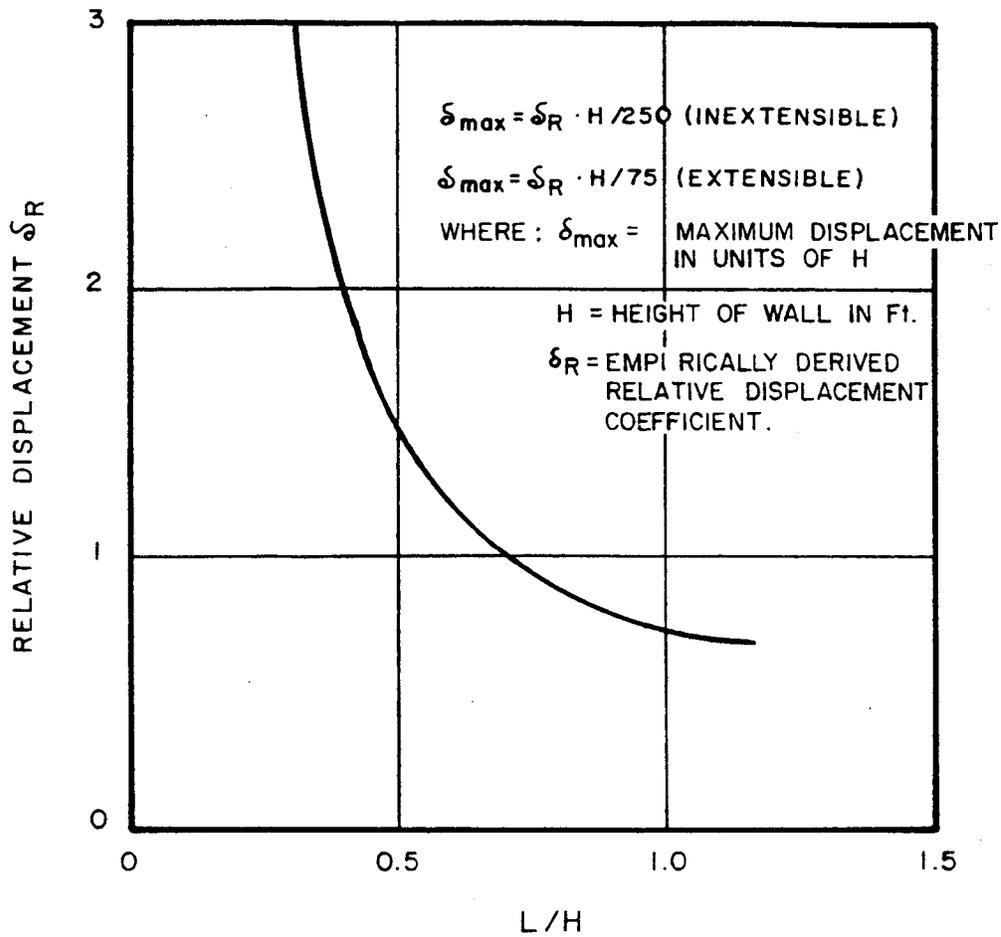
For critical structures requiring precise tolerances, such as bridge abutments, the lateral displacement of the wall has to be calculated more accurately, taking into account the tilting due to the thrust at the back of the wall. A finite element method of calculation is recommended for this analysis.

3.7 DESIGN EXAMPLE

Given a reinforced soil wall design as shown in figure 33, check the reinforcement requirements. Assume that this represents the most critical wall section. The reinforced system consists of standard Reinforced Earth Company panels, with four ribbed galvanized strips per panel. The horizontal spacing between strips is 2.46 ft (0.75 m). The surface area of each panel is 24.2 ft². Strips are 1.969 in wide (50 mm) by 0.157 in (4 mm) thick. Galvanization = 3.4 mils/side (86 μm/side).

Note: Dimensions and properties of the reinforced system used in this example are typical; the user of this manual should always check the actual material properties to be used in the reinforcing system.

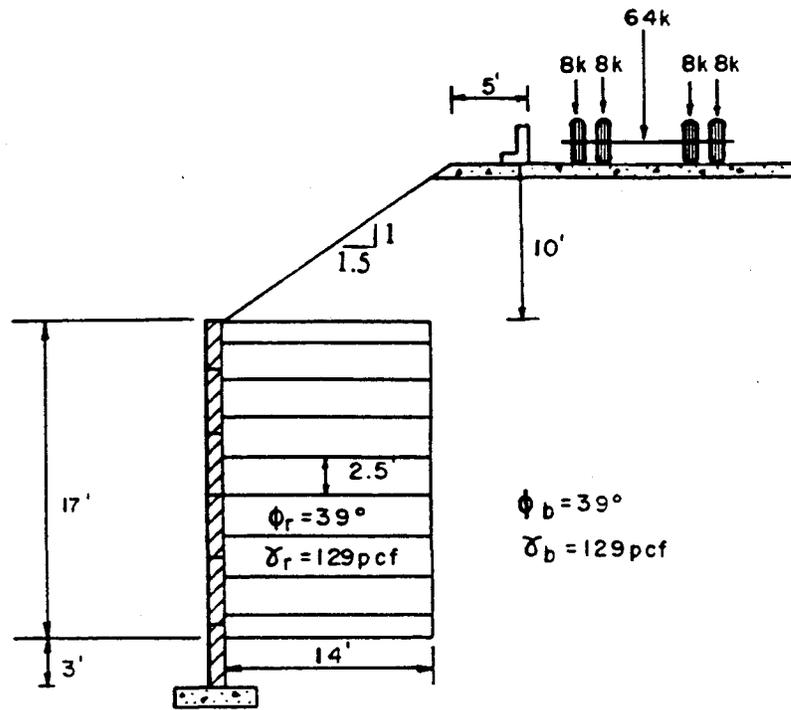
Step 1: Establish design limits, scope of the project, and external loads:



NOTE: INCREASE RELATIVE DISPLACEMENT 25% FOR EVERY 400 PSF OF SURCHARGE.

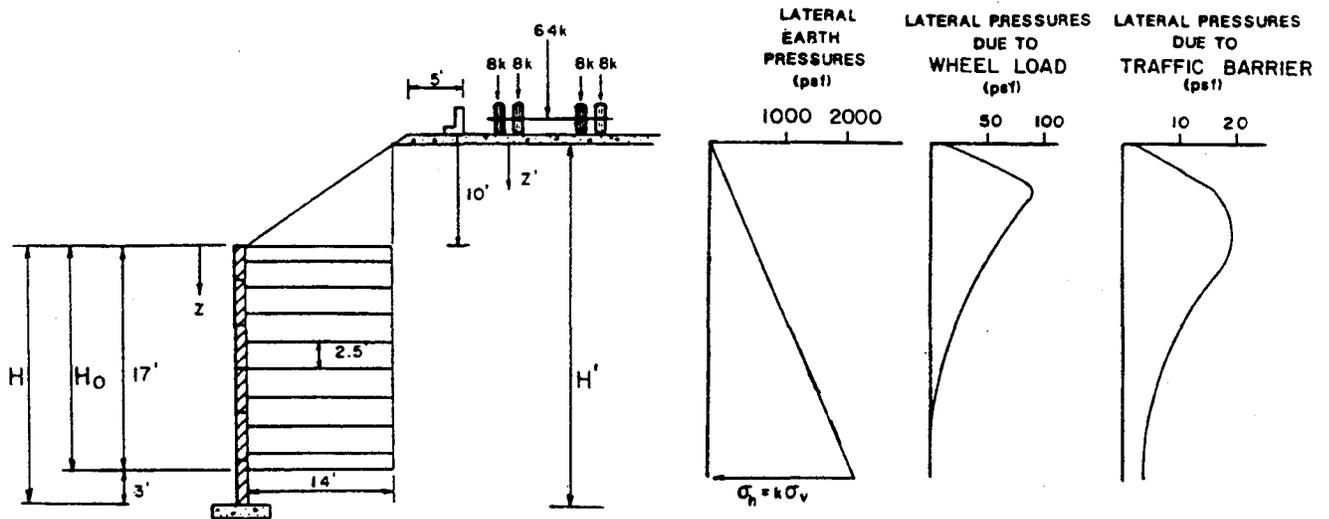
Based on 20 foot high walls, relative displacement increase approximately 25% for every 400 psf of surcharge. Experience indicates that for higher walls, the surcharge effect may be greater.

Figure 32. Empirical curve for estimating anticipated lateral displacement during construction for reinforced fill structures.



$\phi'_f = 38^\circ$
 $\gamma'_f = 125 \text{ pcf}$

a) Given conditions



b) Lateral stress at the back of the reinforced fill section

Figure 33. Design example.

- a. Reinforced wall height $H = 20$ ft.
- b. External wall height, $H_o = 17$ ft.
- c. Wall face batter, $\theta = 90^\circ$.
- d. Slope angle of soil surface $\beta = 33.7^\circ$.
- e. External loads and their locations:
 - . Traffic barrier = 607 lb/ft length.
 - . Traffic load = 8k/wheel.
 - . Impact load from traffic barrier = 2,000 lb/ft
- f. Seismic loading: $\alpha_o = 0.1$
- g. Type of facing: concrete panels.
- h. Vertical spacing of reinforcement $S_v = 2.5$ ft.
- i. Width of reinforced soil wall = 14 ft.

Step 2: Determine engineering properties of foundation soil. The foundation soils are assumed to have the following engineering properties:

$$c_u = 2 \text{ tsf}, c' = 0, \phi' = 38^\circ, \gamma = 125 \text{ pcf.}$$

Groundwater table is located 6 feet below existing ground surface.

Step 3: Determine backfill properties on both reinforced section and retained backfill.

The backfill material on both reinforced section and retained backfill has the following properties

$$\gamma = 129 \text{ pcf}, \phi' = 39^\circ, c' = 0.$$

Coefficient of Uniformity, $C_u = 10$.

Step 4: Establish design factor of safety and performance criteria.

- a. External stability:
 - . Sliding FS = 1.5.
 - . Bearing capacity FS = 2.0.
 - . Overturning FS = 2.0.
 - . Deep seated (overall) stability: FS = 1.5.
 - . Vertical settlement $\leq 3/4$ in.
- b. Internal stability:
 - . Rupture strength.

For inextensible reinforcement, the allowable tensile force per unit width of reinforcement is:

$$T_a = \frac{\sigma_a A_c}{b} = \frac{0.55 \sigma_y A_c}{b}$$

$$\sigma_y = 65 \text{ ksi}$$

A_c = cross sectional area of the steel minus corrosion losses.

b = strip width = 2 in

Corrosion losses: Corrosion rates for mildly corrosive backfill (See section 2.4 and 2.5).

For zinc: 15 $\mu\text{m}/\text{year}$ (first 2 years)
4 $\mu\text{m}/\text{year}$ (thereafter).

For carbon steel: 12 $\mu\text{m}/\text{year}$ (thereafter).

Service life for zinc:

For first 2 years: 2 years (15 $\mu\text{m}/\text{year}$) = 30 μm .
Zinc thickness after 2 years (per side):
86-30 = 56 μm .

Remaining life after 2 years:
(56 μm)/(4 $\mu\text{m}/\text{year}$) = 14 years.

Total life of zinc: 14+2 = 16 years.

Thickness of reinforcement after 100 years:

$$t_{100} = [4,000 \mu\text{m}] - 2[(12 \mu\text{m}/\text{year}) \times (100-16)\text{year}] = 1,984 \mu\text{m} = 0.078 \text{ in}$$

Cross sectional area after 100 years:

$$A_c = 50\text{mm} \times 1.98\text{mm} = 99.0 \text{ mm}^2 = 0.1535 \text{ in}^2$$

$$T_a = \frac{(0.55)(65,000)(0.1535)(12)}{1.969} = 33,444 \text{ lb/ft}$$

• Pullout resistance: FS = 1.5 for granular soils with a 3 ft minimum embedment length.

Step 5: Wall embedment, spacing of reinforcement layers S_v , and reinforcement length are given for this project.

Step 6: Develop the lateral earth pressure diagram at the back of the wall and the distribution of the vertical stresses at the base. Take surcharge loads into account.

a. Determine λ

For inextensible reinforcement,

$$\lambda_p = \phi[1 - (1 - \beta/\phi)(L/H - 0.2)] = 36.3$$

$K_\lambda = 0.402$ using the formula shown in section 3.3d for K_λ (it is a function of ϕ , λ , and β).

b. Calculate P_a

$$P_a = 1/2 K_\lambda \gamma_b (H')^2$$

$$H' = H + L \tan \beta = 20 + 14(\tan 33.7^\circ) = 29.3 \text{ ft}$$

$$P_a = 1/2 (0.402)(129)(29.3)^2 = 22.3 \text{ k}$$

c. Calculate eccentricity of the resulting force on the base as follows:

$$W' = \frac{(L \tan \beta)(L) \gamma}{2} = \frac{(14)(\tan 33.7^\circ)(14)(129)}{2} = 8.4 \text{ k}$$

$$e = \frac{P_a (\cos \lambda)(H'/3) - P_a (\sin \lambda)(L/2) - W'(d - L/2)}{\gamma_r HL + W' + P_a (\sin \lambda)} = 1.1 \text{ ft}$$

d. Calculate the equivalent uniform vertical stress on the base, σ_v :

$$\sigma_v = \frac{\gamma_r HL + W' + P_a \sin \lambda}{L - 2e} = 4.9 \text{ ksf}$$

e. The lateral load resulting from the traffic barrier and traffic load were also determined. These have been plotted in figure 33.

Step 7: Check external wall stability:

a. Sliding along the base

$$\begin{aligned} FS_{\text{sliding}} &= \frac{\Sigma \text{ horizontal resisting forces}}{\Sigma \text{ horizontal sliding forces}} > 1.5 \\ &= \frac{(W + W' + P_a \sin \lambda) \cdot \mu}{P_a \cos \lambda + 1.32} = 2.3 \text{ OK} \end{aligned}$$

Σ horizontal sliding forces includes the forces due to the traffic barrier (0.33) and wheel load (0.99) = 1.32k. See references 1, 25, 27.

where: $W = (L)(H)(\gamma) = (14)(20)(129) = 36.1 \text{ k}$

$$\mu = \tan 38^\circ = 0.78$$

b. Overturning:

$$FS = \frac{\sum \text{Resisting Moments}}{\sum \text{Driving Moments}} \geq 2$$

(Note: Moments about the toe)

- 1) Calculate the driving moment of the thrust, P_a , acting on the H' height.

$$M_D = P_a (\cos \lambda) (H'/3) = 175.5 \text{ k}\cdot\text{ft.}$$

- 2) Calculate the resisting moment due to the weight M_{WR} of all the mass above the base

$$M_{WR} = W'd + W(L/2) = 331.1 \text{ k}\cdot\text{ft.}$$

- 3) Calculate the resisting moment due to the vertical component of the thrust.

$$M_{TR} = P_a (\sin \lambda)(L) = 184.8 \text{ k}\cdot\text{ft.}$$

- 4) The driving moment due to the traffic barrier and wheel load = 26.7 k·ft.⁽¹⁾

- 5) Calculate the factor of safety with respect to overturning.

$$FS_o = \frac{M_{WR} + M_{TR}}{M_D + 26.7} = \frac{331.1 + 184.8}{202.2} = 2.6 > 2.0 \text{ OK.}$$

- 6) The eccentricity, e , is calculated as:

$$e = 1.1 \text{ (from Step 6c).}$$

$$e = 1.1 < \frac{L}{6} = 2.3 \text{ OK.}$$

c. Bearing Capacity Failure:

Using Meyerhof:

$$\sigma_v \leq q_a = \frac{q_{ult}}{FS_{bc}} = \frac{q_{ult}}{2}$$

$$\sigma_v = 4.9 \text{ ksf as determined from Step 6d}$$

$$q_{ult} = 0.5(L - 2e) \gamma_f N_\gamma = 52.0 \text{ ksf}$$

$$\text{where } N_\gamma = 70 \text{ for } \phi'_f = 38^\circ \text{ (47)}$$

$$q_a = \frac{52.0}{2} = 26 \text{ ksf} > \sigma_v \text{ OK.}$$

d. Overall Stability:

The overall stability is checked using rotational and wedge analyses. Since this structure is relatively simple, the reinforced soil section is

traditionally considered as a rigid body and only failure surfaces outside the reinforced mass are considered.

The factor of safety obtained is assumed greater than 1.5.

e. Impact Loading

Impact load from traffic barrier = 2,000 lb per ft length of wall. At the time the truck hits the traffic barrier, the 2,000 lb/ft horizontal force will be transferred by the anchored barrier to the ground surface below, over $L_b = 13$ ft (3.96m) length to resist. Using figure 27, for $L_b = 13$ ft and $\phi_r = 39^\circ$, l_1 equals 27.2 ft (8.30m).

The horizontal stress $\sigma_b = 2F_H/l_1 = 2(2,000 \text{ lb/ft})/27.2 \text{ ft} = 147 \text{ psf}$

The lateral stress distribution at the back of the reinforced wall due to the impact load has a magnitude of 147 psf at the top of the pavement and it is linearly decreasing to zero at a height $l_1 = 27.2$ ft from the top of the pavement.

The resultant force due to the impact load $T_{\text{impact}} = 2.0\text{k}$ acting at 20.9 ft from the base of the wall.

Evaluate sliding and overturning stability for impact loading.

1. Check sliding

$$FS_{\text{sliding}} = \frac{(W + W' + P_a \cos \lambda) \cdot \mu}{P_a \cos \lambda + T_{\text{impact}} + T_{\text{Traffic Barrier}}}$$

$$= \frac{(36.1 + 8.4 + 13.2) \times 0.78}{18 + 2.0 + 0.33} = \frac{45}{20.33} = 2.21 \text{ OK}$$

2. Check Overturning

$$FS_o = \frac{M_{WR} + M_{TR}}{M_D + T_{\text{impact}}(20.9) + 0.33(18)} = \frac{331.1 + 184.8}{175.5 + 2.0(20.9) + 5.94}$$

$$= 2.31 \text{ OK}$$

f. Seismic Loading

$$\alpha_o = 0.1$$

$$\alpha_m = (1.45 - 0.1)(0.1) = 0.135$$

$$P_{IR} = \alpha_m \gamma_r H' L = (0.135)(129)(29.3)(14) = 7.1k$$

$$P_{AE} = 0.375 \alpha_m \gamma_b (H')^2 = 0.375(0.135)(129)(29.3)^2 = 5.6k$$

$$50\% \text{ of } P_{IR} = (0.5)(7.1k) = 3.6k$$

Total horizontal force due to the horizontal inertia force P_{IR} and seismic thrust $P_{AE} = 5.6 + 3.6 = 9.2k$

Check sliding and overturning stability due to seismic loading.

1. Check Sliding

$$FS_{sliding} = \frac{[W + W' + P_a \sin \lambda] \cdot \mu}{P_a \cos \lambda + (9.2) + 1.32}$$

$$= \frac{[36.1 + 8.4 + 22.3 \sin 36.3] 0.78}{(22.3 \cos 36.3) + 9.2 + 1.32}$$

$$= \frac{45.0}{28.5} = 1.6$$

$$\text{Dynamic } FS_{sliding} = 0.75 (\text{static } FS_{sliding}) = 0.75(1.5) = 1.1$$

2. Check Overturning

$$FS_o = \frac{M_{WR} + M_{TR}}{M_D + 0.6H(P_{AE}) + 0.5 P_{IR} (12.7)}$$

Horizontal force P_{IR} acts at the center of gravity of the reinforced zone, plus its overburden which is = 12.7 ft (3.87m) from bottom of reinforced wall.

$$FS_o = \frac{331.1 + 184.8}{202.2 + 0.6(29.3)(5.6) + 0.5(7.1)(12.7)}$$

$$FS_o \approx 1.5$$

$$\text{Dynamic } FS_o = 0.75 (\text{static } FS_o) = 0.75 (2.0) = 1.5 \text{ OK}$$

g. Settlement Estimate:

Settlement of the wall is calculated to be less than the performance requirements of the project (<3/4 in).

Step 8. Internal Load Stability

Calculation of Maximum Tensile Forces in the Reinforcement Layers:

- a. The horizontal stress σ_h was calculated at each reinforcement layer

$$\text{Total } \sigma_h = K(\gamma_r z) + \Delta\sigma_h$$

where $K = K(z)$ from figure 26,

$$K_{ar} = 0.394$$

$$S_R = \frac{EA'}{(H/n)} = \frac{(4,176,000)(0.0004)}{2.5} = 668 \text{ k/ft/ft}$$

$$E = 29,000 \times 144 = 4,176,000 \text{ ksf}$$

$$A' = \frac{(b)(t)}{S_h} = \frac{(1.969)(0.078 \text{ in})}{30 \times 12}$$

$$= 0.0004 \text{ ft}^2/\text{ft}$$

$$\frac{H}{n} = \frac{20}{8} = 2.5 \text{ ft}$$

From figure 26, for $S_R = 668$ the value for K was obtained at each reinforcement layer.

- b. Calculate the maximum tension T_{max} per unit width of the wall in each reinforcement layer

$$T_{max} = S_v \cdot \sigma_h$$

T_{max} has been calculated at each layer below the top of wall to determine the required reinforcement strength.

Layer	Z (ft)	K	σ_v (psf)	σ_h (psf)	$\Delta\sigma_h$ (external loads) (psf)	σ_h Total (psf)	T (lb/ft)
1	1.25	0.49	161	79	65	144	360
2	3.75	0.48	484	232	50	282	705
3	6.25	0.47	806	379	35	414	1,035
4	8.75	0.45	1,129	508	30	538	1,345
5	11.25	0.44	1,451	639	20	659	1,648
6	13.75	0.43	1,774	763	10	773	1,933
7	16.25	0.41	2,096	859	-	859	2,148
8*	18.75	0.40	2,419	968	-	968	1,815

*Foundation soil assumed to partially support the layer

Internal Stability with Respect to Breakage

$$T_{max} \leq T_a R_c \quad \text{where } R_c = b/S_h = 0.0656$$

$$T_{a,R_c} = T_a \left(\frac{b}{S_h} \right) = (33,444) (0.0656) = 2,194 \text{ lb/ft}$$

$T_{a,R_c}^{max} \leq T_{a,R_c} = 2,194$ for all reinforcing layers.
Therefore, design is sufficient with respect to maximum tensile forces in the reinforcement.

c. Tensile Stress in Reinforcement

Allowable tensile stress in reinforcement = 36,000 psi (0.55 x 65 = 36 ksi)

$$\begin{aligned} \text{Maximum Tensile Stress per Strip} &= \frac{(\sigma_{h_{max}}) (\text{Tributary Area})}{\text{Area}_{\text{cross section}}} \\ &= \frac{(859 \text{ psf}) (6.05 \text{ ft}^2)}{(1.98 \times 50) \text{ mm}^2 / 25.4^2} \\ &= 33,869 \text{ psi} < 36,000 \text{ psi} \quad \text{OK} \end{aligned}$$

Tensile stress needed for layer 8 was also calculated, however, because there is a smaller tributary area at the bottom, the stress necessary is less than that needed at layer 7. If the actual stress is greater than 36,000 psi a strip with thickness greater than 4 mm should be used, or the horizontal/vertical spacing be reduced for the affected layers.

d. Tensile Stress at Connection

The reduced cross sectional area due to a bolt hole of 9/16 in (14.29 mm) diameter

$$A_{\text{reduced}} = \frac{4(50 - 14.29) \text{ mm}^2}{(25.4 \text{ mm/in})^2} = 0.221 \text{ in}^2$$

Allowable working tensile stress in the connection strip: 20 ksi (.55 x 36 = 20 ksi),

$$\begin{aligned} \text{Therefore, the allowable tensile load} \\ &= 20,000 \text{ psi} (.221 \text{ in}^2) \\ &= 4,420 \text{ lb} \end{aligned}$$

Note: The reduced thickness was not used in this case, as the strip at the connection is contained between the flanges embedded into the back of the wall face and is therefore less susceptible to corrosion. If the reinforcing element is exposed at the connection, then the reduced thickness should be used.

Actual load at the connection: The T_c was determined as shown in figure 28. T_o values at each layer have been calculated at each layer (based on vertical spacing of 2.5 ft between strips, $2.5 \times 1 = 2.5 \text{ ft}^2$ wall area). The actual load at each connection was then calculated as follows:

$$T_{\text{connection}} = (T_o) \frac{(6.05)}{2.5}$$

The load at the connection exceeds the allowable working tensile stress below the 6th layer. Either more reinforcement should be provided at a reduced vertical spacing, or more steel be provided at the connection.

Layer	Z(ft)	σ_h Total (psf)	Z/H	T_o/T_{max}	T_o (lb/ft)	$T_{\text{connection}}$ (lb)
1	1.25	144	<0.5	0.85	306	740
2	3.75	282	<0.5	0.85	599	1,450
3	6.25	414	<0.5	0.85	880	2,130
4	8.75	538	<0.5	0.85	1,143	2,766
5	11.25	659	0.6	0.87	1,434	3,470
6	13.75	773	0.7	0.91	1,759	4,257
7	16.25	859	0.8	0.94	2,019	4,886
8	18.75	968	0.4	0.98	1,779	4,305

Internal Stability with Respect to Pullout Failure

$$T_{\text{max}} \leq \frac{1}{FS_{Po}} (P_r) R_c$$

For reinforced soil wall with sloping surcharge

$$P_r = CF * \gamma_r Z_{av} L_o \alpha$$

Z_{av} = distance from the ground surface to the midpoint of the reinforcement in the resisting zone.

$$F^* = F_q \alpha_\beta + k\mu^* \alpha_f$$

For steel strip, $F_q \alpha_\beta = NA$, $k=1$ and $\alpha_f=1$ (table 7)

$$F^* = \mu^*$$

For steel strip, $C=2$ $\alpha=1$ therefore $P_r = 2\mu^* \gamma_r Z_{av} L_o \cdot 1$

$$T_{\text{max}} \leq \frac{1}{1.5} (2\mu^* \gamma_r Z_{av} L_o) R_c$$

$$L_o \geq \frac{1.5 T_{\text{max}}}{2 \mu^* \gamma_r Z_{av} R_c}$$

$$R_c = \frac{b}{S_h} = 0.0656$$

$$\gamma_r = 129 \text{ pcf}$$

$$\mu^* = 1.2 + \log C_u = 2.2(\text{max})$$

$$\mu^* = \tan \phi_r = 0.81(\text{min})$$

$$\mu^* = 2.2 - 0.0696 Z_{av.} \quad \text{for } Z_{av.} < 20 \text{ ft (See Figure 30, Vol. II)}$$

$$\mu^* = \tan \phi = 0.81 \quad \text{for } Z_{av.} \geq 20 \text{ ft}$$

$$Z_{av.} = Z + 1/2 L \tan \beta = Z + 4.67 \text{ ft}$$

L_e = length of embedment in the resisting zone

$$L_e \geq 0.08865 \frac{T_{max}}{\mu^* Z_{av.}}$$

Layer No.	Z (ft)	Z _{av.} (ft)	μ^*	L _e (ft)	L _s (ft)	L _t = L _e + L _s
1	1.25	5.92	1.788	3.01	6	9.01
2	3.75	8.42	1.614	4.60	6	10.60
3	6.25	10.92	1.440	5.83	6	11.83
4	8.75	13.42	1.266	7.02	6	13.02
5	11.25	15.92	1.108	8.26	5.25	13.51
6	13.75	18.42	0.918	10.13	3.75	13.88
7	16.25	20.92	0.81	11.24	2.25	13.49
8	18.75	23.42	0.81	8.48	0.75	9.23

Pullout stability is satisfied, since $L_t < 14$ feet for all layers of reinforcement.

Step 9. Check anticipated lateral deformation to determine batter requirements. From figure 32, δ_r for $L/H = 0.7$ is 1.

Since the surcharge load = $0.5 (10') (129 \text{ pcf}) = 645$; increase δ_r by 40 percent, therefore $\delta_r = 1.4$.

$$\text{then } \delta_{max} = 1.4 \left(\frac{H}{250} \right) \text{ from figure 32}$$

$$\delta_{max} = 1.4 \frac{20'}{250} = 0.112 \text{ ft} = 1.3 \text{ in}$$

Since the batter will be 0.75 in per 10 ft vertical height, total batter will allow for 1.5 in of movement, therefore the anticipated lateral deformation is acceptable.

Step 10. Check Internal Stability with Respect to Impact Loading

At each reinforcement layer, the horizontal stress due to the impact load was calculated and added to σ_{htotal} for static

loading. T_{max} was calculated for both static and impact loadings.

Layer	Z (ft)	σ_h total static (psf)	σ_h impact (psf)	σ_h (static + impact) (psf)	T_{max} (lb/ft)
1	1.25	144	80	224	560
2	3.75	282	73	355	888
3	6.25	414	59	473	1,183
4	8.75	538	46	584	1,460
5	11.25	659	32	691	1,728
6	13.75	773	19	792	1,980
7	16.25	859	5	864	2,160
8	18.75	968	0	968	1,815

For impact loading $T_{max} < T_{RC} = 2,194$ lb/ft. Therefore, design is sufficient for internal stability with respect to impact loading.

Maximum tensile stress per strip:

for $\sigma_{hmax} = 864$ psf, the maximum tensile stress per strip =
 $33,869$ psi $< 40,000$ psi, OK

Step 11. Check Internal Stability with Respect to Seismic Loading

$$\alpha_m = (1.45 - 0.1) 0.1 = 0.135$$

The maximum tensile force component T_{m1} in each reinforcement equal to T_{max} has been tabulated below

Layer	Z (ft)	T_{m1} ($\sum_v \sigma_h$) (psf)	W(Z) (lb)	P_{IA}	L_c (ft)	$R_c L_c$	$\epsilon R_c L_c$	T_{m2} (ksf)	$T_{max} + T_{m2}$ (psf)
1	1.25	360	8.56	1.16	3.01	0.197	0.197	1.16	1,520
2	3.75	705	8.88	1.20	4.60	0.302	0.499	0.726	1,431
3	6.25	1,035	9.21	1.24	5.83	0.383	0.882	0.538	1,573
4	8.75	1,345	9.53	1.29	7.02	0.461	1.343	0.443	1,788
5	11.25	1,648	9.85	1.33	8.26	0.542	1.885	0.382	2,030
6	13.75	1,933	10.17	1.37	10.13	0.665	2.550	0.357	2,290
7	16.25	2,148	10.50	1.42	11.24	0.737	3.287	0.318	2,466
8	18.75	1,815	10.82	1.46	8.48	0.556	3.848	0.211	2,026

$$P_{IA} = W(Z)\alpha_m = (W' + \gamma Z)\alpha_m$$

$$R_c = 0.0656$$

Check Breakage Failure

$$T_{max} \leq \frac{1}{0.75} T_a \quad R_c \leq \frac{1}{0.75} 2,194$$

$$T_{max} \leq 2,925 \text{ lb/ft}$$

OK

Check Pullout Failure

$$T_{max} \leq \frac{2 \cdot F_d \cdot \alpha}{0.75 \times 1.5} \gamma z \cdot L \cdot R_c$$

$$T_{max} \leq \frac{2}{0.75 \times 1.5} (0.8 \mu^*) (129) (Z_{av.}) L (0.0656)$$

$$\leq 12.04 \mu^* Z_{av.} L$$

Layer	Z (ft)	Z _{av.} (ft)	μ*	L	For L = 14 ft, T _{max} (lb/ft)
1	1.25	5.92	1.788	8.0	1,019
2	3.75	8.42	1.614	8.0	1,309
3	6.25	10.92	1.440	8.0	1,514
4	8.75	13.42	1.266	8.0	1,636
5	11.25	15.92	1.108	8.75	1,858
6	13.75	18.42	0.918	10.25	2,087
7	16.25	20.92	0.81	11.75	2,397
8	18.75	23.42	0.81	13.25	3,026

Reinforcement length must be increased.

3.8 GUIDELINES FOR THE DESIGN OF COMPLEX REINFORCED SOIL WALLS

a. General Considerations

The following reinforced soil retaining structures are considered:

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

They are illustrated in figure 34.

The shape and location of the maximum tensile force line are generally altered by both the geometry and the loads applied on the complex reinforced soil 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 as indicated in chapter 1. The procedures detailed in chapter 4 for evaluating reinforced embankment slopes could be used to evaluate the global stability of reinforced soil walls.

The following sections give guidelines for each case.

b. Bridge Abutments

Reinforced soil 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 in the internal stability analysis have been given in section 3.5. The same type of procedure has to be used for the internal stability of bridge abutment structures, calculating the horizontal stress σ_h at each level by the following formula (equation 29):

$$\sigma_h = K (\sigma_v + \Delta\sigma_v) + \Delta\sigma_h$$

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

$\Delta\sigma_h$ is the increment of horizontal stress due to the horizontal concentrated surcharge P_h and calculated as shown in figure 27. (When supporting abutments on piles could transmit the lateral stress to the reinforced soil section and a reduction in $\Delta\sigma_h$ is not recommended.)

σ_v is the vertical stress at the base of the wall due to the overburden pressure (equal to $\gamma_z Z$ in figure 34a with Z being the depth to the layer of reinforcement).

In the case of large surcharge slabs (with a support length d greater than H/3) at the top of reinforced soil wall, the shape of the maximum tensile force line has to be modified as indicated in figure 35.

Note that in reinforced soil 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. (22, 23) Therefore, the maximum tensile force line should also be modified for extensible reinforcement if the back edge of the slab extends beyond $d = H \tan (45 - \phi/2)$ from the wall face.

c. Sloping Walls

Walls, as considered earlier in this chapter, have facing inclination θ angles greater than 70° . For sloping faced walls with inclinations of 55° to 80° , the standard wall design approach is conservative and it may be desirable to include the face angle in the analysis. The design steps required for rectangular design with the following modifications can be used to reduce this conservatism. However, in each case, a global stability analysis, for checking both internal and external stability should be performed. The procedures outlined in chapter 4 for evaluating reinforced embankment slopes could be used for the global stability analysis. The main modified steps are:

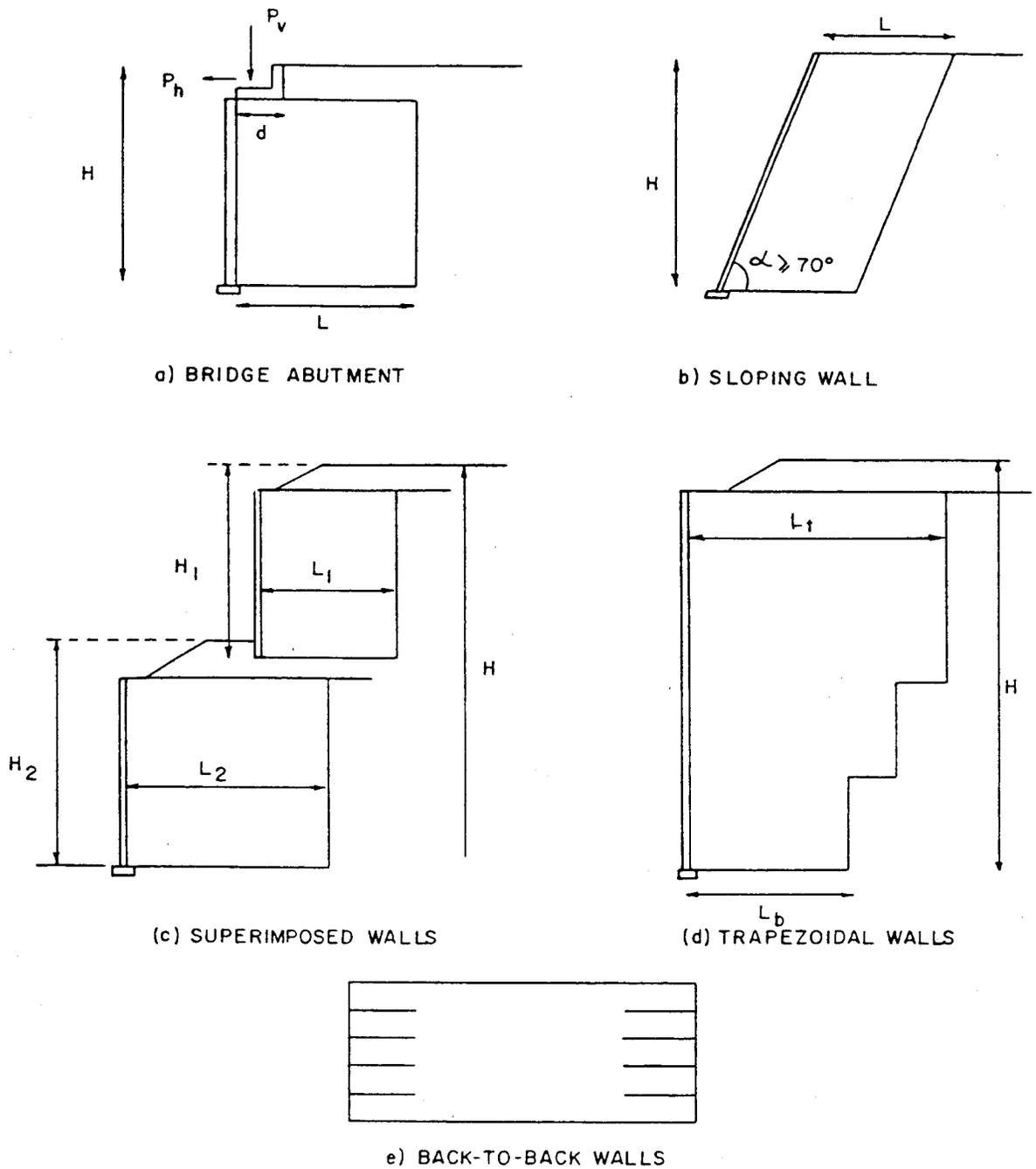


Figure 34. Types of complex reinforced soil retaining structures.

- For preliminary design, the minimum value of the L/H ratio is 0.4.
- The maximum tensile force lines for inextensible and extensible reinforcements are deduced from the ones corresponding to vertical facing, as illustrated in figure 36 by using a reduction coefficient:

$$R_{\theta} = (\theta - \phi_r) / (\pi/2 - \phi_r).$$

- The vertical stress σ_v for external stability evaluation is calculated according to the following formula: ⁽¹⁷⁾

$$\sigma_v = \gamma \cdot \delta \cdot \xi (x/H) \quad (49)$$

where: H is the height of the wall
 δ is a geometrical coefficient
 $\delta = 1$ if $\theta \leq 80^\circ$
 $= L/(L-2e)$ if $\theta > 80^\circ$

(e being the eccentricity calculated from the forces acting on the reinforced soil mass assuming a horizontal thrust P_a)

$\xi (x/H)$ is a function of H and of the distance x to the facing. Values are given by the chart of figure 37 for various values of θ .

- The maximum tensile force per unit width of wall in each reinforcement layer is calculated using the formula:

$$T_{max} = K_{\theta} \sigma_v \cdot S_v \quad (50)$$

where: K_{θ} is a coefficient having a similar distribution to the one for K given in figure 26, by replacing K_o and K_a by the following coefficients:

$$K_{o\theta} = \frac{\sin^2 (\theta - \phi_r)}{\sin \theta \cdot (\sin \theta + \sin \phi_r)} \quad (51)$$

$$K_{a\theta} = \frac{\sin^2 (\theta - \phi_r)}{\sin \theta (\sin \theta + \sin \phi_r)^2} \quad (52)$$

d. Superimposed Walls

The design of superimposed reinforced soil walls is made in two steps:

- 1) Approximate design using simplified design rules for calculating external stability and internal stability.

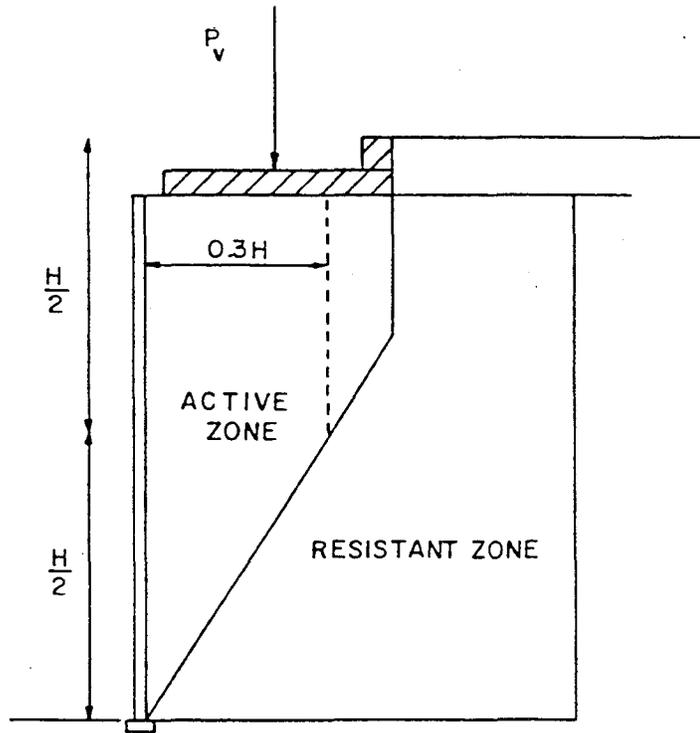
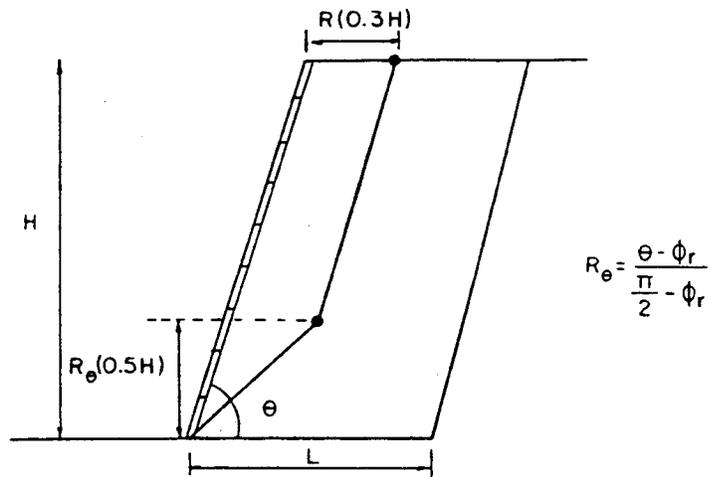
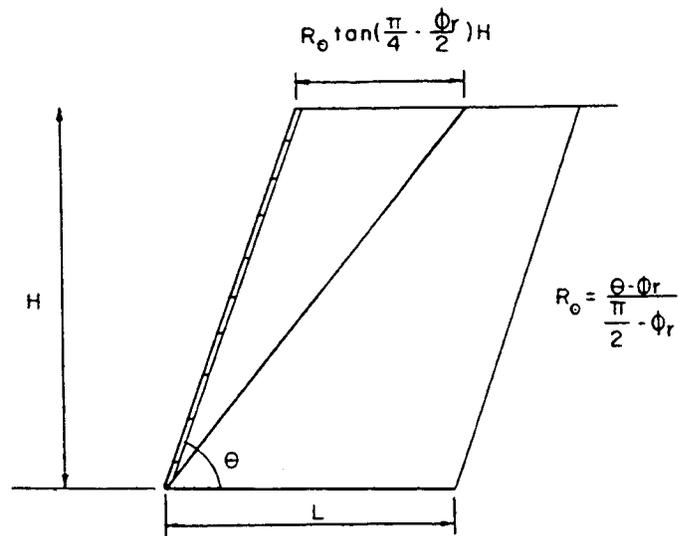


Figure 35. Location of maximum tensile force line in case of large surcharge slabs (inextensible reinforcements).



a) INEXTENSIBLE REINFORCEMENTS



b) EXTENSIBLE REINFORCEMENTS

Figure 36. Location of maximum tensile force line in a reinforced soil sloping wall ($\theta > 70^\circ$).

- 2) Stability analysis, including both internal and external stability using a reinforced soil global stability computer program.

For preliminary design, the following minimum values of L_1 and L_2 should be used:

Upper wall: $L_1 \geq 0.5 H_1$

Lower wall: $L_2 \geq 0.4 H$

(H = total height in figure 33)

The two walls are represented by an equivalent reinforced soil mass, as indicated in figure 38a, having the same cross-section area and a vertical back. Since L_1 and L_2 are not known initially, trial and error must be used. L_1 and L_2 should be set equal to L'_1 and L'_2 for the initial check.

The thrust is inclined at a λ angle to the horizontal:

$$\lambda = \left[1.2 - \frac{(L'_1 + L'_2)}{2H} \right] \phi_b \quad (53)$$

($\lambda = 0$ for extensible reinforcements).

For calculating the internal stability of inextensible reinforcement, the maximum tensile force lines are taken as indicated on figure 38b and the stress at the back of the wall is determined from the total lateral stress assuming a triangular distribution from the top to the bottom of the wall. For internal stability of extensible reinforced structures, the potential failure surface would be modified by shifting the surface by distance D back from the face of the upper section.

e. Walls with a Trapezoidal Section

The design of trapezoidal walls requires two analyses:

- 1) Approximate design using simplified design rules.
- 2) Global stability analysis including both external and internal stability, and performed using a reinforced soil stability program.

Simplified design rules for these structures are as follows:

- The wall is represented by a rectangular block (L_1, H) having the same total height and the same cross-sectional area (figure 39).

- The thrust is calculated at the back of the wall using an inclination angle λ .

$$\begin{aligned} \lambda &= [1.2 - L_1/H] \phi && \text{For inextensible reinforcement} \\ \lambda &= 0 && \text{For extensible reinforcement} \end{aligned}$$

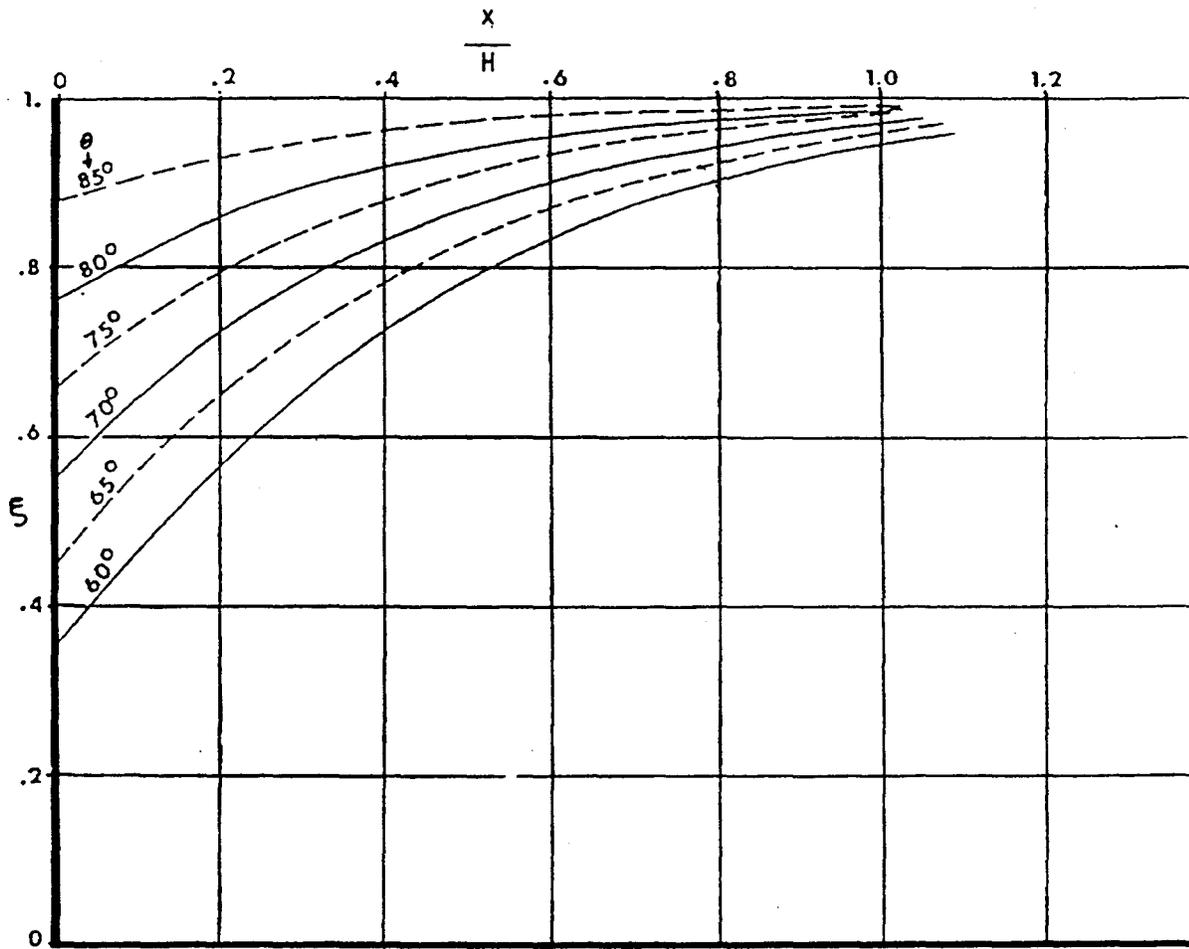
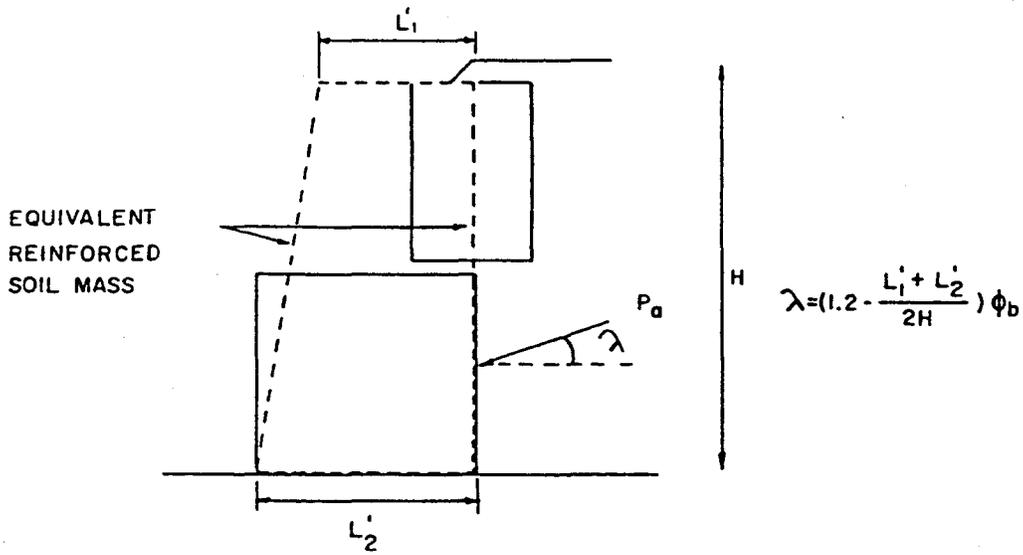


Figure 37. Chart of $\xi (x/H)$ function for various facing inclination θ .



a) EQUIVALENT REINFORCED SOIL MASS

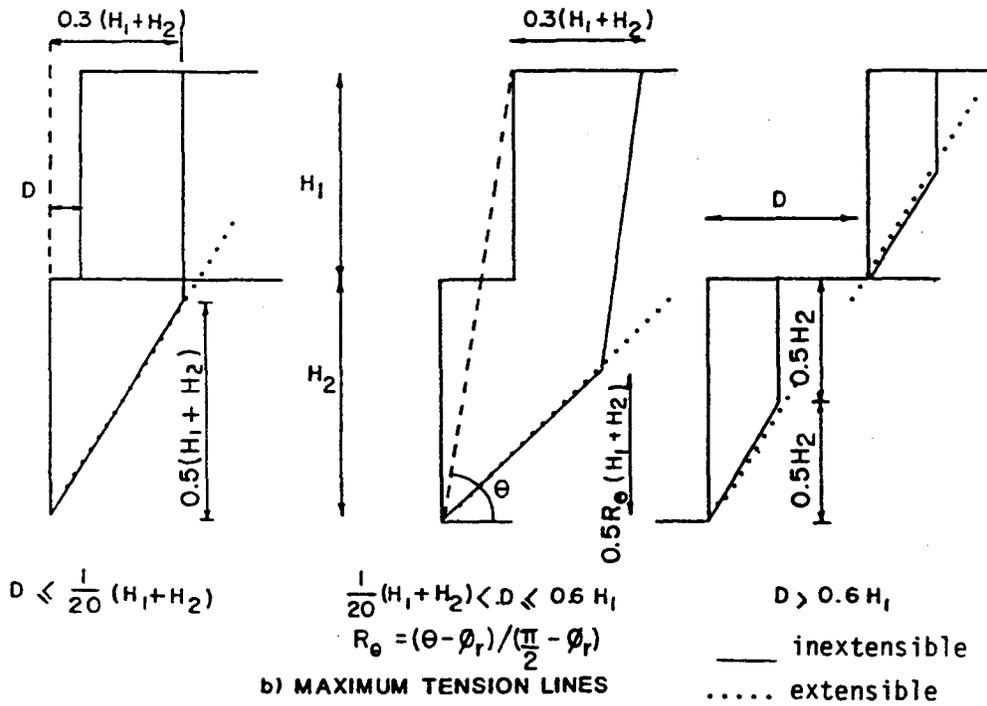


Figure 38. Design rules for superimposed walls.

- The maximum tensile force line is the same as in rectangular walls (bilinear or linear according to the extensibility of the reinforcements).
- For internal stability calculations, the wall is divided in rectangular sections and for each section calculations are conducted according to section 3.5.

f. Back-to-Back Wall Design

The back-to-back design has to be considered in the case of a double-faced wall which is actually two separate walls with parallel facings. This situation can lead to a modified value of backfill thrust which influences the external stability calculations. As indicated on figure 40, 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:

$$D = H \tan (45^\circ - \phi/2) \quad (54)$$

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

$$D > H \tan (45^\circ - \phi/2) \quad (55)$$

full active thrust is mobilized without any inclination on the horizontal ($\lambda = 0$).

- For Case II, there is an overlapping of the reinforcements, so that the two walls interact. Consequently, the two walls are designed independently with the same procedure as in section 3.5, 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.

3.9 INFLUENCE OF EXTERNAL FACTORS ON DESIGN

a. Facing Compressibility

Precast concrete panel facing systems are often used in reinforced soil walls. As these systems are generally less compressible than

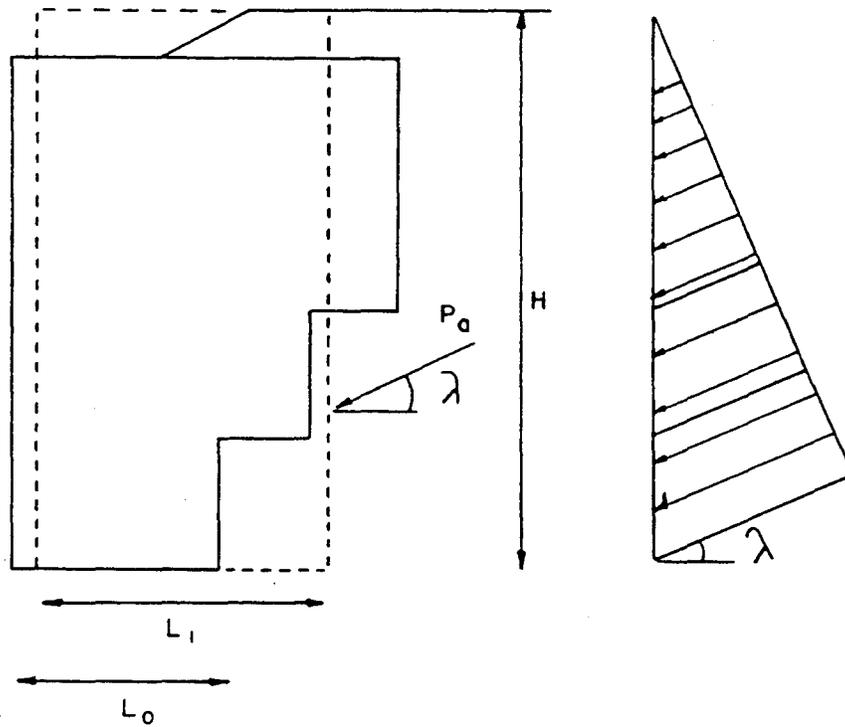
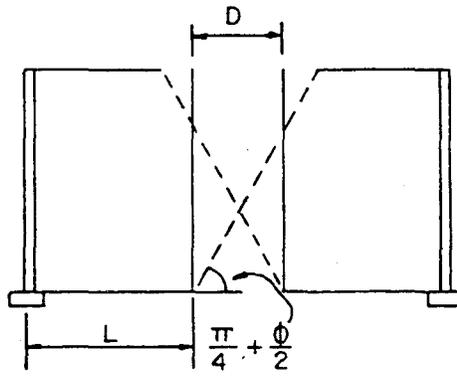
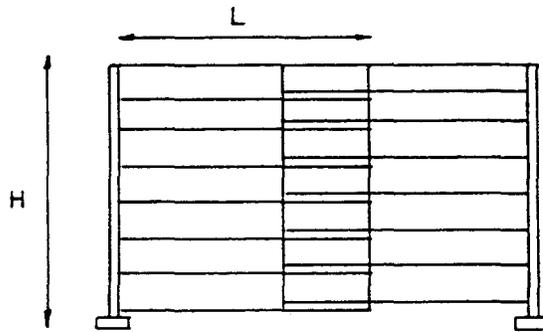


Figure 39. Thrust at the back of a trapezoidal inextensible reinforced wall.



CASE I



CASE II

Figure 40. Back-to-back walls.

the reinforced fill, frictional forces due to relative movement tend to develop between the facing and the reinforced fill leading to increased vertical compressive forces in the facing. This effect can be particularly large for full-height concrete panels. In some cases, corresponding to poorly compacted reinforced fill and/or to fill sensitive to subsidence on wetting, large relatively vertical displacements may occur between the facing and the reinforced fill. Such displacements can lead to overstresses in the reinforcement close to the connections with the panels and eventually to breakage of the reinforcements. This effect may be accentuated by more rigid reinforcements and more rigid reinforcement to panel connections. In order to prevent such difficulties, particular attention has to be given to compaction and quality of the reinforced fill. Additional research is needed to address correction requirements for full height facing panels and caution is advised if these systems are considered.

b. Wall Corners

At a wall corner, it is desirable to place the reinforcements perpendicularly to the facing as it is done in the general case, but this is not possible with sharp angles and in this case, reinforcements are placed obliquely.

The required reinforcements are designed in a conservative manner assuming that the corner has no influence, which leads to design of the wall as if the facing was linear as detailed in Section 3.5.

CHAPTER 4

DESIGN OF REINFORCED SOIL SLOPES

4.1 INTRODUCTION

a. Scope, Purpose, and Organization

There are two main purposes for using reinforcement in engineered slopes:

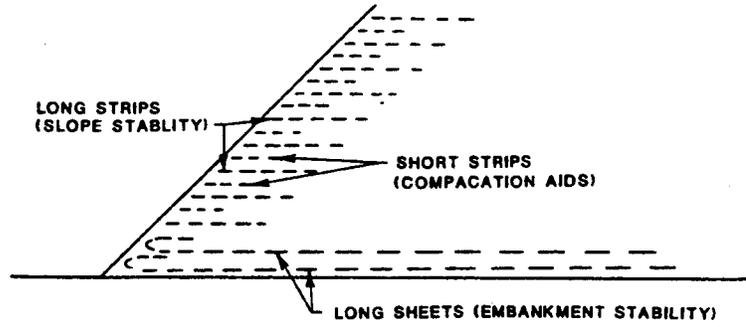
- 1) To increase the stability of the slope, particularly after a failure has occurred or if a steeper than "safe" unreinforced embankment slope is desirable, (see figure 41a).
- 2) To provide improved compaction to the edge of a slope, thus decreasing the tendency for surface sloughing (see figure 41a).

The design of reinforcement for safe steep engineered slopes requires a rigorous analysis that is similar to that needed for designing unreinforced slopes. Step by step design procedures are presented in this chapter. Basic information including the equations and tables needed to perform any given step can be found in:

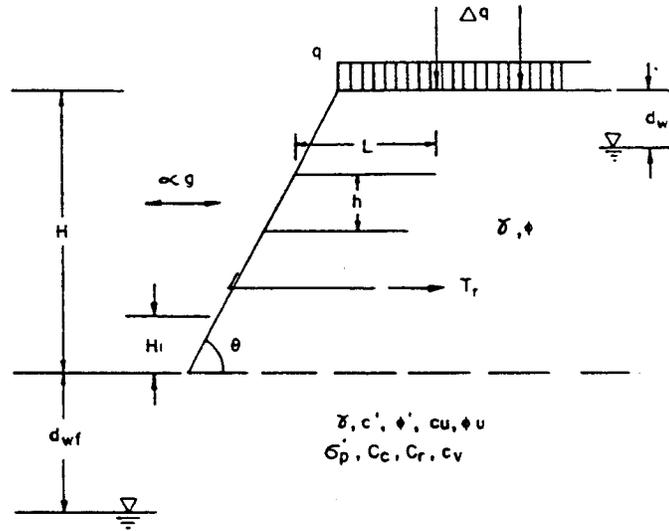
- Chapter 2 for information related to soil properties, reinforcement material properties, and soil-reinforcement interface properties.
- Section 4.1.c of this chapter for a list of the steps necessary for reinforced slope design.
- Sections 4.2 through 4.4 for specifics on slope design.
- Section 4.5 for a detailed design example.
- Chapter 7 for information related to construction.

Further discussion and supporting research results pertaining to design recommendations made in this chapter are included in volume II, Summary of Research and Systems Information.

For the second application, reinforcement, usually geosynthetics, placed at the edges of the embankment slope have been found to provide lateral resistance during compaction, thus allowing for an increase in compacted soil density over that normally achieved. Edge reinforcement also allows compaction equipment to more safely operate near the edge of the slope. Even modest amounts of reinforcement in compacted slopes have been found to reduce sloughing and slope erosion. For this application, the design is simple: place a geotextile, geogrid, or wire mesh reinforcement that will survive construction (see section 2.6.b) at every lift



a) Slope reinforcement using geosynthetics to provide increase slope stability.



b) Requirements for design of reinforced slopes.

Figure 41. Reinforced embankment slopes.

or every other lift along the slope. Only narrow strips about 4 to 6 ft (1.2 to 1.83 m) in width are required and have to be placed in a continuous plane along the edge of the slope.

b. Reinforced Engineered Slope Design Concept

Reinforced slopes are currently analyzed using modified versions of the classical limit equilibrium slope stability methods. A circular or wedge-type potential failure surface is assumed, and 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 the potential failure surface and its allowable design strength. A wide variety of potential failure surfaces must be considered, including deep-seated surfaces through or behind the reinforced zone. The slope stability factor of safety is taken from the critical surface requiring the maximum reinforcement. Detailed design of reinforced slopes is performed by determining the factor of safety with successfully modified reinforcement layouts until the target factor of safety is achieved.

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 ideal method would also include the confinement effects of the reinforcement on the strength of the soil in the vicinity of the reinforcement. Very few of these programs are publicly available, and those are usually limited to specific soil and reinforcement conditions. The methods presented in this chapter use any conventional slope stability computer program and the steps necessary to manually calculate the reinforcement requirements for most any condition.

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 be always in the horizontal direction of the reinforcements as illustrated in figure 41b. However, close to failure, 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 recommended:

- . Inextensible Reinforcements: T parallel to the reinforcements.
- . Extensible Reinforcements: T tangent to the sliding surface.

c. Reinforced Slope Design Steps

The steps for design of a reinforced soil slope are:

- Step 1: Establish the geometric and loading requirements for design.
- Step 2: Determine engineering properties of the natural soils.
- Step 3: Determine properties of available fill.
- Step 4: Establish performance requirements (safety factor values, allowable reinforcement strength, durability criteria).
- Step 5: Check unreinforced stability of the slope.
- Step 6: Design reinforcement to provide stable slope.
- Step 7: Check external stability.

Details required for each step along with equations for analysis are presented in sections 4.2 through 4.4.

- Section 4.2 presents the preliminary design steps 1 through 4.
- Section 4.3 then provides the steps necessary to perform the internal stability analysis, steps 5 and 6.
- Section 4.4 completes the design steps by providing the information required to perform the external stability analysis, step 7.

The procedure assumes that the slope is to be constructed on a stable foundation. It does not include recommendations for deep seated failure analysis. The user is referred to standard soil mechanics texts in cases where the stability of the foundation is at issue. The user is referred to reference 24 for use of reinforcement in the design of embankments over weak foundation soils.

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 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.

4.2 PRELIMINARY DESIGN STEPS

- a. Establish the Geometric and Loading Requirements for Design (see figure 41b):
1. Slope height, H.
 2. Slope angle, θ .
 3. External (surcharge) loading:
 - Surcharge load, q.
 - Temporary live load, Δq .
 - Design seismic acceleration, αg .
- b. Determine the Engineering Properties of the Natural Soils in the Slope
1. Determine foundation and retained soil profiles below and behind the slope and along the alignment to a sufficient depth to evaluate a potential deep seated failure (recommended exploration depth is twice the base width of the reinforced slope or to refusal).
 2. Determine the foundation soil strength parameters (c_u , ϕ_u or c' and ϕ'), unit weight (wet and dry) and consolidation parameters (C_c , C_r , c_v and σ'_p).
 3. Location of the ground water table d_{wf} (especially important if water will exit slope).
 4. If the slope has previously failed and it is to be excavated and rebuilt, make sure that the cause of failure and location of the failure surface have been determined.
- c. Determine Properties of Available Fill
1. Gradation and plasticity index.

Recommended backfill requirements for reinforced engineered slopes:

<u>Sieve Size</u>	<u>Percent Passing</u>
4 in	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 50

Plasticity Index (PI) \leq 20 (AASHTO T-90)
Soundness: Magnesium sulfate soundness loss $<$ 30 percent after 4 cycles.

The maximum aggregate size should be limited to 3/4 inch (19 mm) for extensible reinforcement (e.g. geosynthetics) unless field tests have been or will be performed to evaluate potential strength reductions due to damage during construction.

2. Compaction characteristics based on 95 percent of AASHTO T-99, γ_d and ± 2 percentage points of w_{opt} .
3. Determine recommended lift thickness for backfill material (e.g., 8 in (20 mm) for cohesive soils and 9 to 12 in (229 to 305 mm) for granular soils).
4. Peak shear strength parameters, c_u , ϕ_u and c' , ϕ' .

For granular materials with less than 5 percent passing the No. 200 sieve, use consolidated-drained (CD) triaxial or direct shear tests. Determine and use peak effective stress strength parameters, c' and ϕ' .

For all other soils, determine peak effective stress strength parameters, c' and ϕ' , and total stress strength parameters, c_u and ϕ_u . Use CD direct shear tests (sheared slowly enough that they are drained), or consolidated-undrained (CU) triaxial tests with pore pressures measured.

5. Chemical composition of soil that may affect durability of reinforcement, (pH, chloride, oxidation agents, etc.). See section 2.5. Do not use soils with pH > 12 or pH < 3.

d. Establish Performance Requirements (Recommended minimum design factors of safety are given below; Local Codes may require greater values).

1. External stability:

- Sliding: F.S. = 1.5.
- Deep seated (overall stability): F.S. = 1.3.
- Compound failure (through reinforced zone): F.S. = 1.3.
- Dynamic loading: F.S. = 1.1.
- Settlement--maximum based on project requirements.

2. Internal stability:

- Slope stability: F.S. = 1.3 or greater.
- Allowable tensile force per unit width of reinforcement T_a for each type of reinforcement considered with respect to service life and durability requirements (see chapter 2, section 2.5):

- For steel strips: $T_s = 0.55 \sigma_y t^*$ (56)

where t^* = thickness of strip corrected for corrosion loss.

- For steel grids: $T_s = \frac{0.55 \sigma_y A_c}{b}$ (57)

where A_c = cross section area of all bars (minus corrosion loss) in a width b .

- For geosynthetics: $T_s = \frac{T_{ult} (CRF)}{(FD \cdot FC \cdot FS)}$ (3)

where CRF, FD, FC, and FS are strength reduction factors as described in section 2.4.

- Pullout Resistance: F.S. = 1.5 for granular soils with a 3 ft (0.91-m) minimum length in the resisting zone. Use F.S. = 2 for cohesive soils.

4.3 INTERNAL STABILITY

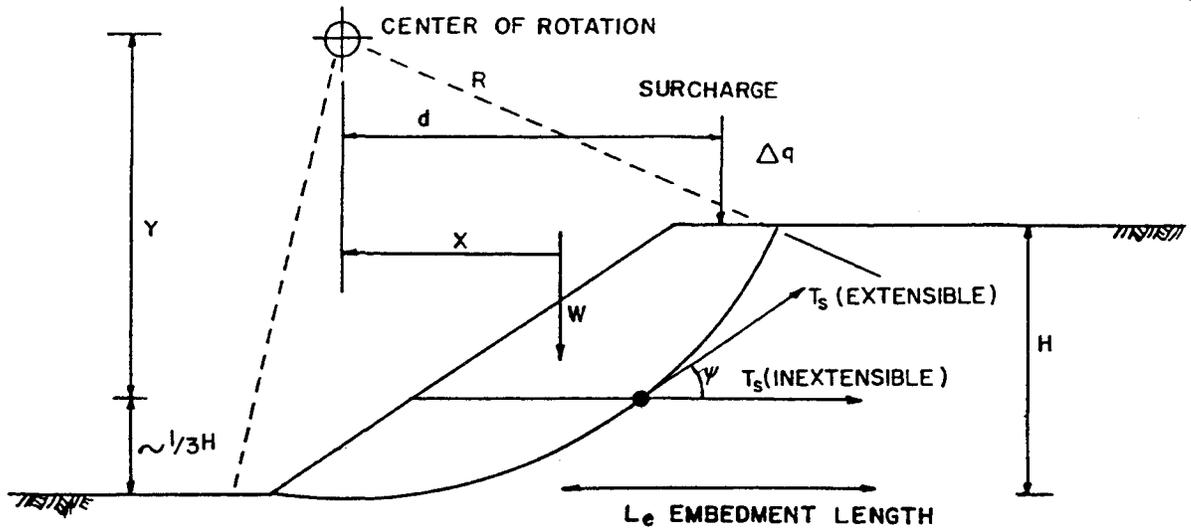
Several simplified approaches are available for the design of slope reinforcement, many of which are contained in the FHWA Geotextile Engineering Manual (see chapter 5 and appendix D of that manual)⁽²⁵⁾. The methods illustrated in figures 42 and 43 which are included in that manual are recommended. Figure 42 shows conventional rotational slip surface methods and can accommodate fairly complex conditions depending on the analytical method used (e.g. Bishop, Janbu, etc.).

Figure 43 presents a simplified method based on a two-part wedge type failure surface in combination with complex circular and noncircular limit equilibrium procedures.⁽²⁶⁾ Some inclination of reinforcement tension was assumed. This method is intended only as a check of the computer generated results and is limited by the assumptions noted on the figure. It is not intended as a single design tool. It is recommended that both the conventional slip surface and simplified chart methods be used, and the results be compared and checked. Judgment in selection of the appropriate design is required.

The following design steps and calculations are necessary for the rotational slip surface method using continuous reinforcement layers:

1. Check Unreinforced Stability

Analyze the slope without reinforcement using conventional stability methods (see FHWA Soils and Foundations Workshop Manual, 1982 or other soil mechanics texts) to determine safety factors and driving moments for potential failure surfaces⁽²⁷⁾. Use both circular and wedge-type surface



Factor of safety of unreinforced slope:

$$F.S._u = \frac{\text{Resisting Moment } (M_R)}{\text{Driving Moment } (M_D)} = \frac{\int_0^{L_{SP}} \tau_f \cdot R \cdot dL}{(Wx + \Delta q \cdot d)} \quad (58)$$

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:

$$F.S. = F.S._u + \frac{T_s \cdot D}{M_D} \quad (59)$$

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 extensible reinforcement
= Y for inextensible reinforcement

Figure 42. Rotational shear approach to required strength of reinforcements.

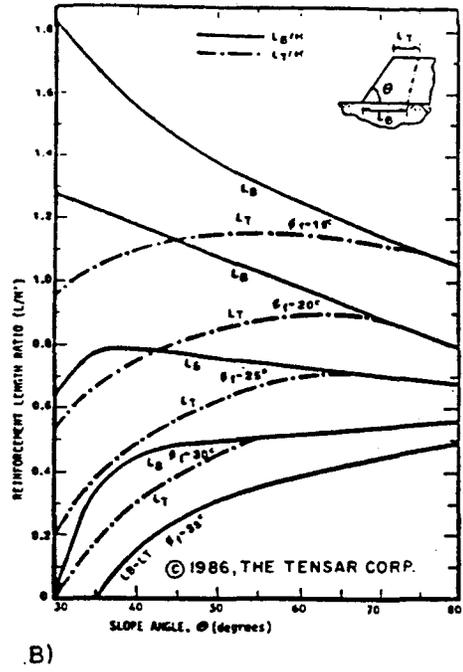
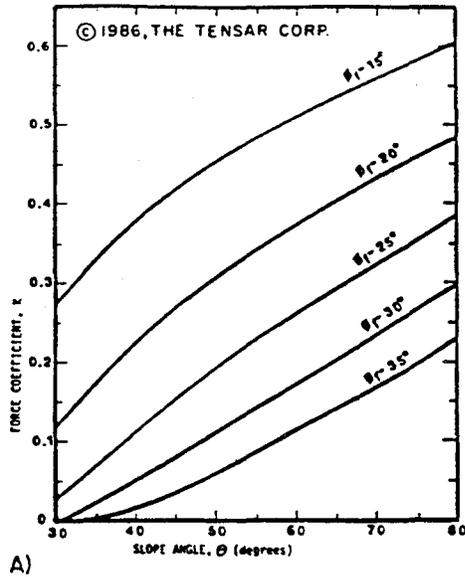


Chart Procedure

Limiting Assumptions:

- Extensible reinforcement.
- Slopes constructed with uniform, cohesionless soil ($c=0$).
- No pore pressures within the slope.
- Competent, level foundation soils.
- No seismic forces.
- Uniform surcharge no greater than $0.2\gamma H$.
- Relatively high soil/reinforcement interface friction angle $\phi_{s_r} = 0.9 \phi_r$ (may not be appropriate for some geotextiles).

1. Determine force coefficient K from Fig. A above where $\phi'_f = \tan^{-1} (\tan \phi_r / FS_R)$.
2. Determine $T_{max} = 0.5K\gamma_r H'$
 where $H' = H + q/\gamma_r$
 q is a uniform surcharge.
3. Determine length of reinforcement L_T and L_B required from chart B.

Figure 43. Chart procedure for confirming reinforced slope design.

shapes to consider failure through the toe, through the face (at many elevations) and in deep seated surfaces below the toe. Computer programs are generally used to speed these analyses. If the reinforced fill and retained fill have significantly different strength properties, it will be necessary to estimate the size of the reinforced soil zone to perform this step. If this estimate subsequently proves to have been poor, this step should be repeated.

To determine the size of the critical zone to be reinforced, examine the full range of potential failure surfaces found to have safety factors less than or equal to the target safety factor for the slope FS_R . Plot all of these surfaces on the cross-section of the slope. The surfaces that just meet the target factor of safety roughly envelope the limits of the critical zone to be reinforced.

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.

2. Calculate the total reinforcement tension T_s required to obtain the required factor of safety FS_R for each potential failure circle inside the critical zone in step 1 that extends through or below the toe of the slope using the following equation:

$$T_s = (FS_R - FS_u) \frac{M_D}{D} \quad (61)$$

where:

- T_s = sum of 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)
 - = vertical distance, Y , to the centroid of T_s for 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_R = target minimum slope safety factor
- FS_u = unreinforced slope safety factor

The largest T_s calculated establishes the required design tension, T_{max} .

3. Determine the required design tension, T_{max} , using the charts in figure 43 and compare with step 2. If substantially different, recheck steps 1 and 2.

4. Determine the distribution of reinforcement:

- For low slopes ($H < 20$ ft [6 m]) assume a uniform distribution of reinforcement and use T_{max} to determine spacing or reinforcement requirements in step 5.
- For high slopes ($H > 20$ ft [6 m]), divide the slope into 2 (top and bottom) or 3 (top, middle, and bottom) reinforcement zones of equal height and use a factored T_{max} in each zone for spacing or reinforcement requirements in step 5. The total required tension in each zone are found from:

For 2 zones:

$$T_{Bottom} = 3/4 T_{max} \quad (62)$$

$$T_{Top} = 1/4 T_{max} \quad (63)$$

For 3 zones:

$$T_{Bottom} = 1/2 T_{max} \quad (64)$$

$$T_{Middle} = 1/3 T_{max} \quad (65)$$

$$T_{Top} = 1/6 T_{max} \quad (66)$$

5. Determine reinforcement vertical spacing S_v :

- For each zone, calculate the design tension T_d requirements 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:

$$T_d = T_a R_c = \frac{T_{zone} S_v}{H_{zone}} = \frac{T_{zone}}{N} \quad (67)$$

- where:
- S_v = multiples of compaction layer thickness for ease of construction
 - T_{zone} = maximum reinforcement tension required for each zone
 - = T_{max} for low slopes ($H < 20$ feet [6 m])
 - H_{zone} = height of zone

= T_{Top} , T_{Middle} , and T_{Bottom} for high slopes ($H > 20$ ft [6 m])

- Use short (4- to 6-ft [1.2 to 1.83 m] lengths of intermediate reinforcement layers to maintain a maximum vertical spacing of 2 ft (61 cm) or less for face stability and compaction quality (figure 44). Intermediate reinforcement should be placed in continuous layers and need not be as strong as the primary reinforcement. For planar reinforcements, if ϕ_r is less than ϕ_r , then ϕ_r should be used in the analysis for the portion of the failure surface intersecting the reinforced soil zone.
6. For critical or complex structures, and when checking a complex design, step 2 should be repeated for a potential failure above each layer of primary reinforcement to make sure distribution is adequate.
 7. Determine the reinforcement lengths required:
 - The embedment length L_e of each reinforcement layer beyond the most critical sliding surface found in step 2 (i.e., circle found for T_{max}) must be sufficient to provide adequate pullout resistance. For the method illustrated in figure 42, use:

$$L_e = \frac{T_a \cdot FS}{F^* \cdot \alpha \cdot \sigma'_v \cdot 2} \quad (68)$$

where F^* , α , and σ'_v , are defined in section 2.8.b. Minimum value of L_e is 3 ft (.91 m). For cohesive soils, check L_e for both short- and long-term pullout conditions. For long-term design, use ϕ'_r with $c_r = 0$. For short-term evaluation, conservatively use ϕ_r with $c_r = 0$ or run pullout tests.

- Plot the reinforcement lengths obtained from the pullout evaluation on a slope cross section containing the rough limits of the critical zone determined in step 1. The length of the lower layers must extend to or beyond the limits of the critical zone. The length required for sliding stability at the base will generally control the length of the lower reinforcement levels. 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 (e.g., see step 8). Make sure that the sum of the reinforcement passing through each failure surface is greater than T_a , from step 2, required for that surface. Only count reinforcement that extend several feet beyond the surface to account for pullout resistance. If the available reinforcement is not sufficient, increase the length of reinforcement not passing through the surface or increase the strength of lower level reinforcement.

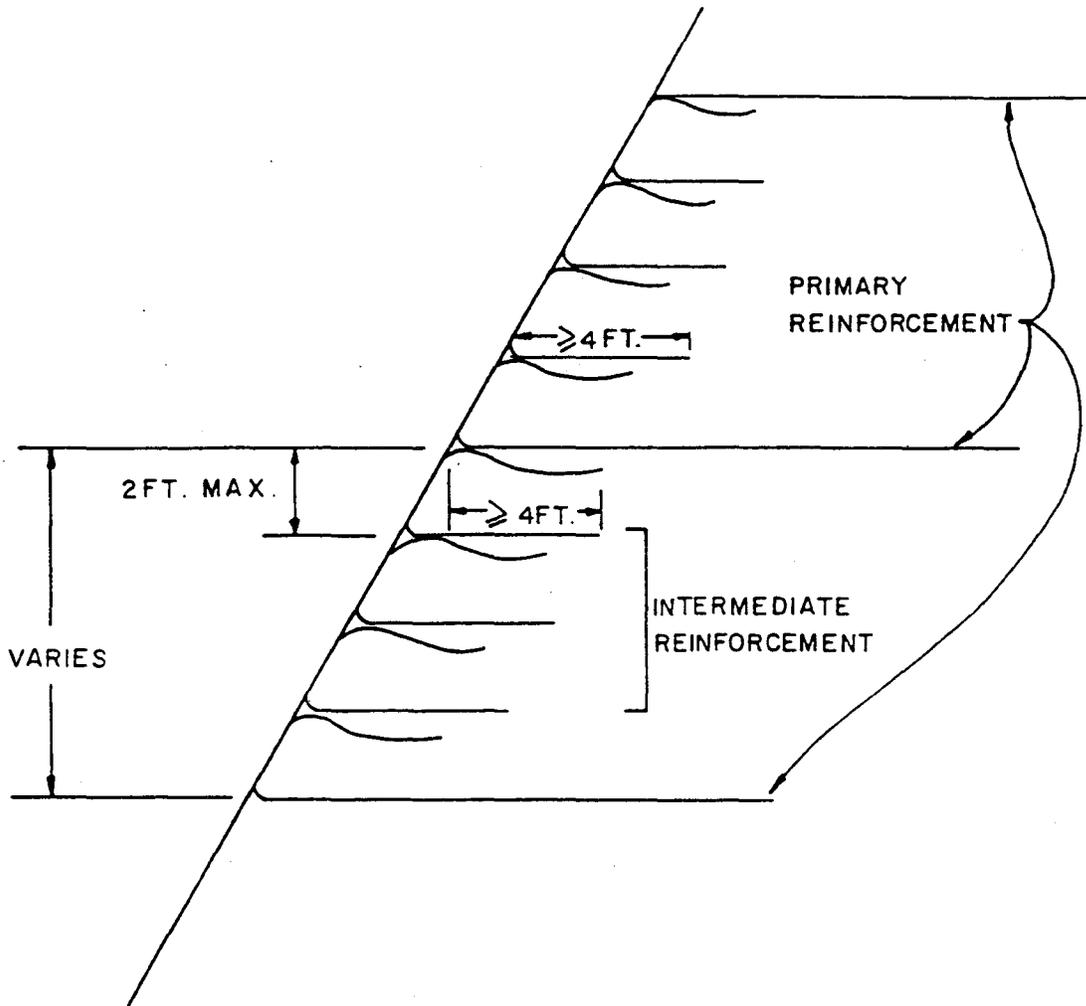


Figure 44. Spacing and embedment requirements for slope reinforcement with intermediate layers.

- Simplify the layout by lengthening some reinforcement layers to create two or three sections of equal reinforcement length. 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 43. L_c is already included in the total length, L_t and L_b from chart B.
8. Checking 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.

4.4 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 include sliding and deep seated overall instability as well as compound failures initiating internally and externally through the reinforced zone. The external stability must be checked for both short and long-term conditions.

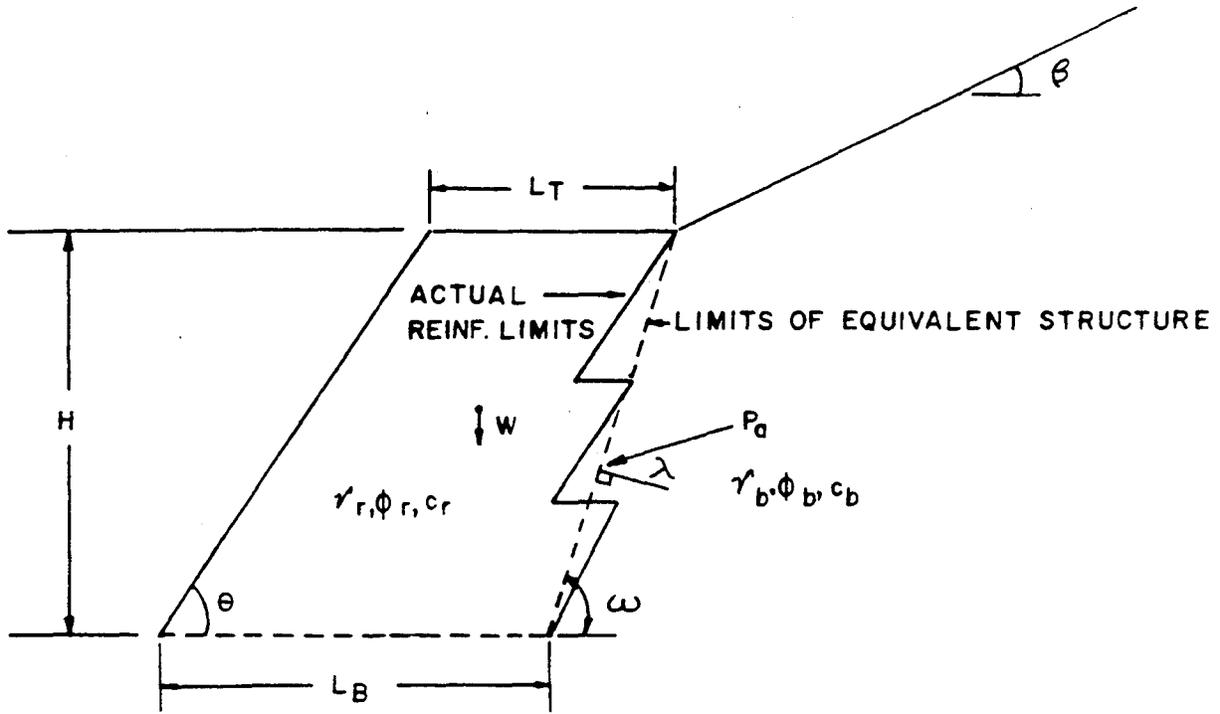
a. Sliding Stability

The reinforced mass must be sufficiently wide at any level to resist sliding along the reinforcement. To evaluate external sliding stability, a wedge type failure surface defined by the limits of the reinforcement can be analyzed using the method used in step 1 and checked using an equivalent rigid structure. For the second approach, a rigid equivalent structure is defined as shown in figure 45a, where the rear boundary is defined as a straight line connecting reinforcement length at top of slope, L_r , with that at the slope base, L_b . The thrust exerted on the rear plane of the reinforced mass is assumed to be parallel to the backfill surface, i.e. $\lambda = \pi/2 - \omega + \beta$, except for the case where $\pi/2 - \omega + \beta$ exceeds ϕ_b . In this case, use $\lambda = \phi_b$.

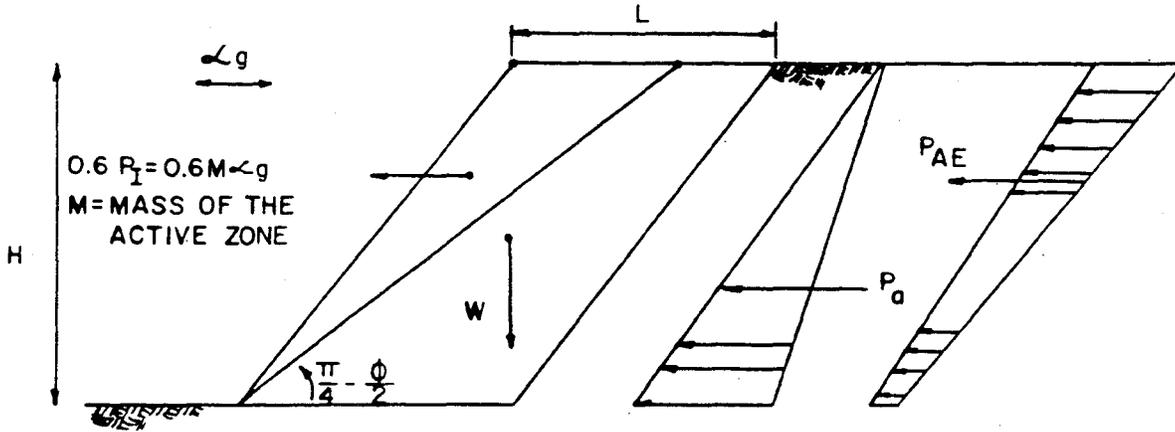
The safety factor is given by the following relationship:

$$F.S._{sliding} = \frac{\text{Resisting Force } P_r}{\text{Sliding Force } P_{SL}} \quad (69)$$

and the calculation steps are:



a) Sliding stability.



b) Seismic stability.

Figure 45. Static and dynamic external stability of a reinforced soil slope.

1. Determine active coefficient K_a (ϕ , θ , λ , ω) using Coulomb's equation:

$$K_a = \left[\frac{\sin(\omega - \phi)/\sin\theta}{\sqrt{\sin(\omega + \lambda)} + \sqrt{\sin(\phi + \lambda)\sin(\phi - \beta)/\sin(\phi - \beta)}} \right]^2 \quad (70)$$

ω : slope angle of equivalent structure limits

β : retained slope angle

2. Calculate the horizontal thrust (sliding force)

$$P_{sL} = P_a \cos(\lambda + \omega - 90) \quad (71)$$

$$= \left[\frac{1}{2} \gamma_b H^2 K_a - 2c_b H \sqrt{K_a} \right] \cos(\lambda + \omega - 90) \quad (72)$$

3. Calculate the resisting force:

$$P_R = W \tan \phi'$$

where ϕ' is the lesser of the friction angle for the foundation soil ϕ'_f , the reinforced soil ϕ'_r , or the soil-reinforcement friction ϕ'_{sr} .

4. Check that the safety factor is greater than 1.5]

$$FS = P_R/P_{sL} \geq 1.5 \quad (73)$$

5. If not, increase the reinforcement length at the base of the slope or both at the base and top of the slope.

b. Deep Seated Global Stability

An analysis should be performed to evaluate stability of potential deep seated failure surfaces completely behind the reinforced soil mass. The analysis performed in step 1 should provide this information. Again, if deep seated failure surfaces are controlling the design, a careful analysis of the embankment support conditions must be performed and foundation improvement methods should be considered.

c. Foundation Settlement

The magnitude of foundation settlement should be determined by using classical geotechnical engineering procedures. If the calculated settlement exceeds project requirements, then the foundation soils must be improved.

4.5 SEISMIC STABILITY

Under a seismic loading, a reinforced soil slope is subjected to two dynamic forces in addition to the static forces:

- An inertia force P_I acting on the active zone of the reinforced soil.
- A dynamic thrust P_{AE} calculated according to pseudo-static dynamic earth pressures using the Mononobe-Okabe method.

These two forces are calculated as indicated on figure 45b where only 60 percent of the inertia force is taken into account because P_{AE} and P_I are unlikely to peak simultaneously.

The seismic external sliding stability is then determined by:

$$[FS] = \frac{W \tan \phi \cdot 0.8}{P_a + P_{AE} + 0.6 P_I} \geq 1.1 \quad (74)$$

(Use ϕ from section 4.4.a)

Check internal stability by using the steps outlined in section 4.3, with an additional horizontal psuedo-static acceleration force, αgW , included in all analysis steps. The target reinforced slope stability safety factor, FS_R , is taken as greater than or equal to 1.1 for seismic analysis. Note that the allowable stress, T_a , for geosynthetic reinforcement may increase under seismic loading as discussed in section 3.5.f.

4.6 DESIGN EXAMPLE

An embankment will be constructed to elevate an existing roadway which currently exists at the toe of a slope with a stable 1.0H to 0.61V configuration. The maximum height of the proposed embankment will be 62 ft and the desired slope of the elevated embankment is 0.84H to 1.0V. It is desired to utilize a geogrid for reinforcing the new slope. The geogrid to be used in the project is a bidirectional geogrid with an ultimate tensile strength of 6,850 lb/ft (10,200 kg/m) (ASTM D4595 wide width method). A uniform surcharge of 250 lb/ft² (1,222 kg/m²) is to be used for the traffic loading condition. Available information indicates that the natural soils have a drained friction angle of 34° and cohesion of 250 lb/ft² (1,222 kg/m²). 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. Establish the Geometric and Loading Requirements for Design

- Slope height, $H = 62$ ft
- Slope angle, $\theta = \tan^{-1} \left(\frac{1.0}{0.84} \right) = 50^\circ$

- c. Retained slope angle, $\omega = \tan^{-1} \left(\frac{0.61}{1.0} \right) = 31.4^\circ$
- d. External loading
 - . Surcharge load, $q = 250 \text{ lb/ft}^2$

Step 2. Determine the Engineering Properties of the Natural Soils in the Slope

For this project, the foundation and existing embankment soils have the following strength parameters

$$\phi' = 34^\circ, c' = 250 \text{ lb/ft}^2$$

$$\text{Depth of water table, } d_{wt} = 5 \text{ ft}$$

Step 3. Determine Properties of Available Fill

The backfill material to be used in the reinforced section was reported to have the following properties.

$$\gamma = 120 \text{ pcf}, \phi' = 34^\circ, c' = 0$$

Step 4. Establish Performance Requirement

- a. External stability
 - . Sliding FS = 1.5
 - . Deep Seated (overall stability) FS = 1.5
- b. Internal stability
 - . Slope stability: FS = 1.5
 - . Allowable tensile force per unit width of reinforcement, T_a , with respect to service life and durability requirements

$$T_a = \frac{T_{ult} (\text{CRF})}{(\text{FD} \cdot \text{FC} \cdot \text{FS})} \quad \text{where } T_{ult} = 6,850 \text{ lb/ft}$$

For the proposed geogrid to be used in the design of the project, the following factors are used:

- FS = 1.5
- FD = durability factor of safety = 1.25
- FC = construction damage factor of safety = 1.2
- CRF = creep reduction factor = 0.5

The above factors of safety are to be based upon either laboratory testing or field experience available by the manufacturer. If not available, then they should be as recommended in chapter 2 of this manual.

Therefore:

$$T_a = \frac{(6,850)(0.5)}{(1.25)(1.2)(1.5)} = 1,520 \text{ lbs/ft} = 1.5\text{k/ft}$$

- . Pullout Resistance: $FS = 1.5$ for granular soils with a 3 ft minimum length in the resisting zone.

Step 5. Check Internal Stability

The internal stability is checked using the rotational slip surface method, as well as the wedge shaped failure surface method to determine the required total reinforcement tension to obtain a factor of safety of 1.5 as follows:

a. Check Unreinforced Stability

The proposed new slope is first analyzed without reinforcement using a computer program such as modified version of STABL II developed by Purdue University. The computer program calculates factors of safety (FS_u) 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. 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. Based on the wedge shaped failure surface analysis, the limits of the critical zone to be reinforced are reduced to 46 ft at the top and 57 ft at the bottom, for the required factor of safety.

- b. The total reinforcement tension, T_s , required to obtain a $FS_R = 1.5$ is then evaluated for each failure surface. The maximum reinforced tension required based on evaluation of 10 critical failure surfaces indicated as $T_{s,max} = T_s = 67$ k/ft, was determined using the following equation:

$$T_s = (FS_R - FS_u) \cdot \frac{M_D}{D} \quad (75)$$

$$T_s = (1.5 - FS_u) \cdot \frac{M_D}{D}$$

$$FS_R = 1.5$$

For T_{max} critical circle (as shown on the design example figure 46)

$FS_u = 0.935$ as determined by the computer program

$M_D = 14,827$ k-ft/ft (as determined by the computer program)

$D =$ radius of critical circle $= 125.6$ ft

$$T_s = (1.5 - 0.935) \frac{14,827}{125.6} = 66.7 \text{ k/ft}$$

c. Chart Design Procedure:

for $\theta = 50^\circ$ and

$$\phi_f' = \tan^{-1} (\tan \phi_r / FS_R) = \tan^{-1} \frac{(\tan 34^\circ)}{1.5} = 24.2^\circ$$

Force coefficient, $K = 0.21$ (from chart A, figure 43)

$$H' = H + q/\gamma_r = 62 + 250/120 = 64 \text{ ft}$$

$$T_{max} = 0.5K\gamma_r (H')^2 = 0.5(0.21)(120)(64)^2 = 52 \text{ k/ft}$$

Values obtained from both procedures are comparable within 25 percent. Use $T_{max} = 67$ k/ft

d. Determine the distribution of reinforcement.

Divide the slope into three reinforcement zones of equal height:

$$T_{bottom} = 1/2 T_{max} = (1/2)(67) = 33.5 \text{ k/ft}$$

$$T_{middle} = 1/3 T_{max} = (1/3)(67) = 22.3 \text{ k/ft}$$

$$T_{top} = 1/6 T_{max} = (1/6)(67) = 11.2 \text{ k/ft}$$

e. Determine reinforcement vertical spacing S_v :

$$\text{Minimum number of layers, } N = \frac{T_{required}}{T_{allowable}} = \frac{67}{1.5} = 44.7$$

Distribute at bottom 1/3 of slope:

$$N_b = \frac{33.5}{1.5} = 22.3 \text{ use } 23$$

At middle 1/3 of slope: $N_m = \frac{22.3}{1.5} = 14.9$ use 15

At upper 1/3 of slope: $N_t = \frac{11.2}{1.5} = 7.5$ use 8

Total number of layers: $46 > 44.7$ OK

Vertical Spacing.

Total Height of Slope = 62 ft

Height for each zone = $\frac{62}{3} = 21$ ft

Required Spacing.

At bottom 1/3 of slope:

$S_{v, \text{required}} = \frac{21}{23} = 0.91$ ft = 11 in Use 8 in

At middle 1/3 of slope:

$S_{v, \text{required}} = \frac{21}{15} = 1.4$ ft = 16.8 in Use 16 in

At top 1/3 of slope:

$S_{v, \text{required}} = \frac{21}{8} = 2.6$ ft = 31.5 in Use 24 in

Provide 6 ft length of intermediate reinforcement layers in the upper 1/3 of the slope, between primary layers.

- f. The reinforcement tension required within the middle and upper third 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 46.

Top 2/3 of slope, $T_{\text{req}} = 31.3$ k/ft $> T_{\text{avail}} = 34.5$ k/ft OK
Top 1/3 of slope, $T_{\text{req}} = 10$ k/ft $> T_{\text{avail}} = 12$ k/ft OK

- g. Determine the reinforcement length required beyond the critical surface used to determine T_{max} .

$$L = \frac{T_a FS}{F \cdot \alpha \cdot \sigma_v \cdot C} = \frac{(1520)(1.5)}{(0.54)(2/3)(120h)(2)}$$
$$= 26.4/h$$

$$\begin{aligned}
T_a &= 1.5 \\
FS &= 1.5 \\
C &= 2 \\
\alpha &= 2/3 \\
F^* &= C_1 \tan \phi = (0.8)(\tan 34^\circ) = 0.54
\end{aligned}$$

At depth, Z, increasing from the top of the crest L_c is found and compared to the available length of reinforcement which extends behind the T_{max} failure surface, as determined by the sliding wedge analysis:

$$\begin{aligned}
Z = 2 \text{ ft, } L_c &= 13.2 \text{ ft, available length, } L = 17 \text{ ft OK} \\
Z = 4 \text{ ft, } L_c &= 6.6 \text{ ft, available length, } L = 16 \text{ ft OK} \\
Z = 6 \text{ ft, } L_c &= 4.4 \text{ ft, available length, } L = 16 \text{ ft OK} \\
Z = 8 \text{ ft, } L_c &= 3.3 \text{ ft, available length, } L = 16 \text{ ft OK} \\
Z > 8 \text{ ft, } L_c &= 3.0 \text{ ft, available length, } L = > 16 \text{ ft OK}
\end{aligned}$$

Checking the length using chart B, figure 43, for $\phi_f = 24.2^\circ$

$$\begin{aligned}
L_T/H' &= 0.65 \rightarrow L_T = 42 \text{ ft} \\
L_B/H' &= 0.8 \rightarrow L_B = 51 \text{ ft}
\end{aligned}$$

Results from both procedures are checked against the wedge failure analysis in Step 5a. Use lengths $L_{top} = 46$ ft and $L_{bottom} = 57$ ft as determined by the computer analyses in step 5a.

- h. The design length was checked for pullout using the slope stability program for failure surfaces extending behind the T_{max} failure surface. The reinforcement available from the layers extending several feet behind each critical zone was compared to and was found greater than the total T_c required for that surface.

Step 6. Check External Stability

a. Sliding Stability

The external stability was checked using the computer program for wedge shaped failure surfaces. The FS obtained for the failure surface outside the reinforced section, defined with a 46 ft (14m) length at the top, and 57 ft (17m) length at the bottom was 1.5 ft (0.5m).

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). The factor of safety for deep seated failure does not meet requirements. Therefore, either the toe of the new slope should be regraded, or the slope would have to be constructed at a flatter angle.

c. Foundation Settlement

Foundation settlement does not exceed project requirements.

CHAPTER 5

DESIGN OF ANCHORED DEADMAN FILL RETAINING SYSTEMS

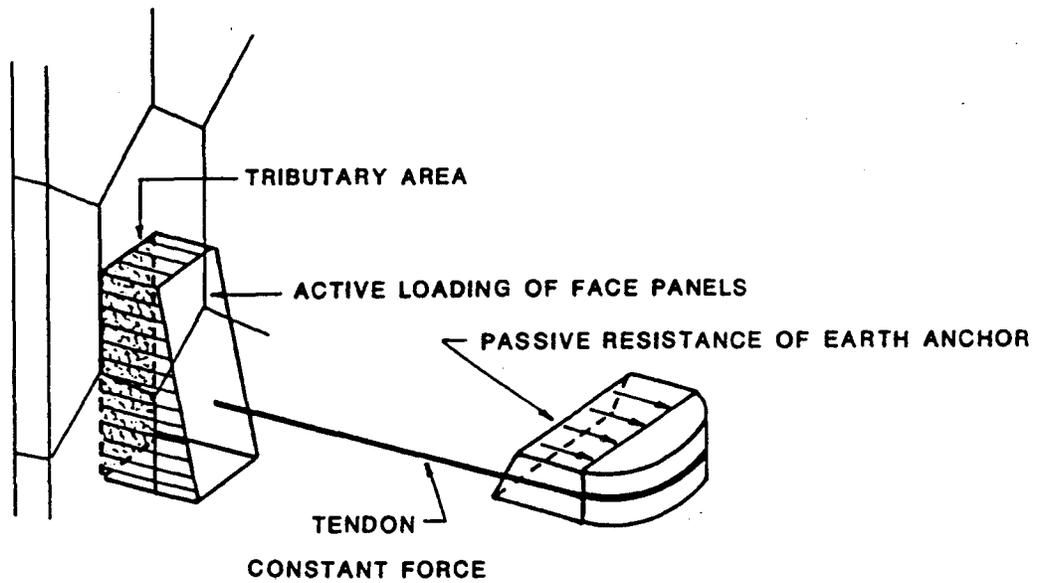
5.1 INTRODUCTION

This chapter presents the design methodology for multianchored retaining systems which derive their pullout resistance from the passive soil pressure on anchored deadman. The structures consist of three basic engineered components including (1) facing made of steel sheet piles (Actumur) or precast reinforced concrete panels, (2) steel tendons made of bars or prestressing strands, and (3) deadman which can be made of precast concrete elements (Geo-Tech System), rocks or concrete rubbles (TRES), metal plates (Actumur), or bent ends of rod reinforcements (Anchored Earth). A description of each type of system can be found in the Description of Systems section of volume II, Summary of Research and Systems Information.

Like soil reinforcement systems, anchored systems use high strength tensile members incorporated within the soil to form gravity retaining structures. Unlike the soil reinforcement systems which transfer the working load to the surrounding soil through frictional stress and/or passive resistance developed along their entire embedment length, anchored deadman systems are designed to ensure the load transfer to the soil through the passive earth resistance developed on the deadman which is located at the free end of the tendon. Therefore, as pointed out in chapter 1, these systems do not create a composite reinforced soil material and their behavior is substantially different from that of reinforced soil systems.

Figure 47 shows schematically the variations of tensile force along an anchored deadman system. The stress transfer is assumed to be primarily through passive resistance and the frictional stress developing along the steel tendon is neglected. As such, the retaining system operates similarly to a tied-back wall and the tensile forces are assumed to be constant along the tendons.

The main difference between these systems and tieback walls which rely upon ground anchors resides in the load transfer mechanism from the anchors to the soil. In ground anchor retaining systems, the load transfer is being realized by the friction mobilized at the grout-ground interfaces whereas in anchored deadman systems, the load transfer is being realized through the passive soil pressure mobilized on the deadman. These two load transfer mechanisms require a significantly different magnitude of soil displacements to be mobilized and can therefore result in a substantially different behavior (i.e., earth pressure distribution on face elements, location of potential failure surface, structure displacements). However, as field experience with multianchored deadman systems is still rather limited, several basic design assumptions for tied back walls have been adapted in this chapter to provide conservative design schemes.



TENDON FORCE F_A - ACTIVE EARTH PRESSURE x WALL AREA SUPPORTED BY TENDON
 ANCHOR CAPACITY F_P - PASSIVE EARTH PRESSURE x EFFECTIVE ANCHOR FACE AREA

Figure 47. Load transfer in anchored walls.

The basic differences in the behavior of these systems as compared with reinforced soil retaining structures imply different design considerations with regard to the engineered structural components.

Unlike reinforced soil systems, in a multianchored deadman system, similarly to a tied back wall, the facing is primarily a structural element which has to withstand both bending moments and shear forces due to the lateral earth pressure of the retained soil and to transfer tension forces to the tendons.

5.2 DESIGN PRINCIPLES

Following the design principles outlined in chapter 3, the design of all anchored retaining systems should ensure their long term internal and external stability with appropriate factors of safety.

Internal stability considerations pertain to the design of the structural elements: the facing, the tendons, the deadman and their connections and imply the following design criteria:

- Bending and shear resistances of the facing elements should be sufficiently high to withstand the working stresses due to the lateral earth pressures applied by the retained backfill and surcharge.
- Tensile resistance of the tendon should be adequate with respect to the forces transferred to the deadman.
- Corrosion protection and analysis is based on a reduced cross sectional tendon area should be considered to account for a specified corrosion rate and ensure the structure performance over the design service period.
- Pullout resistance of the deadman should be high enough to prevent slippage of the anchor in the retained mass.
- Connections of the tendons to the facing and to the deadman elements should be properly designed with an adequate shearing resistance to prevent the tendons from pulling out of the elements.

Evaluation of the external stability is based on engineering considerations which are common to all types of gravity retaining structures and requires assessment of the safety factors with respect to four potential failure modes, including:

- Overturning of the wall.
- Sliding of the wall on its base.
- Bearing capacity failure of the foundation.

- General sliding of the wall and the surrounding soil mass.

The structural design of the anchored wall consists of the following basic steps:

1. Assume an aspect ratio (i.e., ratio of tendon length to wall height); ratio will generally range from 0.5 to 0.8 depending on the backfill material.
2. Evaluate the external stability of the structure.
3. Calculate the lateral earth pressure acting on the face of the wall and the required passive earth pressure on the deadman.
4. Select facing elements, tendon section and spacings, and deadman.
5. Verify the structural stability of the face panels, i.e., that panels can resist bending moments due to the lateral earth pressures and that the shear resistance at the connections of the tendons to the panels is large enough to prevent tendon pullout.
6. Verify that tendon section is large enough to withstand the estimated tension forces transferred to the deadman.
7. Verify the structural stability of the deadmen with respect to the bending moments and shear failure at the connections.
8. Verify the safety factor with respect to pullout failure of the anchor.
9. Select corrosion protection of tendons.
10. Other design considerations: drainage of backfill material, architectural aspects, etc.

5.3 EXTERNAL STABILITY EVALUATION

Procedures to evaluate the external stability of the anchored wall with respect to the potential failure modes are identical with those in section 3.4 of chapter 3 for reinforced fill retaining walls. All of the steps required in that section should be followed for anchored systems.

5.4 INTERNAL STABILITY EVALUATION

The basic design assumptions with regard to the internal stability of the structural components concern:

- Distribution of lateral earth pressure acting on the wall.
 - Passive earth resistance mobilized on the anchor deadman.
 - Tension forces mobilized in the tendons.
 - Inclination of potential failure surface and location of the deadman elements.
 - Pullout capacity of the anchor.
 - Structure displacements.
- a. Lateral Earth Pressure on Facing Elements and Anchor Deadmen

The mobilization of the lateral earth thrust on the anchor deadman is a passive phenomenon. Consequently, it requires a rather large displacement that would generally allow the soil in the active zone, behind the facing, to attain the limit Rankine's active state of stress. Figure 48 illustrates schematically the load transfer to the anchor and typical wall displacements required to mobilize passive and active earth pressures on rigid retaining walls.

Figure 49 shows the results of a full scale experiment on a multianchored wall^(1,2) built with a facing made of fabric attached to anchored vertical concrete columns and with a silty backfill material. The anchors consisted of steel rods attached to concrete vertical plates [3 ft by 3 ft by 0.5 ft (1 m by 1m by 0.15 m)]. Measurements of the lateral earth pressure on the facing elements and on the deadmen show that in this retaining system, the displacement (rotation) of the facing is sufficient to attain the active earth pressure on the facing. The displacement of the anchor rods results in a mobilization of passive lateral earth thrust on the concrete plates to maintain static equilibrium of the anchored system.

The local equilibrium of each anchored rod implies, as illustrated in figure 48, that the force transferred by the face panel to the anchor is equal to:

$$F_{am} = \sigma_{am} \times S_{face} \quad (76)$$

The force transferred by the anchor to the deadman is equal to:

$$F_{pm} = \sigma_{pm} \times S_{deadman} \quad (77)$$

Hence,

$$\frac{\sigma_{pm}}{\sigma_{am}} = \frac{S_{facing}}{S_{deadman}} \quad (78)$$

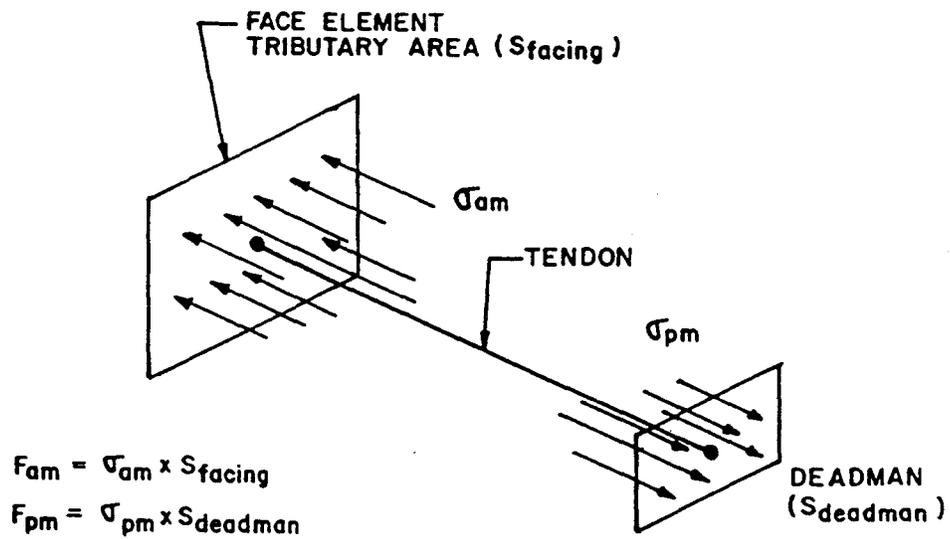
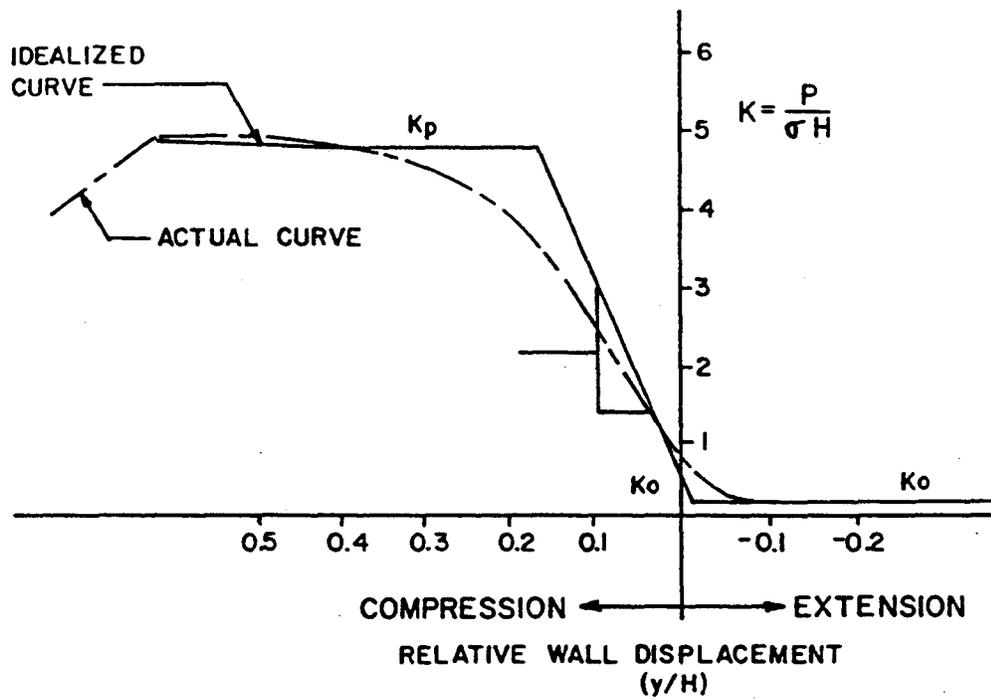
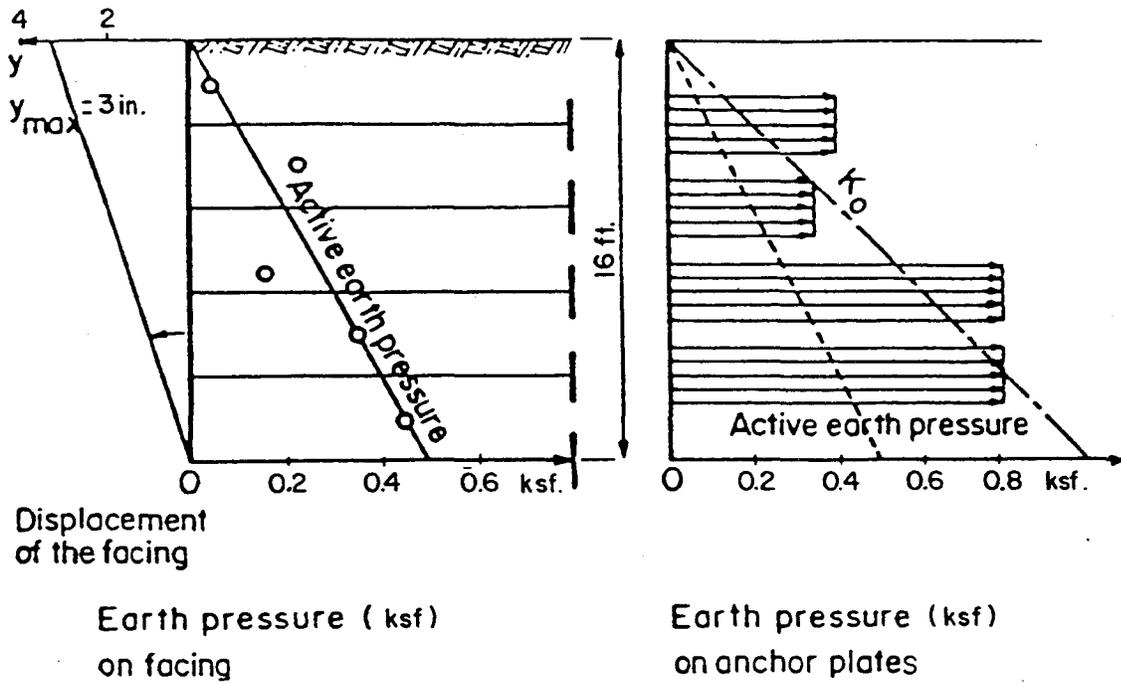


Figure 48. Load transfer and associated displacement to mobilize resisting pressures.



Note: 2.54 cm = 1 in
 47.9 kN/m² = 1 ksf

Note: 2.54 cm = 1 in
 47.9 kN/m² = 1 ksf

Figure 49. Behavior of a multianchored wall.⁽²⁸⁾

where: σ_{pm} is the passive earth pressure mobilized on the deadman element
 σ_{am} is the lateral active soil pressure acting on the facing element
 S_{facing} is the tributary surface area of the facing per each anchor rod
 $S_{deadman}$ is the surface area of the deadman element for each anchor rod.

Equation 76 implies that the surface area ratio $R = S_{facing} / S_{deadman}$ governs the lateral earth pressure mobilized on the facing elements and on the deadmen and thereby the anchor displacements.

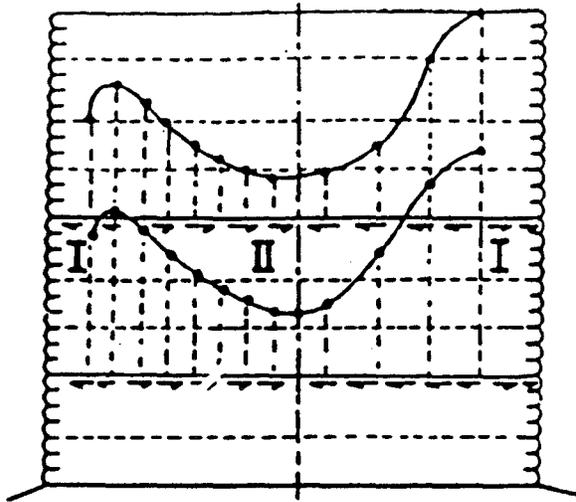
For $R = 1$, the lateral displacements of the backfill material is assumed to be restrained and therefore:

$$\sigma_{am} = \sigma_{pm} = K_0 \sigma_v \quad (79)$$

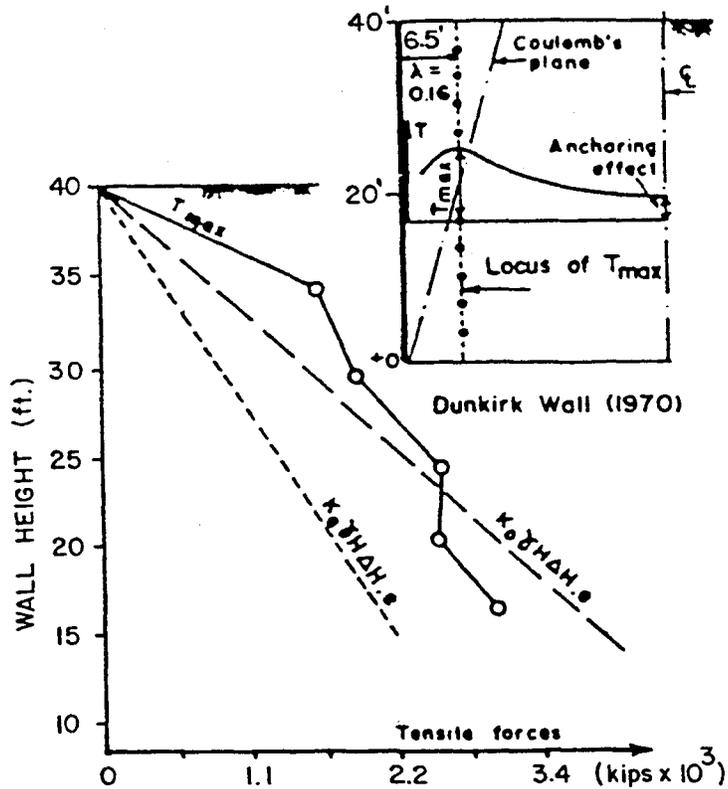
where: $K_0 = 1 - \sin \phi'$ is the coefficient of earth pressure at rest
 ϕ' = the internal friction angle of the backfill material
 σ_v = the vertical stress at the level of the anchor.

A particular example of a multianchored wall with surface ratio of $R = 1$ is a Reinforced Earth wall with double facings constructed in France. ⁽²⁹⁾ In this 43 ft (13 m) high wall, reinforced earth metallic strips are used to connect the two metallic facings. The variation of tension forces along the reinforcements is shown in figure 50a, illustrating that the tension forces generated in these strips are being transferred to the soil through both interface friction and passive earth resistance mobilized on the facing elements. As shown in figure 50b, maximum tension forces generated in the strips correspond to the "at rest" earth pressure.

As R increases, the surface area of the deadman decreases, the passive earth pressure required to maintain equilibrium increases, as well as the anchor displacements which can be determined from pullout tests. The limit passive earth



d) TENSILE FORCES ALONG THE REINFORCEMENTS



b) MAXIMUM TENSILE FORCES DISTRIBUTION

Note: 4.45 kN = 1 kip

Figure 50. Tensile forces distributed in a double-faced Reinforced Earth wall.

pressure is governed by the bearing capacity of the backfill materials which controls the pullout resistance of the anchor.

Current design practices with regard to the assumed lateral earth pressures on the face elements varies with the technology.

Anchored earth systems which use bent rods to mobilize the passive earth resistance are currently designed, as outlined in NCHRP-290, assuming Rankine's active lateral earth pressure.⁽²⁾ However, a full-scale experiment on an instrumented anchored earth wall has shown that, as illustrated in figure 51, tension forces measured in the rods exceed those predicted for K_0 state of stress in the soil.⁽³⁰⁾

Geo-Tech anchored walls are currently designed assuming "at rest" earth pressure distribution on the face panels yielding more conservative design schemes. This design assumption is consistent and supported by the results of the laboratory model tests on reduced scale instrumented models of anchored walls conducted under the present study.⁽³¹⁾

TRES retaining wall systems are designed assuming the active and passive earth pressures to be fully mobilized on the face panels and deadmen, respectively. This design assumption is probably consistent with the R values corresponding to the deadman element used in practice. However, no experimental data have yet been provided to support this design assumption which could lead to underestimating the lateral earth pressures on the face panels.

Actumur walls are designed following conventional procedures which are commonly used in design of anchored sheet piles. Several approaches have been developed to calculate the lateral earth pressures on anchored sheet piles which can be broadly classified into two main categories:

1. Active-passive earth pressures of the retained ground and the foundation soil on the back and front faces of the sheet pile, respectively; the net earth pressure diagram is mainly a function of the embedment depth and boundary displacements (fixed versus free end displacement at the bottom end of the sheet pile).
2. "P-y" elasto-plastic method, derived from Winkler's theory for the bending of beams on elastic supporting media; this approach allows the design engineer to evaluate the effect of the bending stiffness of the sheet pile and of the tendon elongation and displacements on the lateral earth pressure distributions.⁽³²⁾

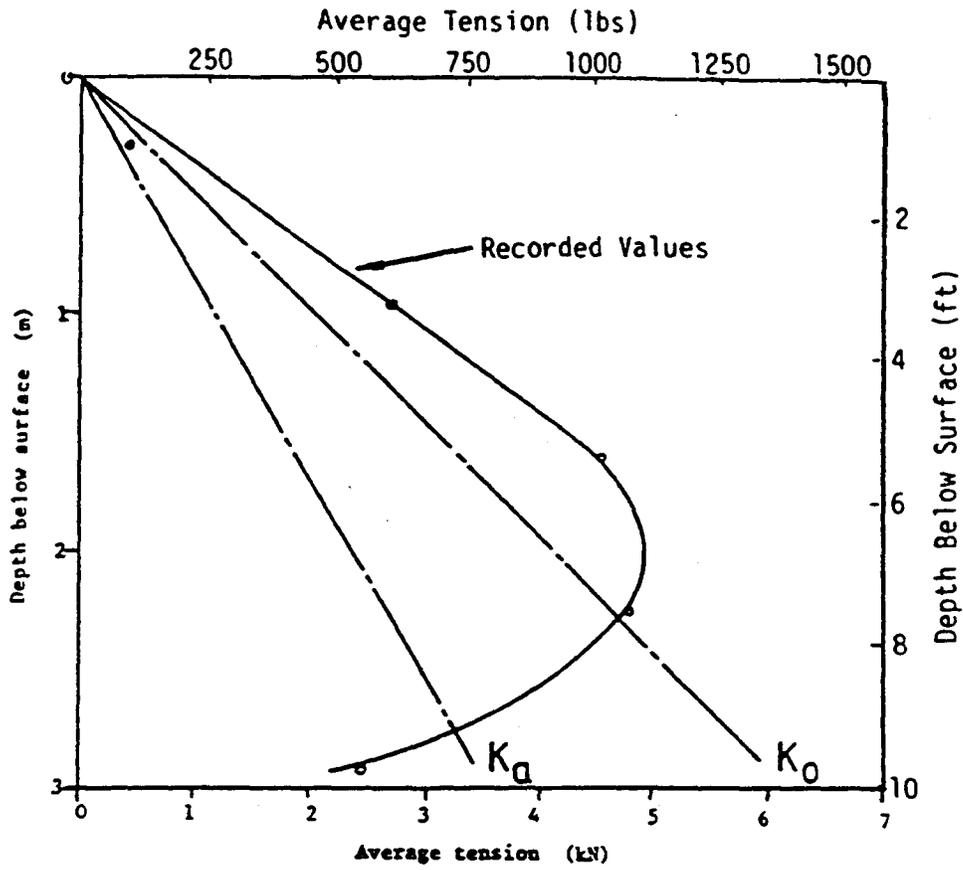


Figure 51. Vertical distribution of average anchor tension at end of construction. (30)

The forces in the anchors are calculated either from the global equilibrium conditions (moments and forces) of the sheet pile or, in the case of multi anchored walls, from the local equilibrium at each level of anchors (i.e., the force in the anchor is equalized to the lateral earth pressure on the sheet pile multiplied by the tributary face area per each anchor).

The current design assumptions with respect to the lateral earth pressures on the facing elements and anchor deadmen are summarized in table 10. For all the systems under consideration based on the review of current design practice and the available experimental data, the following recommendations are proposed:

Table 10. Recommended coefficients of lateral earth pressures on face panels and deadmen.

System	Current Design Practice face/deadmen	Recommendations		Experimental Data
		face $\sigma_{am} \setminus \sigma_v$	deadmen $\sigma_{pm} \setminus \sigma_v$	
Anchored Earth	K_a /N.A.	K_o	N.A.	full scale experiment ⁽³⁰⁾
Geo-Tech Wall	K_o	K_o	Eq. 76 assuming k_o at face	reduced scale models ⁽³¹⁾
TRES Systems	K_a/K_p	K_o	Eq. 76 assuming k_a at face	no data
Actumur	anchored sheet pile design	K_o or current procedures	Eq. 76 assuming k_a at face or current procedures	no data

Note:

$$K_a = \tan^2 \left(\frac{\pi}{4} - \frac{\phi}{2} \right)$$

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

$$K_p = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right)$$

b. Tension Forces in the Tendons

Tension forces in the tendons can be calculated from eq. 76, that is:

$$F = \sigma_{an} \times S_{facing} \quad \text{with: } \sigma_{an} = K \cdot \sigma_v; \text{ and } \sigma_v = \gamma z.$$

The earth pressure σ_{an} acting on the facing element at the anchor level z is calculated using the recommended earth pressure coefficients indicated in table 10.

Table 11 yields for granular and cohesive soils the values of tension forces in the tendons as a nondimensional parameter:

$$T_N = \frac{F}{\gamma z \cdot S_H \cdot S_v}$$

Table 11. Estimate of tension forces in the tendons.

System	Granular Soils	Cohesive Soils
Anchored Earth	K_o	K_o
Geo-Tech wall	K_o	K_o
TRES system	K_a	$K_a - 2\sqrt{K_a} \frac{c'}{\gamma z}$
Actumur	K_a or sheet pile design procedure	$K_a - 2\sqrt{K_a} \frac{c'}{\gamma z}$ or sheet pile design procedure

It should be noticed that the available systems are broadly classified in two main categories:

1. Systems that are anticipated to require large soil displacements to mobilize the passive earth resistance on the deadman elements such as TRES systems. Such systems will allow soil displacement that would result in a Rankine's active state of stress and fully mobilized shear strength of the soil along the potential failure surface. Therefore, in a cohesive soil, allowance should be made for the effect of soil cohesion on the lateral earth pressure acting on the facing element. If used, cohesive soils should have a plastic index of less than 20.

2. Systems that will restrain soil displacements and maintain a "quasi" K_0 state of stress. Such systems will not allow the shear strength of the soil to be fully mobilized and therefore no allowance is considered for the effect of soil cohesion on the lateral earth pressure acting on the facing element. This assumption is conservative and would result in an overestimate of the tension forces in the tendons. It is anticipated that further research and full-scale experiments in cohesive soils will permit development of more rational design method that can substantially affect construction costs.

Surcharge effect of surcharge loading on tension forces in the tendons is estimated using the procedure indicated in chapter 3 and assuming a diffusion at one horizontal to two vertical.

c. Location of Deadman

For the anchor to be effective, the deadman has to be located behind the potential failure surface. The location of the failure surface is highly dependent upon the soil displacement required to mobilize the passive earth pressure on the deadmen and therefore on the anchor spacings and deadmen geometry. As R increases, the displacement increases and the soil at the active zone attains the Rankine's active state of stress.

It can therefore be expected that as R increases, the potential failure surface will approach the Rankine's failure plane inclined at $(\pi/4 + \phi/2)$ to the horizontal. As R approaches one, the deadmen are assumed to restrain the soil displacement and the potential failure surface will therefore approach that observed in Reinforced Soil walls with inextensible reinforcements.

Measurements of tension forces along the strip reinforcements in the Reinforced Earth wall with double facings, illustrated in figure 51, show that the maximum tensile force line which is assumed to coincide with the potential failure surface is quite similar to that observed on inextensibly reinforced fill walls. The assumption of Rankine's failure plane is therefore expected to result in a substantial overestimate of the width of the active zone in structures with R values close to one (figure 51).

Experimental data to support the design assumptions with regard to the inclination of the potential failure surface in multianchored walls are not available. A conservative approach which is currently used (NCHRP 290) in the design of anchored earth walls and in the design of tied back walls (FHWA DP-68-1R) is therefore recommended, assuming that the potential failure surface coincides with Rankine's failure plane.⁽²⁾ The minimum distance between the deadman and the potential failure surface should ensure that the soil mass susceptible to undergo bearing capacity failure due to anchor pullout is located at a minimum distance of $H/5$ (where H is the structure height) from the potential failure surface.

This requirement is consistent with FHWA recommendations for the design of tie back walls.⁽³³⁾ Hence, the total length of the tendon at a level z should be equal to (figure 52):

$$L(z) = L_o(z) + H/5 + B \cdot \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \quad (80)$$

where: $L_o(z)$ is the width of the active zone in the considered level

$$\text{i.e., } L_o(z) = (H-z) \tan\left(\frac{\pi}{4} + \frac{\phi}{2}\right)$$

B is the width or height of the deadman element.

Further research is required to develop more rational design guidelines with regard to the location of the failure surface which can substantially affect the construction costs.

d. Pullout Capacity

The pullout capacity of the anchor deadman can be estimated from bearing capacity formula for deeply embedded strip footings (i.e., embedment depth z significantly greater than the height t of the deadman). A generic equation for the pullout capacity can be derived:

$$P_r = [c \cdot t \cdot F_c + \gamma'z \cdot F_q + \frac{1}{2} \gamma'z t^2 F_f] B \quad (81)$$

where: c is the drained soil cohesion
 γ' is the effective unit weight of the soil
t and B are, respectively, the height and width of the deadman
 F_c , F_q , and F_f are, respectively, cohesion, surcharge (or embedment) and friction bearing capacity factors.

The first term is generally negligible as the soil cohesion c is usually small for the backfill materials currently used. In the second term, the bearing capacity factor F_q , defined as the ratio of the effective bearing resistance σ_b' to the effective vertical stress σ_v' (i.e., $F_q = \sigma_b'/\sigma_v'$, see figure 53), can be calculated from the equation developed by Murray for anchored earth:⁽³⁰⁾

For individual anchor

$$F_q = \tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) \frac{\exp[2(\pi - \Omega) \tan \phi']}{\cos \Omega} \quad (82)$$

where $\Omega = 70^\circ$ which approximates the bearing wedge.

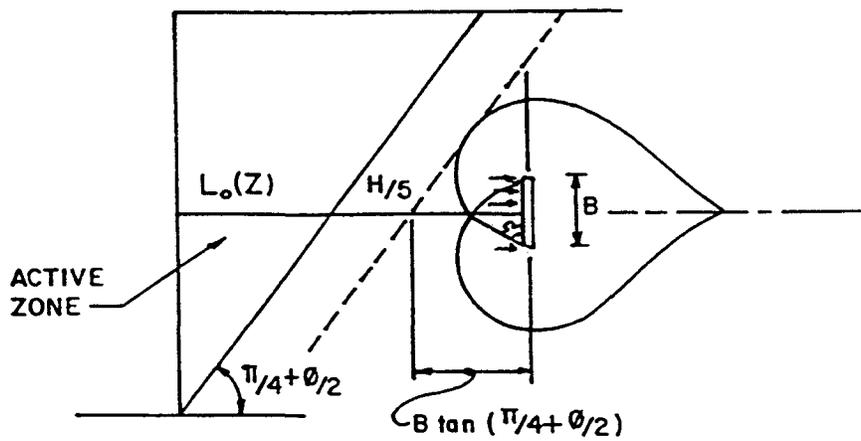
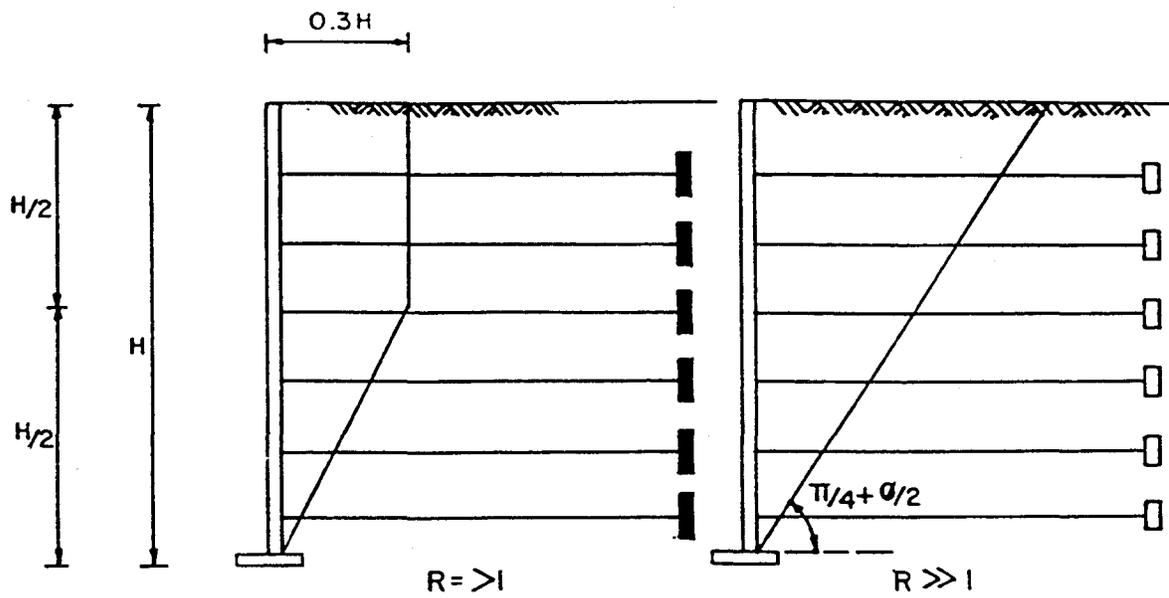


Figure 52. Recommended design procedure to establish tendon length.

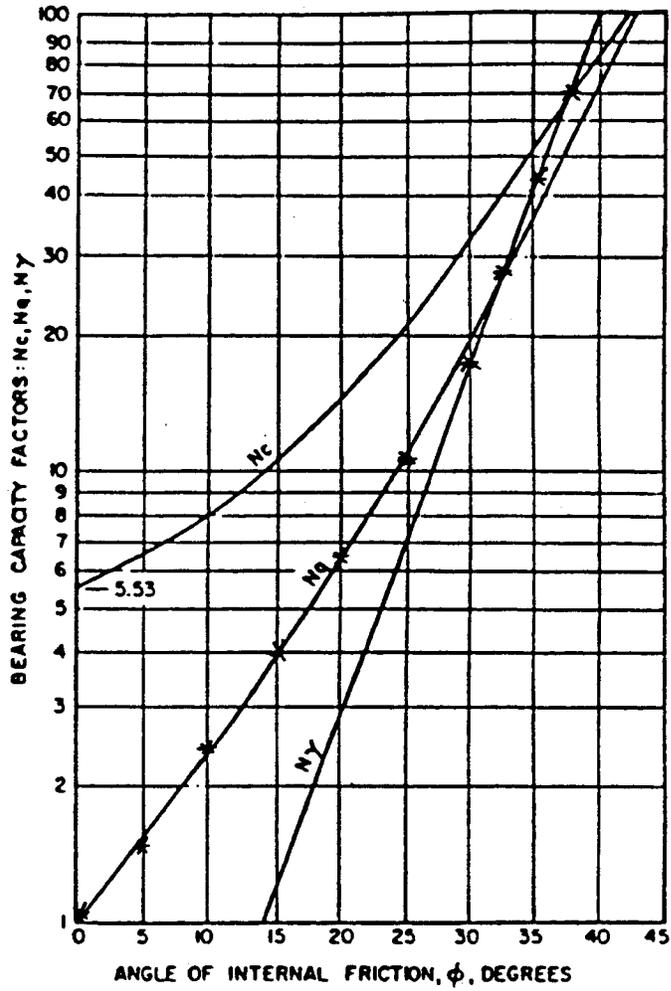


Figure 53. Bearing capacity factors for the estimate of anchor pullout resistance.

For closely spaced anchors

$$F_q = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \frac{\exp[2(\frac{3\pi}{4} - \omega) \tan \phi]}{\cos \omega} \quad (83)$$

where $\omega = 70^\circ$.

Figure 53 yields bearing capacity factors for the estimate of pullout capacity of the anchors proposed by Geo-Tech, Inc. The F_q values are quite consistent with those calculated using equation 84:⁽³⁴⁾

$$F_q = \tan^2 \left(\frac{\pi}{4} + \frac{\phi}{2} \right) \exp[\pi \cdot \tan \phi] \quad (84)$$

It should be noted that present design recommendations of Geotech, Inc., imply using a factored ϕ' value ($\phi' = 0.6 \times \phi'$) and assume that the lateral confining stress at the level of each anchor is given by the Rankine's active earth pressure (i.e., the overburden pressure in the second term is multiplied by the active earth pressure coefficient, K_a). These design recommendations would result in a substantial underestimate of the pullout resistance of the anchors.

e. Structure Displacements

The maximum lateral displacement at the top of the anchored wall can be estimated simulating the wall with a structural beam under linear earth pressure distribution.⁽³⁵⁾ The maximum lateral deflection at the top, provided anchor displacement is negligible, is given by:

$$\frac{Y}{H} = \frac{K_a \cdot \gamma H^4}{30 EI} + \frac{K_a \cdot \gamma H^4}{5GL} \quad (85)$$

where: $I = L^3/12$, $E = 2G(1+\nu)$, G is the shear modulus,
 $\nu = 0.33$ is the Poisson's coefficient
 L = length of anchor rod (from back of face to the anchor)

or:

$$\frac{Y}{H} = \frac{K_a}{5} \cdot \frac{\gamma H}{G} \cdot \left[\left(\frac{H}{L} \right) + 0.75 \left(\frac{H}{L} \right)^3 \right]$$

Example: assume $\phi = 35^\circ$, $G/\gamma H = 60$ (for a typical sand)

for $H/L = 2 \Rightarrow y/H = 7.2 \times 10^{-3}$
for $H/L = 1 \Rightarrow y/H = 1.6 \times 10^{-3}$

5.5 STRUCTURAL DESIGN

a. Face Elements

Selection and design of face elements for each technology is based on conventional structural design procedures. Precast reinforced concrete panels, used with anchored earth, TRES systems or Geo-Tech walls, are simulated by reinforced concrete beams (or rafts) resting on elastic foundation soils. The analytical model is schematically illustrated in figure 54. Available analytical solutions and AASHTO design recommendations are used to calculate the maximum bending moments in the face panels as a function of the assumed lateral earth pressures and the estimated forces in the tendons. Using AASHTO structural design procedures for the beam model, the moments may be taken as follows:

$$\text{At each anchor} \quad M = \frac{NS}{12}$$

$$\text{Between anchors} \quad M = \frac{NS}{24}$$

where N is the tensile force in the anchor and S is the distance between anchors (S_h and S_v).

Using AASHTO structural design procedures for rafts, the moments may be taken as follows:

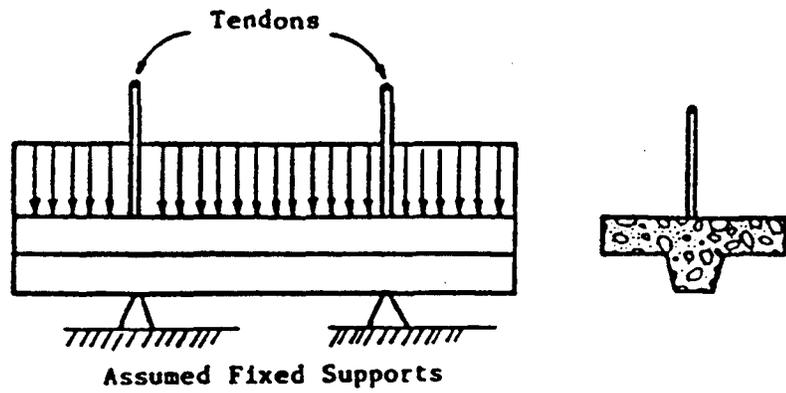
$$\text{At each anchor} \quad M = 0.70 \frac{NS}{12}$$

$$\text{In between anchors} \quad M = 0.57 \frac{NS}{24}$$

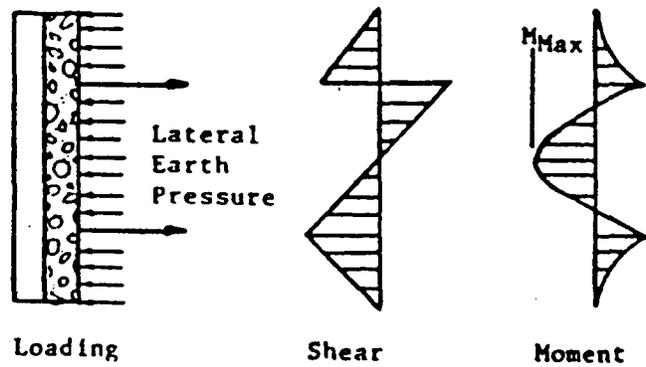
Considering Winkler's solution for a beam or raft on an elastic foundation, reduction factors for moments can be computed. However, for a wide range of soil conditions and feasible anchor spacing, such reduction factors should be generally neglected considering uncertainties in design.

Structural design may be carried out using either working stress or ultimate strength methods. It is recommended that AASHTO ultimate strength methods using the raft model be used.

A detailed design procedure for the face elements is beyond the scope of this section and the user is referred to standard reinforced concrete design text. Available design specifications for the precast reinforced concrete panels used with the above specified technologies are reported in Description of Systems Section of volume II.



a) Face panel analytical model.



b) Panel analysis procedure.

Figure 54. Design of face element.

b. Tendons and Corrosion Protection

The tendons are designed to resist the tension forces transferred to the deadman with an adequate corrosion protection. Each technology uses different tendons and corrosion protection systems.

Geo-Tech walls use tendons formed with ASTM A242 low alloy high strength steel which is a highly corrosion resistant steel. The corrosion protection is provided by an 8 mil (315 μm) coating of fusion bonded epoxy that also coats the threaded connections. In addition to epoxy coatings, corrosion protection of the tendons is provided for by using reduced allowable design stress (i.e., the allowable design stress is 0.66 times the allowable design stress of the steel).

Flexible tendons, using epoxy-coated prestressing strands, are also available. They are attached to the facing panels at two points and pass through the deadman eliminating the mechanical connection within the fill. At the panel, the flexible tendon is anchored using a conventional strand anchorage device.

TRES systems use tendons made from standard galvanized or epoxy coated steel rebar bent into a U shape and wrapped around the deadman. Epoxy coated tendons are usually used for adverse soils or marine applications.

Anchored earth systems use low fabrication cast bent rods. The anchors are formed from mild steel bars of 0.6 to 0.8 in (15 to 20 mm) diameter having a screw threaded portion at its front end. The corrosion resistance of the bent rods used in anchored earth walls is still not well known, specifically pregalvanized steel may be subject to corrosion at the welded portion of the rod. It is therefore generally recommended that the anchors be galvanized after bending and welding. However, to date, corrosion rate of the bent rods remains to be further investigated and suitable corrosion protection system to be developed for long-term application. It is recommended that the sacrificial rod thickness concept used in design of reinforced soil walls be used to determine the required diameter of the steel based on the required design life of the project and the backfill to be used (see chapter 2).

c. Anchor Deadmen

The design of the anchor deadmen and their connections to the tendons should ensure (1) the required pullout capacity, and (2) adequate shear resistance at the connections to prevent the tendon from pulling out of the deadman. The design concept is based generally on converting the tension forces in the tendons to compressive forces within the deadman element. As such, reinforcing steel requirement of the deadman element is minimal and is controlled by normal temperature expansion considerations. This minimal requirement for reinforcing steel is an important feature of the deadman design because corrosion of the reinforcing steel in the buried deadman is difficult to protect.

CHAPTER 6

DESIGN OF NAILED SOIL RETAINING STRUCTURES

6.1 INTRODUCTION

a. Scope, Purpose, and Organization

In this chapter, the design approaches to predict the nail forces and to evaluate the stability of a stabilized mass using nails are presented.

Information related to soil properties, reinforcement material properties, and soil-reinforcement interaction needed to perform the analysis are covered in chapter 2.

Construction techniques for soil nailing are covered in chapter 7.

The background information used as a basis for this chapter was developed in a separate FHWA project (Manual of Practice for Soil Nailing. ⁽¹⁵⁾ Comments and supporting research pertaining to design recommendations made in this chapter are included in the Soil Nailing section of volume II, Summary of Research and Systems Information.

b. Basic Behavior and Design Concepts

The basic design concept of a nailed soil retaining structure relies upon the:

- Transfer of resisting tensile forces generated in the inclusions in the active zone into the ground in the resistant zone through friction (or adhesion) mobilized at the soil-nail interface.
- Passive resistance developed on the surface perpendicular to the direction of soil-nail relative movement.

The frictional interaction between the ground and the quasi "non-extensible" steel inclusions restrain the ground movement during and after excavation. The resisting tensile forces mobilized in the inclusions induce an apparent increase of normal stresses along potential sliding surfaces (or rock joints) increasing the overall shear resistance of the native ground. Nails placed across a potential slip surface of a slope can resist the shear and bending moment through the development of passive resistance. The main engineering concern in the design of these retaining systems is to ensure that ground-inclusion interaction is effectively mobilized to restrain ground displacements and can ensure the structural stability with an appropriate factor of safety.

imilarly to reinforced fill systems, ground nailing by closely paced, passive inclusions results in a composite coherent material. As schematically illustrated in figure 55, the maximum tensile forces generated in the nails are significantly greater than those transferred to the facing. The locus of maximum tensile forces separates the nailed soil mass into two zones: an active zone (or potential sliding soil or rock wedge) where lateral shear stresses are mobilized, and a resistant (or stable) zone where the generated nail forces are transferred into the ground. Laboratory model tests have demonstrated that this maximum tensile force line coincides with the potential sliding surface in the soil. (36)

The soil-nail interaction is mobilized during construction and a certain value of structure displacement occurs as the resisting forces are progressively generated in the nails. Therefore, it has been essential to monitor actual structures, to measure the facing displacements in different types of soils and to verify that they are compatible with performance criteria. Measured horizontal facing displacements in several soil nailed structures indicate that, in nonplastic soils, maximum facing displacement is generally less than 0.3 percent of the structure height. (11, 37, 38, 39, 40) This ground movement is comparable to that observed in braced retaining systems.

c. Ground Nailing Design Steps

As for most reinforced fill systems, the design procedure for a nailed soil retaining structure should include the following tasks:

- For the specified structure geometry (depth and cut slope inclination), ground profile, and boundary (surcharge) loadings, estimate working nail forces and location of the potential sliding surface.
- Select the relevant reinforcement (type, cross-sectional area, length, inclination, and spacing) and verify local stability at each reinforcement level, i.e. verify that nail resistance (strength and pull-out capacity) is sufficient to withstand the estimated working forces with an acceptable factor of safety.
- Verify that the global stability of the nailed soil structure and the surrounding ground is maintained during and after excavation with an acceptable factor of safety. For the case where soil nailing is used in slope stabilization, the global stability of the nailed slope also has to be checked.

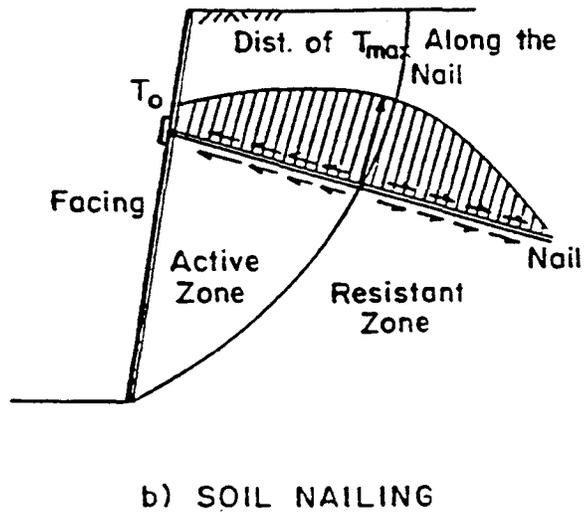
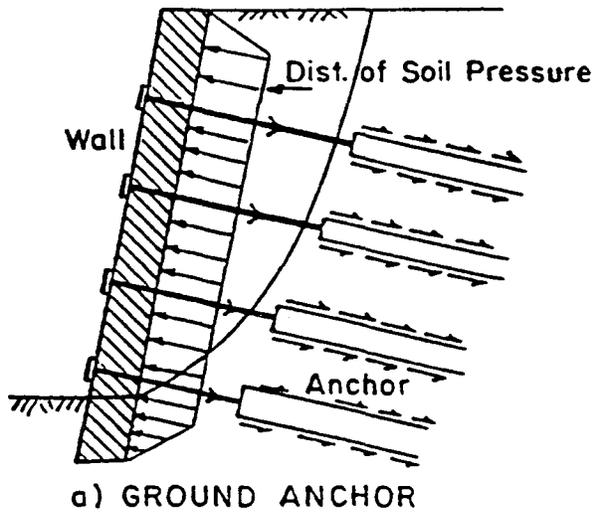


Figure 55. Transfer mechanism in ground anchors and soil nails.

- Estimate the system of forces acting on the facing (i.e. lateral earth pressure and nail forces at the connections) and design the facing for specified architectural and durability criteria.
- For permanent structures select corrosion protection relevant to site conditions.
- Select drainage system for ground water piezometric levels.

An initial reinforcement scheme can be developed using design charts developed under the FHWA "Soil Nailing" project for the evaluation of the:

- Local stability using the kinematical limit equilibrium analysis.
- "Global" stability using the Modified Davis method.⁽¹⁵⁾

Sections 6.2 and 6.3 outline the design procedures currently used. Section 6.4 provides a design example.

6.2 DESIGN PROCEDURES

a. Design Parameters

The design parameters for the composite nailed soil system include the mechanical properties of the soil and inclusions as well as parameters characterizing the different mechanisms of soil-reinforcement interaction. They can be classified in the following groups:

- Mechanical properties of the in-situ soil, specifically shear strength characteristics (i.e. internal friction angle, and cohesion).
- Geometric properties of the nails (i.e. nail diameter, shape, length and inclination) and of the structure (i.e. vertical and horizontal nail spacings, inclination of the facing and of the upper ground surface).
- Mechanical properties of the reinforcements, specifically tensile and shearing resistances and bending stiffness.
- Parameters related to the soil-reinforcement frictional interaction (i.e. the ultimate interface lateral shear strength).
- Parameters related to the normal soil-reinforcement interaction by lateral earth thrust on the reinforcement, particularly the limit passive pressure of the soil and the modulus of lateral soil reaction.

- Parameters related to the construction process (i.e. nail installation, drilling and grouting methods, facing technology, etc.).
- External load systems including surcharges, environmental loading, embankment slopes, water flow, and seepage forces.

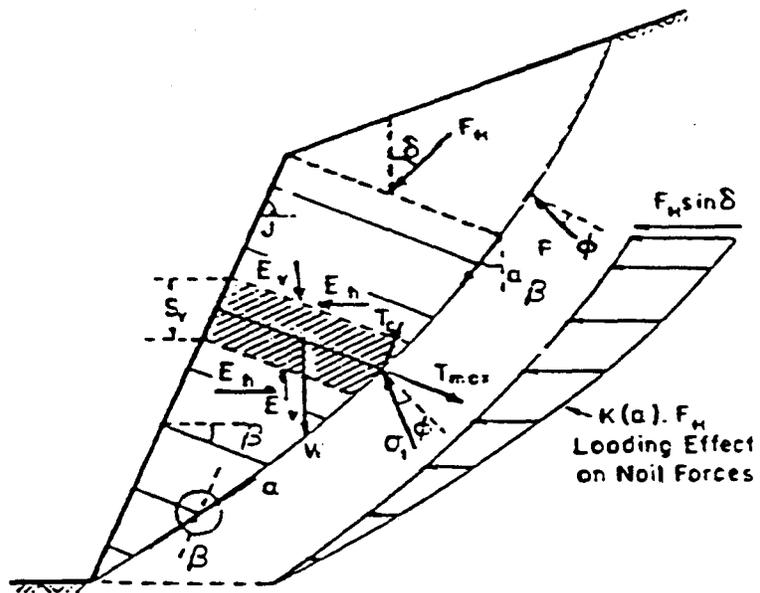
b. Estimate of Nail Forces

Kinematical limit equilibrium analysis

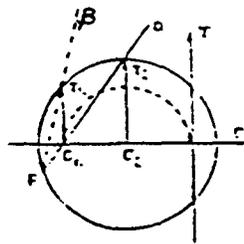
Nail forces in relatively homogenous soil strata can be computed by using the kinematical limit equilibrium analysis. This limit equilibrium analysis approach was developed for the design of nailed soil retaining structures.^(41, 42) It permits an evaluation of the effect of the main design parameters (i.e., structure geometry, inclination, spacing, and bending stiffness of nails) on the tension and shear forces generated in the nails during and after construction. The main design assumptions, shown in figure 56, are:

- Failure occurs by a quasi-rigid body rotation of the active zone which is limited by either a circular or a log-spiral failure surface.
- The locus of the maximum tension and maximum shear forces at failure coincides with the failure surface developed in the soil.
- The shearing resistance of the soil, defined by the Coulomb failure criterion, is entirely mobilized along the sliding surface.
- The shearing resistance of stiff inclusions, defined by the Tresca failure criterion, is mobilized in the direction of the sliding surface in the soil. According to the Tresca criterion, yielding at the inclusion depends only on the maximum shearing strength of the inclusion.
- The horizontal components of the interslice forces E_h acting on each slice (figure 56) are equal.
- The effect of a slope (or surcharge F_h), at the upper surface of the nailed soil mass, on the tension forces in the inclusions decreases linearly along the failure surface, as shown in figure 56a.

The effect of the bending stiffness is analyzed using a conventional "p - y" analysis procedure, considering the relatively flexible nail analagous to a laterally loaded infinitely long pile. This solution implies that at the failure



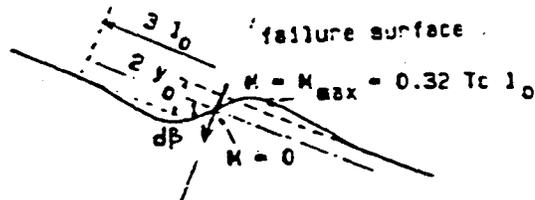
a) MECHANICS OF FAILURE AND DESIGN ASSUMPTIONS



$$T_{mcs} = \sigma_H \cdot A_S; T_c = T_N \cdot A_S$$

A_S = SECTION AREA OF NAIL

b) STATE OF STRESS IN THE INCLUSION



c) THEORETICAL SOLUTION FOR INFINITELY LONG BAR ADOPTED FOR DESIGN PURPOSES

Figure 56. Kinematical limit analysis approach.

surface, the bending moment in the nail is zero whereas the tension and shear forces are maximum. It involves a normalized bending stiffness parameter, defined as:

$$N = \frac{K_h \cdot D \cdot L_o^2}{\gamma \cdot H \cdot S_h \cdot S_v} \quad (86)$$

where:

$$L_o = \left[\frac{4 EI}{K_h D} \right]^{1/4} \quad (87)$$

is the transfer length which characterizes the relative stiffness of the nail to that of the soil. The length of the inclusion L is usually greater in practice than three times the transfer length L_o . It can therefore generally be considered as infinitely long;

D is diameter of the nail;

E and I are the elastic modulus and the moment of inertia of the nail, respectively;

K_h is the modulus of lateral soil reaction.

For design purpose, the charts shown in figure 57, as used in Soletanche practice, can be used to obtain values of K_h as a function of soil shear strength parameters.

A unique, kinematically admissible, failure surface which verifies all the equilibrium conditions of the active zone can be defined. In order to establish the geometry of this failure surface it is necessary to determine its inclination A_o with respect to the vertical at the intersection with the upper ground surface. Observations on both full-scale structures and laboratory model walls show that for the relatively flexible nails the failure surface is practically vertical at the upper part of the structure. (12, 36)

The normal soil stress along this failure surface is calculated using Kotter's equation. The maximum tension force (T_{max}) in each nail is calculated from the horizontal force equilibrium of the slice containing the nail. Analysis of the state of stress in the nail yields the ratio of the mobilized shear (T_v) to tension (T_{max}) forces as a function of the nail inclination with respect to the failure surface.

Detailed design of the nailed soil structure requires an appropriate computer code. However, for preliminary design and design evaluation in homogeneous soils, simplified yet conservative, design charts such as those prepared for the Manual

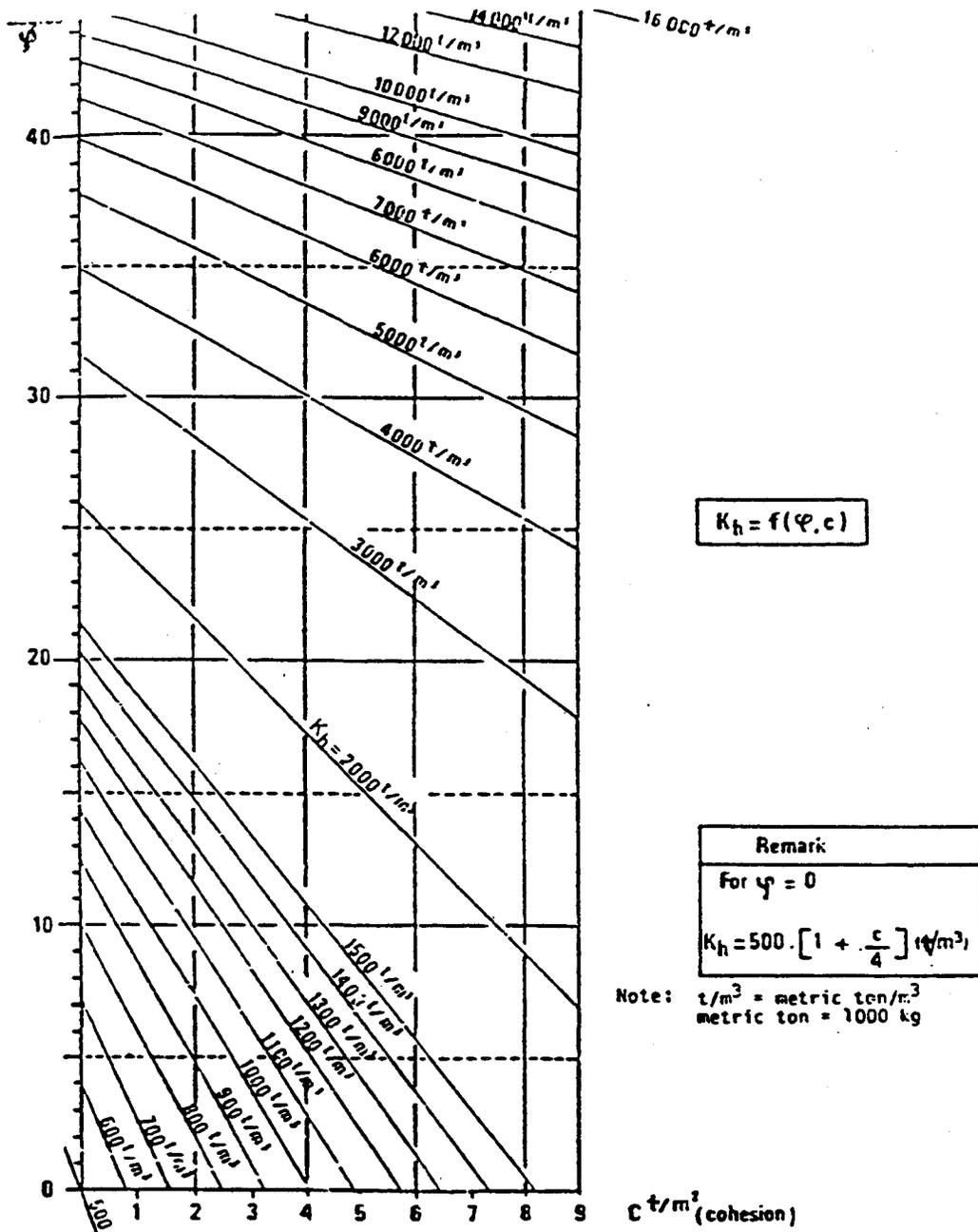


Figure 57. Horizontal subgrade reaction as a function of the soil shear parameters (according to Soletanche practice).

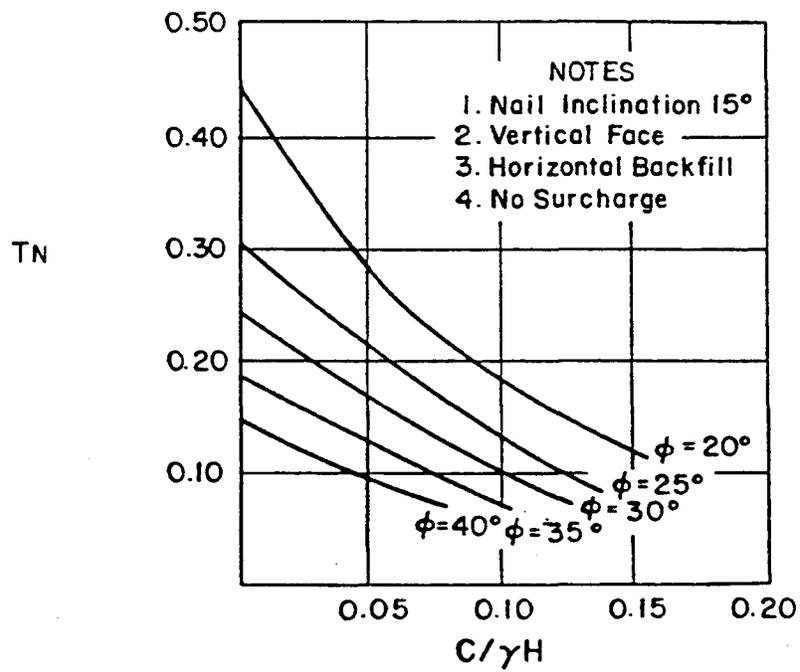
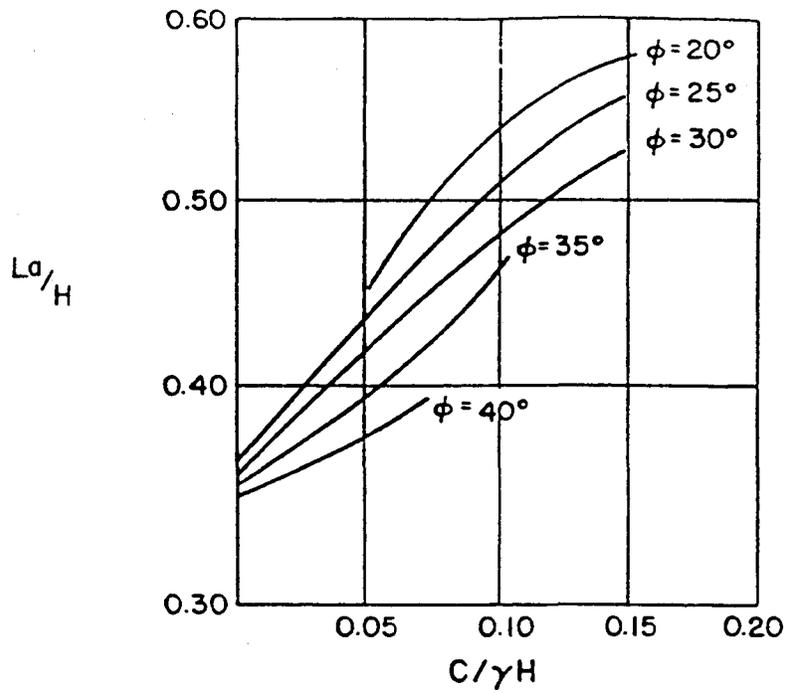


Figure 58. Design charts for perfectly flexible nails ($N=0$).

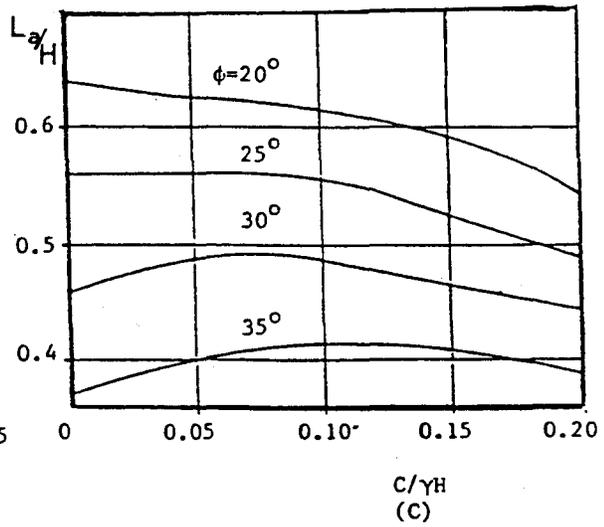
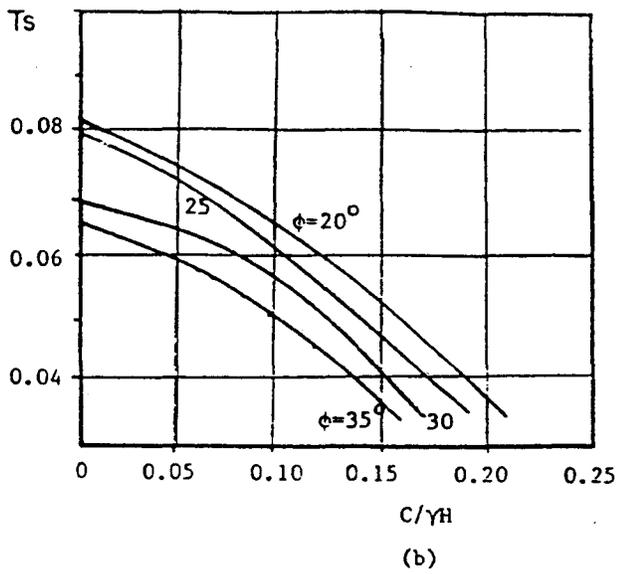
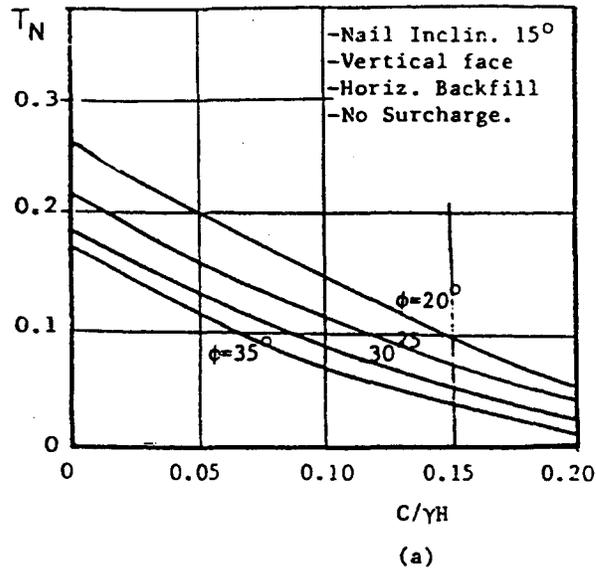


Figure 59. Design charts by kinematical method ($N = 0.33$).

of Practice for Soil Nailing may be used to evaluate the maximum values of L/H , T_N and T_s .⁽²⁵⁾ Design charts for perfectly flexible nails and for #8 rebar which are frequently used in practice are shown in figures 58 and 59, respectively. The design charts for the #8 rebar are established for $N = 0.33$ (e.g., a wall height of 39 ft (12 m) in a silty sand with a K_h of 185 lb/in³ (50,000kN/m³) and nail spacings of $S_v = S_h = 4.4$ ft (1.35 m)).⁽²⁵⁾

The kinematical limit equilibrium analysis enables the determination of three design parameters, for each reinforcement level z/H , which are to be used in the stability check. They are the length of the nail in the active zone L , the maximum tension force actually mobilized in the nail T_{max} , and the maximum shear force actually mobilized in the nail T_c . These parameters are expressed in normalized dimensionless form, as shown in figure 58:

$$T_N = \frac{T_{max}}{\gamma H \cdot S_h \cdot S_v} \quad (88)$$

$$T_s = \frac{T_c}{\gamma H \cdot S_h \cdot S_v} \quad (89)$$

Figure 60 illustrates the results for nail forces obtained for a typical nailed-soil wall with a vertical facing ($J = 90^\circ$) a horizontal ground surface, soil strength characteristics of $\phi' = 35^\circ$ and $(c'/\gamma H) = 0.05$, nail inclination of $\beta = 15^\circ$ and different values of the bending stiffness parameter N (perfectly flexible, $N=1$, and perfectly rigid, $N=10$).

Preliminary design analysis

For preliminary design in simple cases of uniform granular soil strata and a horizontal ground surface, design diagrams proposed by Terzaghi and Peck and Tschebotarioff for the design of braced excavations can be used to estimate working tensile forces generated in the nails.^(43, 44, 45) These diagrams are illustrated schematically in figure 61. Note that Terzaghi and Peck's design diagram for sands has been slightly modified in order to calculate nail forces. The maximum tension force mobilized in the nail T_{max} is expressed as a normalized, nondimensional parameter:

$$T_N = \frac{T_{max}}{\gamma H \cdot S_h \cdot S_v} \quad (90)$$

at the relative depth of z/H ,

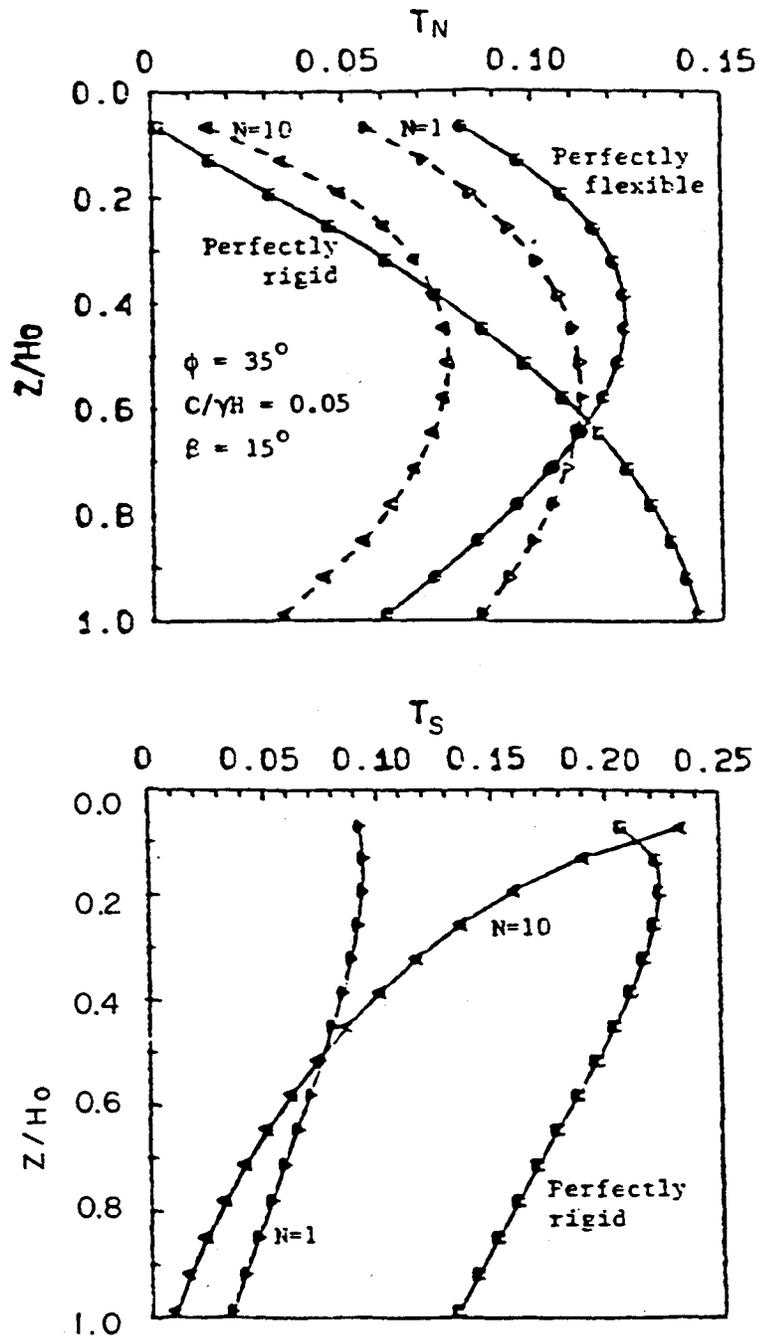
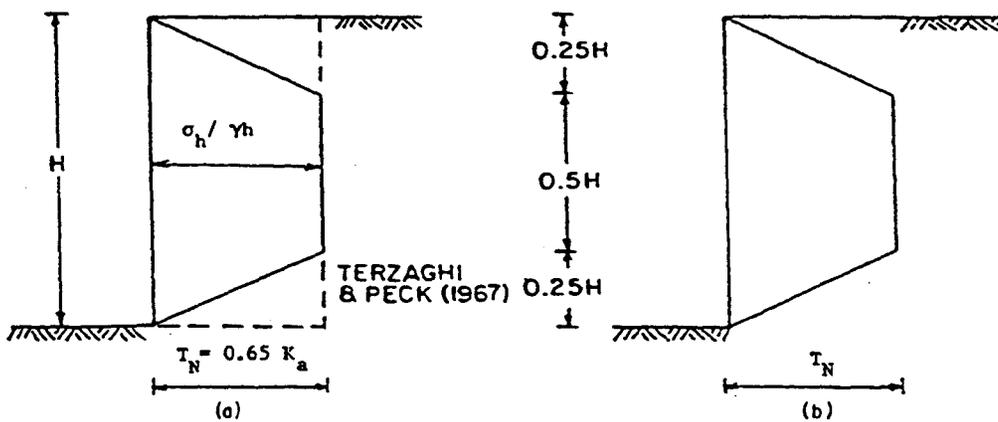


Figure 60. Variation of T_N and T_S for different relative nail rigidities.

NOTES:

- Vertical cut slope
- Horizontal upper surface.

σ_h = lateral earth pressure
 γh = overburden pressure



SAND : $(\frac{c'}{\gamma H} \leq 0.05) : T_N = 0.65 K_a$

$K_a = \tan^2 (\pi/4 - \phi/2)$

CLAYEY SAND : $T_N = K_a (1 - \frac{4c'}{\gamma H \sqrt{K_a}}) \leq 0.65 K_a$

CLAY : $T_N = 0.2 \gamma H \rightarrow 0.4 \gamma H$

Figure 61. Empirical earth pressure design diagram.

where: H is the total structure height (or excavation depth)
 S_h and S_v are, respectively the horizontal and vertical spacings between the nails.

For sands ($c'/\gamma H < 0.05$, where c' is an apparent soil cohesion):

$$T_N = 0.65 K_a \quad (91)$$

where: $K_a = \tan^2 \left(\frac{\pi}{4} - \frac{\phi'}{2} \right)$ is the Rankine active earth pressure coefficient,

For a cohesive soil with both cohesion (c') and friction angle (ϕ'):

$$T_N = K_a \left[1 - \frac{4 c'}{\gamma H} \left(\frac{1}{K_a} \right)^{0.5} \right] < 0.65 K_a \quad (92)$$

The use of the empirical earth pressure diagrams in the design of soil nailed retaining structures presents some severe limitations. In particular, these diagrams correspond to conventional cases of bracing supports with simple geometry of a vertical wall, horizontal ground surface and lateral braces. Therefore, they cannot be used to assess the effect of design parameters such as inclination of the facing, inclination and rigidity of the inclusions, surcharge, etc., on the working forces in the inclusions and structure displacements. They do not provide any estimates of the shear forces and bending moments that develop in the nails. In addition, in cohesive soils the empirical earth pressure diagram is highly sensitive to small variations in soil properties and is, therefore, difficult to use in design. Therefore, empirical diagrams are not recommended for final designs but should only be used for preliminary evaluation or a check of a final design developed by another method.

c. Evaluation of Local Stability

With the data derived from the kinematical limit equilibrium analysis the following iterative design procedure is used:

1. Select the nail type (bending stiffness - EI , allowable tension stress - F_{a11} , diameter - D , and spacings - S_v , S_h),
2. For the specified soil properties (γ , K_a , c' , ϕ'), selected nail type (EI , F_{a11}), nail inclination β , and structure height H , determine the non-dimensional parameter: L_a / H , T_N and T_s , from the design charts (figures 58 and 59) or a computer program.

3. Verify that the selected reinforcement satisfies the breakage / excessive bending failure criteria (Eqs. 93, 94, and 95):

(a) Breakage failure of the inclusion

For flexible nails which withstand only tension forces:

$$\frac{F_{all} \cdot A_s}{\gamma H \cdot S_v \cdot S_h} > T_N \quad (93)$$

where F_{all} and A_s are the allowable tensile stress and cross-sectional area of the inclusion, respectively.

For rigid nails which can withstand both tension and shear forces, considering Tresca's failure criterion:

$$\frac{F_{all} \cdot A_s}{\gamma H \cdot S_v \cdot S_h} > K_{\bullet q} \quad (94)$$

where: $K_{\bullet q} = [T_N^2 + 4 \cdot T_s^2]^{1/2}$

$$T_s = \frac{T_c}{\gamma H \cdot S_h \cdot S_v}$$

and T_c is the maximum shear force in the inclusion.

(b) Failure by excessive bending of a stiff inclusion

$$M_p > SF_m \cdot M_{max} \quad (95)$$

where: SF_m is a factor of safety with respect to plastic bending. If the allowable stress concept is used to define the tension in the reinforced zone use $SF_m = 1$, otherwise use $SF_m = 1.5$.

M_p is the plastic bending moment resistance of the nail.

For a grouted nail, an equivalent plastic bending moment resistance is calculated considering that the grout has a compressive strength f_y of 3,000 psi (21 MPa), and zero tensile strength.

The bending moment M_{max} is derived from the "p - y" analysis:

$$M_{max} = 0.32 T_c \cdot L_o, \quad \text{hence:}$$

$$\frac{M_p / L_o}{\gamma H \cdot S_v \cdot S_h} > 0.32 SF_n \cdot T_s \quad (96)$$

check stability against pullout:

for a design value of the ultimate lateral shear stress τ_{ult} , determine the resistive zone length L_r/H that satisfies the pullout failure criteria (eq. 97) with a minimum safety factor of $SF_{p.o.} = 2$ at all reinforcement levels, using

pullout failure of the inclusion:

$$\frac{T_{max}}{\pi \cdot D_g \cdot l_r} < \frac{\tau_{ult}}{SF_{p.o.}} \quad (97)$$

where: T_{max} is the maximum tensile force in the nail,

D_g is the diameter of the grouted nail

l_r is the resistive length, and

$SF_{p.o.}$ is the safety factor with respect to pullout

This design criterion implies that for a nailed cut slope, the structure geometry defined by the L/H ratio (where L is the total nail length) should be verified at each reinforcement level:

$$\frac{L}{H} > \frac{L_a}{H} + SF_{p.o.} \left[\frac{T_N}{\pi \cdot \mu} \right] \quad (98)$$

where: $T_N = \frac{T_{max}}{\gamma H \cdot S_h \cdot S_v}$

$$\mu = \frac{\tau_{ult} \cdot D_g}{\gamma \cdot S_h \cdot S_v}$$

and L_a is the nail length in the active zone

d. Evaluation of the Global Stability

This analysis consists of evaluating a global safety factor of the nailed soil retaining structure and the surrounding ground with respect to a rotational or translational failure along potential sliding surfaces. It requires determination of the critical sliding surface which may be dictated by the stratification of the subsurface soil, or, in rocks, by an existing system of joints and discontinuities. The potential sliding surface can be located totally inside or outside the soil nailed retaining structure, or partially inside and partially outside the nailed zone.

Evaluation of the global safety factor is generally based on the application of limit equilibrium methods. Slope stability analysis procedures have been developed to account for the available pullout resistance, tension, and shearing resistance of the inclusions crossing the potential sliding surfaces. The limit equilibrium methods commonly used are outlined in the Soil Nailing section of volume II. ^(11, 12, 46)

The Davis method is recommended for global stability analysis, because of its simplicity and availability in the public domain. ⁽⁵³⁾ In the original version of the Davis method, pullout resistance of the nails is estimated using Coulomb's failure criterion to calculate the interface lateral shear stress. This approach was found to be restrictive and not consistent with currently available pullout results. Moreover, nail section and length are assumed to be uniform. To overcome these limitations, the original method developed has been extended by Elias and Juran to permit the consideration of: ⁽¹⁵⁾

- Input interface limit lateral shear force per unit length of nail obtained from pullout tests.
- Input design data per nail (i.e. length, section, tension strength, grouted nail diameter).
- Facing inclination.
- Embankment slope at the top of the wall.

The safety factor currently used in the Davis method, is defined by eq. 99:

$$SF = SF_c = SF_\phi = SF_1 \quad (99)$$

where SF_c and SF_ϕ are factors of safety with respect to shear strength parameters of soil, and SF_1 is the factor of safety with respect to the ultimate interface lateral shear stress.

$$\begin{aligned} SF_c &= c/c_m \\ SF_\phi &= \tan \phi / \tan \phi_m \\ SF_1 &= \tau_{ult} / \tau_m \end{aligned}$$

(τ_{ult} and τ_m are respectively the ultimate lateral interface shear stress and the mobilized lateral shear stress)
 c_m and ϕ_m are the soil cohesion and internal friction angle actually mobilized along the critical potential failure surface.

When eq. 99 is used, the actual shear strength parameters of the soil should be used and the global factor of safety should be at least 1.5. It has been observed that this design procedure overestimates the safety factor with respect to the shear strength parameters of the soil under working stress conditions. A more rational design procedure is to assume the shear resistance of the soil is fully mobilized along the potential failure surface. Hence, the Davis method is modified by Elias and Juran to permit consideration of the factor of safety defined by: ⁽¹⁵⁾

$$\frac{SF_c}{SF_\phi} = \frac{SF_\phi}{SF_c} = 1 \quad (100)$$

For this definition of the factor of safety, the residual shear strength parameters of the soil (c and ϕ) should be factored by 1.25 as recommended by Gassler and Gudehus. ⁽³⁸⁾ The global factor of safety as defined by eq. 100 should be at least 2. This definition of the factor of safety more appropriately represents the uncertainty associated with each of the design parameters.

For design of more complex structures, i.e., in stratified soils, with groundwater flow, or in cases of mixed structures (e.g. combining ground anchors and soil nailing) the French method using a proprietary computer program, can be used. ⁽¹²⁾ The French method as outlined in the Soil Nailing section of volume II should be used with actual shear strength parameters, and the safety factor, as defined by eq. 99, should be at least 1.5.

6.3 FACING

For design purpose the concrete facing elements (shotcrete, cast in place concrete or prefabricated panels) are considered analogous to a beam or raft, of a unit width equal to the nail spacing (S_v or S_h), supported by the nails. The nail forces at the connections are assumed to be equal to the maximum nail forces, calculated as in section 6.2.b. The facing design follows conventional ACI structural design procedures using either working stress or ultimate strength methods.

For the recommended minimum concrete strength of 4,500 psi (31 MPa) and reinforcing steel mesh, yield strength of $f_y = 60$ ksi (413 MPa), the required facing thickness h is given (in in.) by:

$$h = [1.39 \cdot M_{max}]^{0.5} \quad (101)$$

where: M_{max} is the maximum moment, which is given by:

considering a beam of a section width $L = S_v$ or S_h ,

$$\text{at each nail: } M_{\max} = T_{\max} \cdot L / 12 \quad (102)$$

$$\text{between nails: } M_{\max} = T_{\max} \cdot L / 24 \quad (103)$$

considering a raft of a unit width $L = S_v$ or S_h ,

$$\text{at each nail zone: } M_{\max} = 0.70 \cdot T_{\max} \cdot L / 12 \quad (104)$$

$$\text{in between nail zone: } M_{\max} = 0.57 \cdot T_{\max} \cdot L / 24 \quad (105)$$

Following the ACI design procedure, the required steel section A_s (in² per unit ft of facing) is given by:

$$M_{\text{ult}} = 1.4 \cdot M_{\max} = 0.9 \cdot A_s \cdot f_y \cdot [d - 0.65 \cdot A_s] \quad (106)$$

where d is the shotcrete thickness (in in.) from the steel centerline to the furthest edge of the raft.

For permanent applications, the shotcrete thickness will vary from 4 to 7 in (100 to 180 mm) with two layers of steel meshes required to satisfy both positive and negative moments.

6.4 DESIGN EXAMPLE

Given:

Soil Characteristics:

$$\phi = 35^\circ$$

$$\frac{c}{\gamma \cdot H} = 0.05; \quad \gamma = 19 \text{ kN/m}^3 \quad (121 \text{ lb/ft}^3)$$

$$T_{\text{ult}} = 120 \text{ N/m}^2; \quad K_h = 50,000 \text{ kN/m}^3 \quad (159 \text{ ton/ft}^3)$$

Nail Type = #8 Rebar

$$\text{Yield Strength of Steel} = f_y^s = 420 \text{ MPa} \quad (60 \text{ kip/in}^2)$$

$$\text{Compressive Strength of Concrete} = f_y^c = 21 \text{ MPa} \quad (3 \text{ kip/in}^2)$$

$$F_{s11} = 168 \text{ MPa} \quad (24,000 \text{ lb/in}^2)$$

$$D_b = 0.0254 \text{ m} \quad (1 \text{ in})$$

$$D_g = 0.1016 \text{ m} \quad (4 \text{ in})$$

$$EI = 4 \text{ kN} \cdot \text{m}^2$$

$$M_p = (420,000) (0.4244) \pi \cdot (0.0127)^3 + (0.5) (0.4244) \pi \cdot (0/0508^2 - 0.0127^3) (21,000)$$

$$M_p = 2.95 \text{ kN} \cdot \text{m}$$

$$l_o = 0.335 \text{ m}$$

Structure Geometry:

$$H = 12.0 \text{ m}$$

$$S_v = S_H = 1.35 \text{ m}$$

$$\beta = 15^\circ$$

I. Check for Local Stability

Using design charts (Figure 59)

$$T_s = 0.0589 \quad T_N = 0.13$$

$$\frac{L_a}{H} = 0.4$$

$$K_{eq} = \sqrt{T_N^2 + 4T_s^2} = 0.175$$

a) Breakage failure criteria:

$$\frac{F_{all} \cdot A_s}{\gamma \cdot H \cdot S_v \cdot S_H} = \frac{(168,000 \text{ kN/m}^2) (0.00051 \text{ m}^2)}{(19 \text{ KN/M}^3)(12 \text{ m})(1.35)^2} = 0.2022$$

$$> K_{eq \text{ max}} = 0.175$$

b) Excessive bending failure criteria: ($SF_m = 1.0$)

$$\frac{M_p / I_o}{\gamma \cdot H \cdot S_v \cdot S_H} = \frac{(2.95 / 0.335)}{(19)(12)(1.35)^2} = 0.0212 > (0.32 SF_m \cdot T_s =$$

$$0.32 \times 1.0 \times 0.0589 = 0.0188$$

c) Determination of nail length to satisfy pullout failure criteria:

$$\left(\frac{L}{H}\right) = \left(\frac{L_a}{H}\right) + \left(\frac{T_N}{\pi} \cdot \frac{SF_{Po}}{\mu}\right), \quad SF_{Po} = 2.0$$

$$\mu = \frac{\tau_{ult} \cdot D_g}{\gamma \cdot S_v \cdot S_H} = 0.352$$

Therefore, $\frac{L}{H} = 0.4 + \frac{0.13 \cdot 2}{\pi \cdot 0.352} = 0.63$

i.e. required $L = 7.6$ in

The evaluation of the local stability of each nail with respect to pullout yields the required nail length as follows:

No.	Z(m)	Z/H	L_a/H	T_N	T_S	μ	L/H	$K_{eq} = \sqrt{T_N^2 + 4 T_S^2}$
1	0.675	0.0563	0.3695	0.0653	0.054	0.352	0.4876	0.126
2	2.025	0.1688	0.3429	0.0958	0.0584	0.352	0.5161	0.151
3	3.375	0.2813	0.3112	0.1169	0.0589	0.352	0.5226	0.166
4	4.725	0.3938	0.2746	0.1306	0.0571	0.352	0.5108	0.173
5	6.075	0.5063	0.2332	0.1382	0.0541	0.352	0.4831	0.175
6	7.425	0.6188	0.1870	0.1407	0.0503	0.352	0.4415	0.173
7	8.775	0.7313	0.1360	0.1391	0.0461	0.352	0.3875	0.166
8	10.125	0.8438	0.0799	0.1338	0.04147	0.352	0.3219	0.157
9	11.475	0.9563	0.0277	0.1269	0.03735	0.352	0.2572	0.147

This level-by-level design yields the min L/H of 0.526, i.e. $L = 6.3$ in.

II. Check for Global Stability

SOIL NAILING: DESIGN EXAMPLE (COMPUTER GENERATED ANALYSIS)⁽¹⁵⁾

UNIT WT = 19.000 kN/m³

NAIL INCLINATION ANGLE = 15.00 degrees

INCLINATION OF CUT FACE FROM VERTICAL (ALPHA) = 0.000

BACKFILL ANGLE (BETA) = 0.000

STRESS RATIO = 0.05

COHESION = 11.400 kPa

FRICTION ANGLE = 35.00 degrees

DIAMETER OF NAIL = 0.150 m

VERTICAL SPACING OF NAIL = 1.350 m

HORIZONTAL SPACING OF NAIL = 1.350 m

*** INPUT FACTOR OF SAFETY FOR COHESION AND FRICTION = 1.00 ***

LENGTH OF NAIL = 5.500 m

HEIGHT OF WALL = 11.400 m

AMAX = 1,350

SURCHARGE = 0.000

NAIL LEVEL = 1

DEPTH = 0.68 m

LENGTH OF NAIL = 6.20 m

AREA OF REINFORCEMENT BAR = 0.0005000

YIELD STRENGTH = .4200E+06

NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 2 DEPTH = 2.00 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 m² YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 3 DEPTH = 3.38 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 m² YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 4 DEPTH = 4.72 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 m² YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 5 DEPTH = 6.00 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 6 DEPTH = 7.43 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 m² YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 7 DEPTH = 8.78 m LENGTH OF NAIL = 6.20 m
AREA OF REINFORCEMENT BAR = .0005000 m² YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

NAIL LEVEL = 8 DEPTH = 10.13 m LENGTH OF NAIL = 6.20
AREA OF REINFORCEMENT BAR = .0005000 YIELD STRENGTH = .4200E+06
NAIL PULL-OUT RESISTANCE (PER UNIT AREA) = 120.000

*** MINIMUM FACTOR OF SAFETY AGAINST PARABOLIC SLIP FAILURE = 1.95

* OUTPUT OF PROGRAM SONAIL :

DATA ENTRY :

1 - Soil Angle of Internal Friction. [PHI] = 35.000
2 - Inclination of Vert. Wall with Horiz. [J] = 30.000
3 - Inclination of Embankment with Horiz. [GAMMA] = 0.000
4 - Inclination of Nails with Horiz. [BETA] = 15.000
5 - Angle of Failure at top. [AO] = 8.000

- 6 - Increment of Angle A [INCR] = 1.000
7 - Start at Angle of Failure at Bottom. [AF+PHI] = 68.000
8 - Stop at Angle of Failure at Bottom. [AF+PHI] = 68.000
9 - (Soil Cohesion/gamma*H) ratio. [CGH] = 0.050
10 - Type of Reinforcement. [CASE] = 3
Case 1: Flexible Reinforcement [Tc= 0.].
Case 2: Reinforced Does not withstand compression.
Case 3: Compression is superpositioned on the Reinf.
11 - The Ratio (Ks.B/Gamma.H) = 0.330
12 - The Ratio (Sv.Sh/Lo**2.) = 1.000

N	Z/H _o	L _a /H _o	T _{max}	T _c	K _{oq}	
47.00	.0401239	.3729840	-2.8022	.0602094	.0529835	.1218777
48.00	.0807722	.3642336	-3.6302	.0729684	.0556255	.1330458
49.00	.1219391	.3546798	-4.1744	.0844446	.0573763	.1424748
50.00	.1636185	.3443072	-4.5050	.0946893	.0584454	.1504310
51.00	.2058040	.3331000	-4.6684	.1037564	.0589756	.1570919
52.00	.2484886	.3210427	-4.6975	.1117003	.0590703	.1625858
53.00	.2916652	.3081201	-4.6162	.1185741	.0588072	.1670118
54.00	.3353261	.2943163	-4.4434	.1244290	.0592462	.1704495
55.00	.3794631	.2796162	-4.1925	.1293139	.0574351	.1729660
56.00	.4240678	.2640043	-3.8751	.1332752	.0564123	.1746187
57.00	.4691313	.2474651	-3.5002	.1363574	.0552090	.1754579
58.00	.5146441	.2299835	-3.0752	.1386016	.538518	.1755286
59.00	.5605966	.2115440	-2.6062	.1400475	.0523602	.1748711
60.00	.6069784	.1921213	-2.0936	.1497320	.0507555	.1735228
61.00	.6537789	.1717306	-1.5562	.1406901	.0490510	.1715159
62.00	.7000869	.1503264	-0.9831	.1399547	.0472804	.1688881
63.00	.7385908	.1279040	-0.3826	.1305572	.0453947	.1656527
64.00	.7965786	.1044483	0.2426	.1365269	.0434637	.1613517
65.00	.8449376	.0799447	0.8901	.1338918	.0414758	.1575055
66.00	.8936549	.0543785	1.5574	.1306786	.0394382	.1526381
67.00	.9427168	.0277350	2.2429	.1269124	.0373573	.1472719
68.00	.9921095	.0000000	2.9446	.1226175	.0352387	.1414288

Sum of T_{max} is 2.6599283

AF+PHI	AB+PHI	H /R	MWT	MTmax	MTC	MC	MTQT
68.00	46.00	.5765	.07274	.04262	.00990	.0	.00278

CHAPTER 7

CONSTRUCTION AND FIELD OBSERVATION

7.1 GENERAL

Construction of reinforced soil 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 installation of the facing elements (tensioning of the reinforcement may also be required). Special skills or equipment are usually not required, and locally available labor can usually be used. Most proprietary vendors provide training for construction of their systems.

In-situ reinforcement construction is equally simple, consisting essentially of installing nails or micropiles by driving or pre-augering and grouting, then attaching a facing element (either precast concrete or wire mesh and shotcrete gunite) to prevent surface sloughing. In-situ reinforcement does require some special installation equipment.

Ease of construction, without the need for specialist workmen, is generally a primary advantage of reinforced soil systems over conventional gravity wall systems. However, 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 the specific site conditions, to the backfill materials available for construction of fill type systems, and to facing requirements. The requirements for each of these considerations with regard to the various types of reinforcement systems are reviewed in this chapter as follows:

- Section 7.2 provides a review of general requirements for field observation of any system.
- Section 7.3 includes the construction requirements for reinforced fill systems with precast facing panels.
- Section 7.4 considers anchored type system construction.
- Section 7.5 covers reinforced fill wall and slope construction with wrapped type facing.
- Sections 7.6 reviews the construction of in-situ reinforcement by soil nailing.

7.2 REQUIREMENTS FOR FIELD OBSERVATION

Prior to erection of the structure, the designer and personnel responsible for observing the field construction of the retaining structure should become thoroughly familiar with the following items:

- The plans and specifications.
- The site conditions relevant to construction requirements.
- Material requirements.
- Construction sequences for the specific reinforcement system.

a. Plans and Specifications

Specification requirements for reinforced soil systems are reviewed in chapter 8. The owner's field representatives should very 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, and details for drainage requirements and utility construction. The contractor's documents should be checked to make sure that the latest issue of the approved for construction 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. The engineer should be contacted immediately if unanticipated conditions are encountered.

Site accessibility will be required for construction equipment, including loaded dump trucks, front-end loaders, vibratory roller compaction equipment, and a small crane for facing erection. On-site storage will be required for reinforcement facing panels, reinforcement material, and possibly backfill materials.

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 reinforced soil 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.

Reinforced soil 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 near 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 which 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 to prevent water exiting the face of the wall and causing erosion and/or face stains. (Most precast facing panel systems are very permeable.) Drainage of a less permeable fill can be attained by installing a drainage system at the back and base of the wall or by alternating the less porous fill with layers of free draining materials. These porous layers should be connected to a zone of porous material, drainage panels or vertical drains placed behind the wall face. 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 critical planes of weakness within the structure with drainage layers.

In certain situations, vertical or inclined zones of drainage material can be provided behind the reinforced soil mass to collect and drain seepage water. Such systems should be installed prior to placement of any reinforced backfill, unless they are within the reinforced soil volume.

c. Material Requirements⁽¹³⁾

Material components should be examined at the casting yard (for systems with precast panels) and on site. Material acceptance should be based on a combination of material testing, certification, and visual observations.

When delivered to the project site, the inspector should carefully inspect all material (precast facing elements, reinforcing elements, bearing pads, facing joint materials and reinforced backfill). On site, all system components should be satisfactorily stored and handled to avoid damage. The material supplier's erection manual should contain additional information on this matter.

Precast 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, facing panels should be cast on a flat surface. Especially important, to minimize corrosion, is that coil embeds, tie strip guides and other connection devices should not be in contact with or be attached to the facing element reinforcing steel (see chapter 2 on system durability).

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 (minimum recommended 4,000 psi [27.6 MPa]).
- 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 recommended:

- Overall dimensions - 1/2 in (12.7 mm).

- Connection device locations - 1 in (25.4 mm).
- Element squareness - 1/2 in (12.7 mm) difference between diagonals.
- Surface finish - 1/8 inch in 5 ft (2.1 mm in 1 m) (smooth surface).
- Surface finish - 5/16 inch in 5 ft (5.2 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.

Reinforcing Elements - Reinforcing materials (strips, mesh, sheets) should arrive at the project site securely bundled or packaged to avoid damage. They come in a variety of material 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 or engineer for verification testing.

Protective coatings, i.e. galvanization (thickness 2 oz/ft² [610 gm/m²]) or epoxy (thickness 18 mils [457 μm]) should be verified by certification or agency conducted tests and be checked for defects.

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.

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 wall distress. With walls having segmental concrete panel facings, for example, walls over 40 ft (12.2 m) in height, walls subjected to large surcharge loads, and walls on very compressible foundations, panel joint design and joint material become a critical performance factor.

Reinforced Backfill - The backfill in reinforced soil 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 as discussed in chapter 2. The inspector must check that the reinforced backfill strictly conforms to the requirements of the specification. Depending on the owner/agency's procedures, backfill tests may be performed by either the contractor or the owner. In either case, the results of these tests, and periodic tests conducted during the work for quality assurance, should be the basis for approving the backfill materials. During construction, gradation and plasticity index testing of the reinforced backfill should be performed, at least once for every 2,000 yd³ (1,530 m³) of material placed, and whenever the appearance or behavior of the material noticeably changes. For example, additional tests should be immediately performed if either excessive panel movement or backfill pumping occurs during construction.

Special design and construction considerations are necessary on projects where poorer quality natural materials, lightweight fills and waste materials, such as fly ash, are being used. The unit weight, permeability, shear strength, soundness, and electrochemical properties must be thoroughly investigated before these materials are considered acceptable.

d. Construction Sequence

The general construction steps for the different reinforcement systems are covered separately in sections 7.3 through 7.6. For each specific type of reinforcement there are specific requirements related to each aspect of construction. These should be included in the specifications and are also usually contained in the literature of proprietary systems. The system suppliers generally provide some degree of technical assistance for construction and correction of construction problems. Most suppliers will also provide an individual on site to advise the contractor as to correct construction procedures, though these technical advisors will not generally be on site full time. However, they should be on site roughly two or three days initially and periodically thereafter, depending on the contractors' previous experience with the system. In many public agencies, additional sources of technical assistance are personnel in the central office geotechnical or foundation unit who have had previous experience with similar construction projects. Consultants involved in the project may also have special knowledge of construction considerations.

7.3 CONSTRUCTION OF REINFORCED FILL SYSTEMS WITH PRECAST FACING ELEMENTS

The construction of a multilayered soil reinforcement system with precast facing elements is carried out in the following steps:

1. Preparation of subgrade, which 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, if required (see previous section 7.2.a).

2. Placement of a leveling pad for the erection of the facing elements (see section 7.3.a).
3. Erection of the first row of facing panels on the prepared leveling pad. The first row facing panels may be full or half height panels, depending upon the type of facing utilized. The first tier of panels must be shored up to maintain stability and alignment (see section 7.3.b).
4. Placement 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 (see section 7.3.c).
5. Placement of the first layer of reinforcing elements on the backfill (see section 7.3.d).
6. Placement of the backfill over the reinforcing elements to the level of the next reinforcement layer and compaction of the backfill. Steps 4 and 5 are repeated for each successive layer (section 7.3.e).

a. Leveling Pad

A cast-in-place or precast concrete leveling pad should be placed at the foundation elevation for all reinforced fill structures with precast facing elements. The purpose of the pad is to serve as a guide for facing panel erection and not to act as a structural foundation support. The pad should have minimum dimensions of 6 in (12.7 mm) thick by 12 in (30.5 mm) wide and should have a minimum 2,000 psi (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 1/8 in (3.2 mm) to the design elevation is recommended. If the leveling pad is not at the correct elevation, then the top of 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.

b. Erection of Facing Panels

Facings may consist of either precast concrete panels, or metal facing panels or fully flexible wrap type facings including welded wire mesh, geotextile or geogrid.

The erection of segmental facing panels and placement of the soil backfill proceed simultaneously. There are some differences in the construction procedures for walls with rigid precast concrete full height facing panels, walls with partial height, segmental precast concrete or metal panels that can articulate at the horizontal joints between adjacent panels and those with full flexible wrap type facings. Flexible facings are covered in section 7.5.

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. 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 which have segmental precast panels also have some form of construction alignment dowels which aid in proper erection. Typical backward batter for segmental precast panels is 1/4 in per foot (20.8 mm per meter) of panel height.

Full height precast panels 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. 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 and 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.

- Most reinforced fill systems will 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

A key to good performance is consistent compaction. Wall fill lift thickness must be controlled based on specification requirements and vertical distribution of reinforcement elements. The uniform loose lift thickness of the reinforced backfill should not exceed 12 in (305 mm).

Moisture and density control is imperative. Even when using high quality granular materials, problems can occur if compaction control is not exercised. 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 3/4 in (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. 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.

Compaction Equipment -- With the exception of the 3 ft (0.91 m) zone directly behind the facing elements or slope face, large, smooth drum, vibratory rollers should generally be used to obtain

the desired compaction. Sheepsfoot rollers should not be permitted because of possible damage to the reinforcements. When compacting uniform medium to fine sands (in excess of 60% 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 3-ft (0.91-m) of the wall or slope face, use small, single or double drum, walk behind vibratory rollers or vibratory plate compactors. 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 3 ft (0.91 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 under-compaction due to 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 2 feet of wall height for every 100 linear ft (30 m) of wall is recommended.

d. Placement of Reinforcing Elements

Reinforcing elements should be installed in strict compliance with the spacing and length requirements shown on the plans. Reinforcements should generally be placed perpendicular to the back of the facing panel. In specific limited 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 3 in (76 mm) minimum thickness of wall fill. Under no circumstances should adjacent back-to-back walls be connected to the same reinforcing element.

Curved walls create special problems with 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 mechanically stabilized embankment structures around deep foundation elements or drainage structures. These are project specific details where good construction workmanship should be stressed. Several options are available with regard to deep foundations: drive piles prior to wall erection 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 reinforced fill system has a unique facing connection detail. All connections must be made in accordance with the manufacturer's recommendations. For example, on Reinforced Earth structures: bolts must fit up through and strips must be located between both tie strip flanges; bolts must be perpendicular to the steel surfaces; and bolts must 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 have to be pretensioned a sufficient amount 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.

The following alignment tolerances are recommended:

- Adjacent facing panel joint gaps (all reinforcements) - $3/4$ in \pm $1/4$ in (19 mm \pm 6.3 mm).
- Precast face panel (all reinforcements) - $3/4$ in in 10 ft (6.25 mm per m) (horizontal and vertical directions).
- Wrapped face walls (e.g. welded wire or geosynthetic facing) - 2 in in 10 ft (16.7 mm per m) (horizontal and vertical directions).
- Wrapped face walls (e.g. welded wire or geosynthetic facing) overall vertical - 1 in in 10 ft (8.3 mm per m)

- Reinforcement placement elevations - ± 1 in (25.4 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 which 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 wall fill and reinforcing elements, followed by the resetting of the panels. Decisions to reject structure sections which are out of alignment should be made rapidly, because panel resetting and wall fill handling are very 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 a freeze-thaw cycle. Construction should be stopped immediately and the situation evaluated by qualified geotechnical personnel 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 prevent erosion of fill through the facing joints (synthetic foam and geotextiles details are typically used). Excessively large panel joint spacings or joint openings which are highly variable result in a very unattractive end product.

Wooden wedges 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.

Once again, the timeliness of alignment monitoring should be stressed. The following specification language has been used very effectively, "plumbness and tolerances of each facing element shall be checked prior to erection of the next panel level. Should any facing elements be out of alignment, the fill should be removed and the elements reset to proper tolerances."

The wall ends (lateral limits of construction) should be checked to make sure that they are properly embedded to avoid soil erosion around the structure. This design detail must often be set by construction personnel as the wall construction proceeds.

Care should be exercised to check that all required drainage elements are properly installed behind and beneath the structure as shown on the plans.

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.

The completion of wall construction must sometimes be coordinated with related construction features: for example, the casting of concrete barriers on walls where post construction settlements are anticipated must be delayed until the majority of settlement has occurred. This problem would hopefully be identified during design and addressed by a special note.

Table 12 gives a summary of several out-of-tolerance conditions and their possible causes.

7.4 ANCHORED SOIL SYSTEMS

Construction of reinforced soil structures using anchored systems proceeds in a series of successive stages similarly to those for multilayer reinforced fill structure. Each stage consists of the assembly of a new layer of facing elements, placement of the corresponding earth fill, compaction of the fill, placement of a layer of reinforcement, placement of the anchorage element.

The first step in the construction involves subgrade preparation similar to any reinforced soil system. It includes removal of any undesirable soils from the area to be occupied by the reinforced soil structure. The next step is to construct a leveling pad of concrete to serve as a construction guide, as detailed in section 7.3.a. The leveling pad is used for the erection of the facing panels, which in existing anchor systems consist of precast concrete panels. The first tier of these facing elements must be secured by shoring or bracing, as their stability depends on the resistance provided by the anchors which are not installed until after the first lift of fill is placed.

After erecting the first tier of panels, which may consist of half panels in the case of the Anchored Earth wall or the American Geo-Tech wall, backfill is placed to a level equal to the first level of reinforcement. At the same time, the deadman rock wall would be built up to that level in the case of the TRES system. The backfill is compacted in layers.

Table 12. Out-of-tolerance conditions and possible causes.

Reinforced soil structures are to be erected in strict compliance with the structural and aesthetic requirements of the plans, specifications, and contract documents. The desired results can generally be achieved through the use of quality materials, correct construction/erection procedures, and proper inspection. However, there may be occasions when dimensional tolerances and/or aesthetic limits are exceeded. Corrective measures should quickly be taken to bring the work within acceptable limits.

Presented below are several out-of-tolerance conditions and their possible causes.

<u>CONDITION</u>	<u>POSSIBLE CAUSE</u>
1. Distress in wall: a. Differential settlement or low spot in wall. b. Overall wall leaning beyond vertical alignment tolerance. c. Panel contact, resulting in spalling/chipping.	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 chipping.	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. Large backfill placing and/or compaction equipment working within 3 feet zone of back of wall facing panels. c. Backfill material placed wet of optimum moisture content. Backfill contains excessive fine materials (beyond the specifications for percent of materials passing a No. 200 sieve). d. Backfill material pushed against back of facing panel before being 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.
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.
5. Wall out of horizontal alignment tolerance, or bulging.	5.a. See Causes 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. Subsequent panels are spalling or chipping.	6.a. Panels are not level. Differential settlement (see Cause 1a). 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. d. See Cause 3i.

After the fill has been placed to the level of the reinforcement layer, the first level of reinforcing elements are set and loosely secured to the facing panels. The deadman block used in the American Geo-Tech System is placed at the specified location level, which may be slightly lower than the level of the reinforcing rod. All backfill requires compaction to the specified density.

The next step is to place the next tier of facing panels and the next lift of fill over the reinforcement and over and around the anchorage element. Flexible joint material is placed between the various panels as they are erected as with multilayered reinforced fills.

The construction proceeds by repeating the above steps of erecting the panels, placing the reinforcing elements, and the deadman, and the backfill materials until the top of the wall is reached. At that time, the top elements or a coping is placed on the wall and the backfill finished to design grade.

Internal Drainage

The various proprietary systems have different types of provisions for internal drainage. The use of free draining material in the backfill in systems with drainage outlets insures good drainage of the backfill and thus provides prevention of hydrostatic pressures on the facing elements. Drainage from the retained mass may be by natural means such as gravity flow towards a low place or by artificial means.

The most important consideration is permeability of the backfill. Proper gradation of the material and appropriate filters at transitions between the different types of soils are essential and should be provided as the backfilling proceeds. If pipelines are installed for collection of any seepage water, they should have a proper slope for drainage. The size of holes or slots in the drain pipes should be compatible with the gradation of the soil retained. The designer should verify that placement of drainage layers and materials (especially geotextiles) in the reinforced structure does not provide unanticipated weak zones for potential soil failure.

Construction specifications should indicate sizes of filter materials and drain pipes and these must be followed during the construction. If weep holes are provided on the facing element for drainage of seepage water, they should not be blocked during construction, and necessary precautionary maintenance must be done to insure their long term operation.

7.5 CONSTRUCTION OF REINFORCED FILL WALL AND SLOPE SYSTEMS WITH FLEXIBLE FACINGS

Construction of flexible faced reinforced fill retaining walls or slopes, 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 only a level grade is required for the erection of the starting facing element. A concrete footing or leveling pad is not usually required unless precast elements are to be attached to the system after construction. Reinforced embankment slopes will only require a facing if the slope angle is greater than the angle of repose of the fill or to prevent erosion. In situations where a facing is not used, the slope face is vegetated to provide long-term stability and erosion control. In these cases, an erosion control mesh or geotextile may be placed on the slope face during or after construction of the slope to provide short term erosion protection. The slope should be seeded as soon as practically possible. When a facing system is used, construction of the face proceeds in exactly the same way as for wall construction.

Construction again proceeds in a series of steps involving placement of reinforcement and soil fill.

Following is the usual construction sequence:

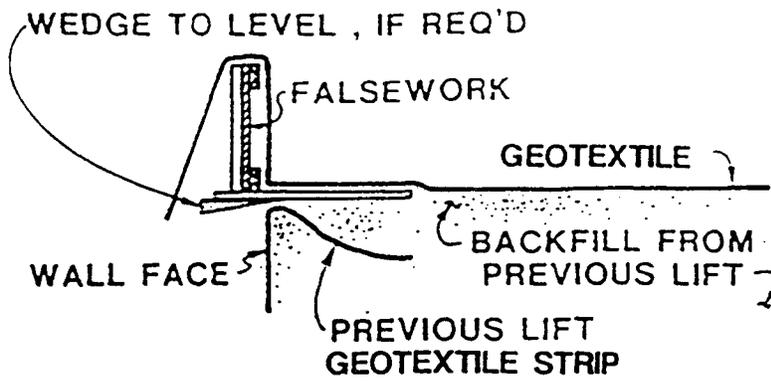
1. Site preparation:

- Clear and grub site and make any required excavations. (Remove all slide debris if a slope reconstruction project).
- Prepare level subgrade for a width equal to the length of reinforcement plus 1 ft (305 mm) for placement of first level of reinforcing. The excavated area should be carefully inspected, and any loose or soft foundation soils should be compacted or excavated and replaced with compacted backfill material.
- Proofroll subgrade at the base of the wall or slope with roller or rubber tired vehicle.
- If precast facing units are to be attached after construction of the wall or slope, a concrete leveling pad as described in section 7.3.a should be constructed.

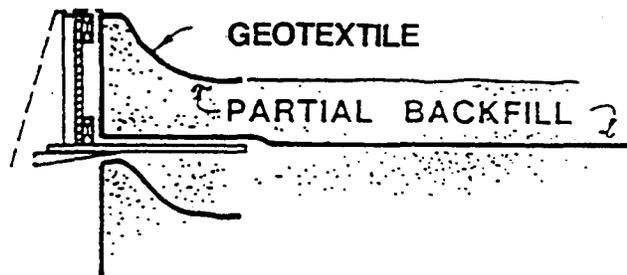
2. Place the first reinforcing layer.

- Reinforcement with anisotropic strength properties (i.e. many geosynthetics) should be placed with the principal strength direction perpendicular to face of slope. It is often most convenient to unroll the reinforcement with the roll or machine direction parallel to the face, especially for slopes not requiring a facing. If this is done, then the cross machine tensile strength must be greater than the design tension requirements.

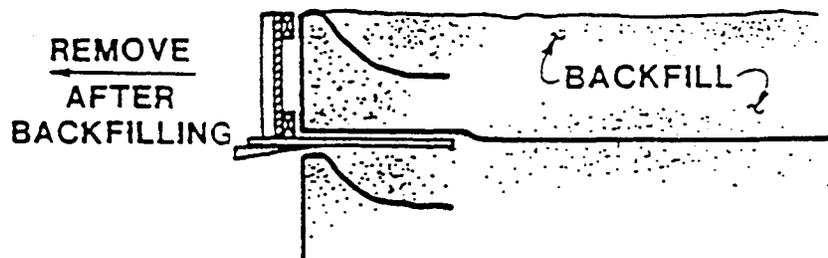
- Secure reinforcement with retaining pins to prevent movement during fill placement.
 - Overlap adjacent sheets a minimum of 6 in (152 mm) along the edges perpendicular to the slope. Alternatively, with geogrid or wire mesh reinforcement, the edges may be clipped or tied together. The strength of these overlaps or connections is not critical, except where the reinforcement forms the face.
3. Place backfill in reinforced section.
- Place fill to required lift thickness (less than 12 in [< 305 mm] loose thickness) on the reinforcement using a front end loader or dump truck operating on previously placed fill or natural ground.
 - Maintain a minimum of 6 in (152 mm) between reinforcement and wheels of construction equipment.
 - Compact with a vibratory roller or plate type compactor. 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 wall or slope face to help maintain face alignment. If compaction results in significant lateral movement at the face, short reinforcement strips placed parallel to the slope and intermediate between the primary reinforcement layers (e.g., every lift or two) will prove especially useful in slopes where formwork and/or wrapped facings are not required.
4. Compaction control.
- Provide close control on the water content and density of the backfill. It should be compacted at at least 95 percent of the AASHTO T-99 maximum density within 2 percentage points of optimum moisture.
 - If the backfill is a coarse aggregate, then a relative density or a method type compaction specification should be used.
5. Face construction for walls and steep slopes.
- Place the geosynthetic layers using face forms as shown in figure 62.



① PLACE FALSEWORK AND GEOTEXTILE ON PREVIOUS LIFT



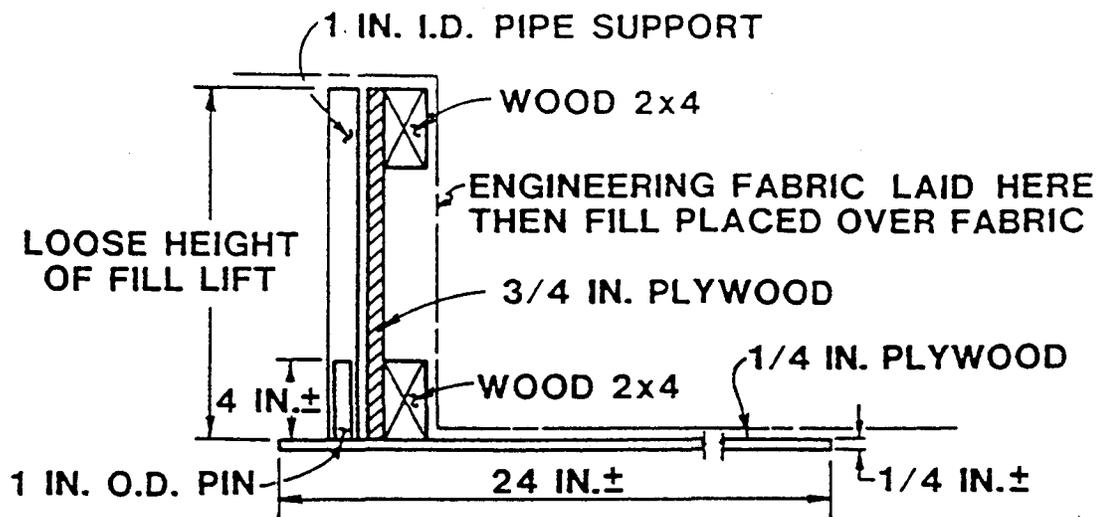
② PLACE/COMPACT PARTIAL BACKFILL AND OVERLAP FABRIC



③ PLACE/COMPACT REMAINDER OF BACKFILL LIFT

Figure 62. Lift construction sequence for engineering fabric reinforced soil walls. (47)

- For temporary support of forms at the face to allow compaction of the backfill against the wall face, form holders should be placed at the base of each layer at 4-ft (1.22 m) horizontal intervals. Details of temporary form work are shown in figure 63a. 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 3 ft (0.91 m) of the wall face, a hand-operated vibratory compactor is recommended.
 - The return-type method or successive layer tie method as shown in figure 63b 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 4 ft (1.22 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 64 shows some alternative facing systems for flexible faced walls and slopes.
6. Face construction for shallow slopes.
- Form work may not be required for slopes of 1 horizontal to 1 vertical or less. In this case, the reinforcement can simply be turned up at the face of the slope and returned into the embankment below the next reinforcement layer. Reinforced slopes with angles of 1H:1V or less built with partially cohesive soils and vegetated upon completion may not require a wrap around facing. Prior to tensioning the return reinforcement and before placing the next lift, the backfill should be at least lightly tamped to aid in maintaining the face shape. If the facing of the slope can be maintained during construction, an erosion treatment consisting of either lightweight geotextiles, geogrids, or meshes could be attached to the reinforcement after construction by tying or clipping methods. In this case, the slope face should be covered with the erosion control material or Visqueen at the end of each construction day to prevent erosion during construction.



A. ELEVATION VIEW OF WALL FACE FALSEWORK

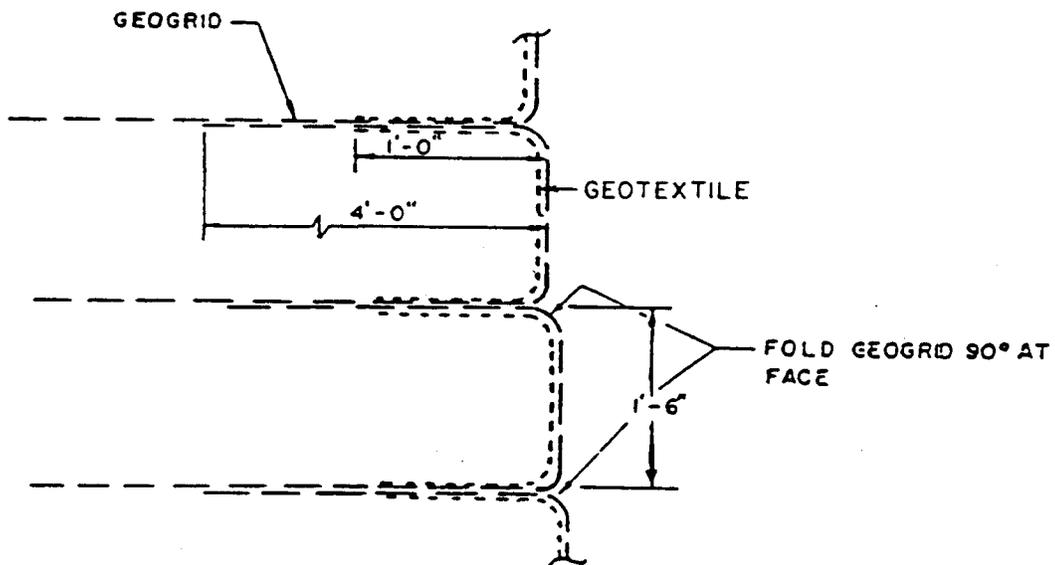


Figure 63. Typical geosynthetic face construction detail.

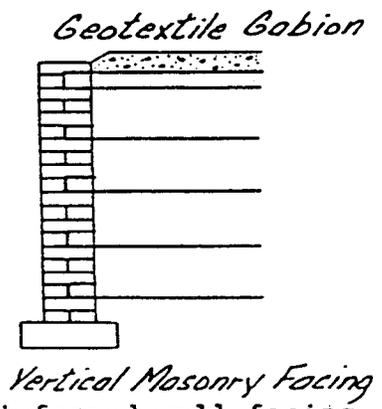
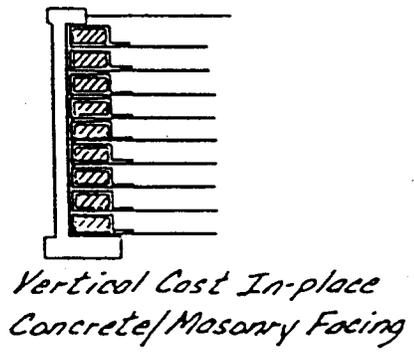
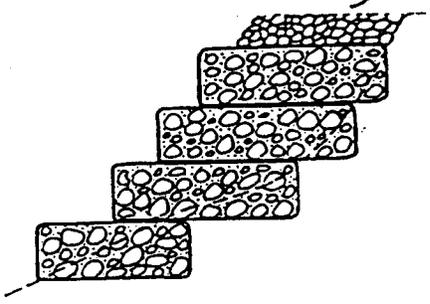
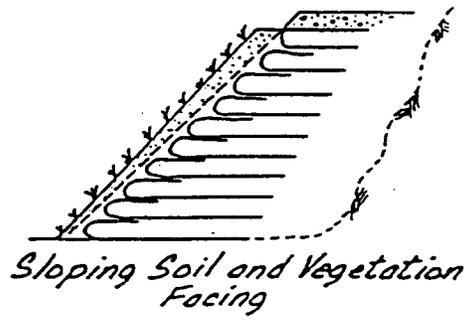
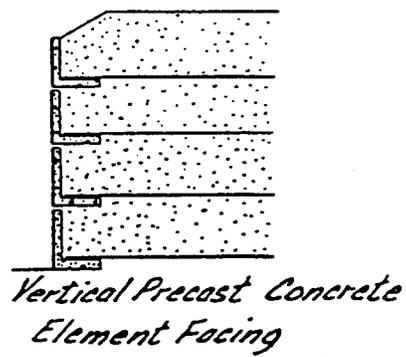
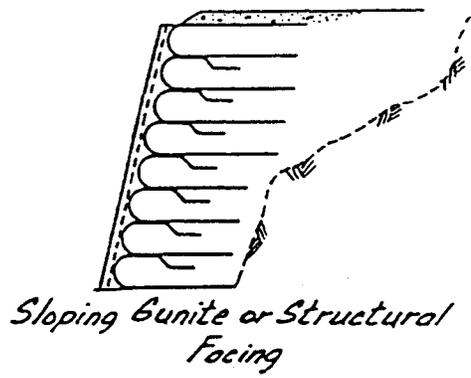
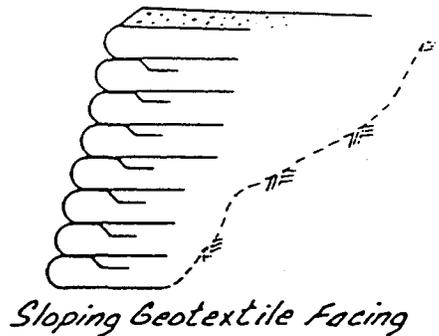
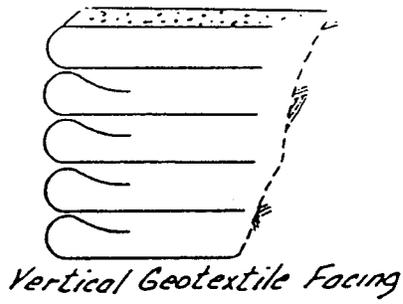


Figure 64. Types of geotextile reinforced wall facing.

7. Continue with additional reinforcing materials and backfill.

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

7.6 IN-SITU REINFORCEMENTS BY SOIL NAILING

In the nailing of excavations, a reinforced retaining structure is created as the excavation proceeds downward from the existing ground surface. The procedures for installation of the reinforcing elements are the reverse of those associated with the construction of a multilayer reinforced fill structure. The successive steps are as follows:

1. Excavation is carried out to a level slightly below the first row of reinforcement or nails. The depth of cut in this case is usually limited to about 5 to 6 ft (1.5 to 2 m), but in some cases could be as much as 10 ft (3 m), depending on the type of soil. In soils which start to ravel in this excavation stage, temporary supports may be required to maintain stability for the period required for the installation of the reinforcing elements and the facing elements.
2. Installation of the first row of reinforcing elements or nails.
3. Installation of facing elements, if any (wire mesh and shotcrete or prefabricated elements).
4. Connection of the facing elements to the reinforcing.
5. If the design requires drainage of the retained soil by artificial means, such as by installing drain pipes, then drilling of the drainage holes and installation of drainage pipes proceeds simultaneously with the installation of the nails in stages as described above. The drainage holes may be aligned up slope or horizontal to bring seepage water to a drainage layer which can be placed behind the facing elements.
6. The above steps are repeated for the next stage of excavation and installation of the reinforcing nails until the required depth is reached.
7. If a drainage system is provided behind the facing panels, then drain pipes with suitable outlets should be installed at the toe of the slope. Sometimes weep holes are provided in the facing.

a. Equipment Required and Construction Methods

The equipment required for soil nailing consists of the usual excavating and earth moving equipment, equipment for the installation of reinforcing elements and drainage holes, and for installation of the facing, which is usually shotcreting equipment.

Installation of the reinforcing nails may be by a small vibro-percussion hydraulic rammer which is used to drive the reinforcing bars, or it may consist of drilling and grouting equipment if the reinforcing elements are to be grouted in place. The driving technique is rapid and economical, however, it may create problems in the case of long reinforcing bars and in soils containing boulders or other obstructions. Furthermore, it is difficult to control orientation of the bars if they deflect during the driving. For long-term performance, corrosion of reinforcing bars will be a problem. If they are installed in drilled holes and properly grouted, corrosion protection can be obtained.

For the grouted in place type of installation, the drilling equipment may consist of a rotary type rig which uses drilling mud or compressed air or it may be a continuous flight solid stem or hollow stem auger rig.

For the installation of the reinforcing elements by the drilling and grouting in place method, a borehole 3 to 6 in (76 to 152 mm) diameter is first drilled to the desired depth by one of the above methods, and then the reinforcing element or rod is inserted in the hole.

The selected method of drilling depends on the type of soils and preference of the contractor. In stiff cohesive soils where the borehole can remain open unsupported for the short time required for the insertion of the reinforcing element and filling with grout, a hollow stem or a continuous flight solid stem auger can be used. Once the hole is advanced to the desired depth, the reinforcing element is inserted using appropriate spacers as necessary to maintain the rod in its central position, and grout is pumped under gravity or using very low pressure to fill the annular space around the bar. Grouting is done from the bottom up. If a solid stem auger is used to drill the hole, the rod and a grout pipe are inserted in an unsupported hole, using centralizers so that the rod stays in the center of the hole. The grout pipe has its tip close to the end of the rod and it is withdrawn so the grout is placed and fills the hole from bottom up. In the case of drilling with a hollow stem auger, the reinforcing bar is inserted through the hollow stem, which is also used for pumping of the grout. The auger is removed as the grout flows upward.

Hollow stem auger drilling would be appropriate in loose soils and also in certain sandy soils in which the drilled hole will not remain open unsupported.

Another method for drilling is to use a rotary rig with the cuttings removed either by drilling mud or by compressed air. In this case, the hole is maintained open by a steel casing. The reinforcing element is inserted through the casing along with a grout pipe extending to the bottom of the hole. The casing pipe and the grout pipe are withdrawn as the grout filling continues from the bottom up. If the hole is drilled using drilling mud, then there may not be any need to use a steel casing to maintain stability, as the mud usually can keep the borehole open. In this case, the rod is inserted in the drilled hole along with the grout pipe, using appropriate centering rings so that the reinforcing rod stays in the center of the hole. The grout pipe is withdrawn as the level of the grout rises in the hole from the bottom up.

Soon after excavation, the nails are installed and welded wire mesh or prefabricated facing panels are attached to the installed nails. In the case of drilled reinforcing elements, the connection of the facing element to the rod may be through a steel plate bolted on the rod. In the case of driven nails, a similar connection can be made if damage to the threads at the end of the rod can be prevented during driving.

b. Facings

The facing in a nailed soil retaining structure has only a minor structural role. The maximum tensile forces generated in the nails are in fact significantly greater than those transferred to the facing. The main function of the facing is to ensure the local stability of the ground between the nails. Hence, the facing has to be continuous, fit the irregularities of the cut slope surface, and be flexible enough to withstand ground displacement during excavation. Depending on the application and soil (or rock) type, four kinds of facing are presently used:

Shotcrete Facing (4 to 10 in [102 to 254 mm] thick) is currently used for most temporary retaining structures in soils. This facing provides a continuous, flexible surface layer that can fill voids and cracks in the surrounding ground. It is generally reinforced with a welded wire mesh, and its required thickness is obtained by successive layers of shotcrete (4 to 6 in [102 to 152 mm] thick). This technique is relatively simple and inexpensive, but it may not provide the technical quality and aesthetics required for permanent structures. In particular, the durability of the shotcrete facing can be affected by groundwater, seepage, and environmental factors such as climatic changes, e.g., freezing, which may induce cracking. In addition, construction of a shotcrete facing makes provision of efficient drainage at the concrete-soil interface difficult.

Welded Wire Mesh is generally used to provide a facing in fragmented rocks or intermediate soils (chalk, marl, shales).

Concrete and Steel Facings. Cast-in-place reinforced concrete facing is, to date, most frequently used for permanent structures. However, prefabricated concrete or steel panels have been recently developed for permanent structures. These panels can be designed to meet a variety of aesthetic, environmental, and durability criteria. They provide appropriate technical solutions for integrating continuous drainage behind the facing. Concrete panels have also been used in combination with prefabricated steel panels and cast-in-place concrete.

Grouted nails are generally attached to the facing (mesh or shotcrete) by bolting the bars to the square steel plate (12 to 16 in [305 to 406 mm] wide); whereas driven bars are generally attached to the facing by cladding or other suitable methods.

c. Drainage

Groundwater is a major concern in the construction of nailed soil retaining structures. An appropriate drainage system should be provided to (a) prevent generation of excessive hydrostatic pressures on the facing (or the structural elements), (b) protect the facing element and particularly shotcrete facing from deterioration induced by water contact, (c) prevent saturation of the nailed ground which can significantly affect the structure displacement and may cause instabilities during and after excavation. In soil nailing, weep holes consisting of shallow drainage pipes (plastic pipes 4 in [102 mm] in diameter, 12 to 16 in [305 to 406 mm] long) are usually used to protect the facing, while inclined slotted plastic tubes are used for drainage of the nailed ground. In the case of permanent structures with prefabricated panels, a continuous drain such as a geotextile or prefabricated drainage board, can be placed behind the facing.

d. Precautions and Observations Required for In-Situ Soil Reinforcement

Quality assurance measures which are required in in-situ soil reinforcement are:

1. Maintenance of stability of the excavated faces at different stages of excavation and prior to installation of the facing elements, if any.
2. Installation of the reinforcing elements at correct orientation and spacing to the correct design depth.
3. Proper location of the reinforcing rods in the drilled hole, including use of centralizers in the case of holes drilled by solid stem auger.
4. Proper grouting or filling of the hole around the reinforcing element. The amount of grout inserted in the hole should be measured to see that it agrees with the theoretical volume of the hole.

5. Assurance of adequate shotcrete strength, proper placement of wire mesh in the shotcrete, and proper connection of the facing elements, if used, to the reinforcing rods or nails.
6. Proper installation of the drainage holes and the drains, including proper gradation of backfill material and slots in the drainage pipes.

CHAPTER 8

MONITORING OF REINFORCED SOIL STRUCTURES

8.1 INTRODUCTION

Although several thousand reinforced soil structures have been constructed in the United States, very few of them have been instrumented. For example, a recent review of polymeric reinforced soil structures by Yako and Christopher found only 13 structures with documented instrumentation results out of 200 estimated to have been constructed in North America.⁽⁴⁸⁾ Table 13 summarizes these case histories. The small number of instrumented cases is both surprising and unfortunate, because confidence in the use of reinforced soil retaining structures and improvements in design will be enhanced only by proven performance of such systems. To this end, it is important to monitor performance behavior of future reinforced soil structures.

Monitoring can be of limited nature, with the intention of obtaining data on performance. Alternatively, it can be more comprehensive for one or more of the following purposes:

- Confirming design stress levels and monitoring safety during construction.
- Allowing construction procedures to be modified for safety or economy.
- Controlling construction rates.
- Enhancing knowledge of the behavior of reinforced soil structures, to provide a base reference for future designs, with the possibility of improving design procedures and/or reducing costs.
- Providing insight into maintenance requirements, by long-term performance monitoring.

This chapter includes details necessary to plan and implement both limited and comprehensive monitoring programs for reinforced soil systems. Recommendations for appropriate instrumentation are included.

8.2 LIMITED MONITORING PROGRAM

Limited observations and monitoring will typically include:

1. Horizontal movements of the face.
2. Vertical movements of the surface of the overall structure.
3. Local movements or deterioration of the facing elements.

Table 13. Instrumented polymeric reinforced fill walls and slopes.

Project	Facing	Reinforcement Type	Measurement ¹	Instrumentation	Reference
Olympic National Forest Road Shelton, WA	Wrapped, vert. CSR asphalt emulsion coat	Non-woven needlepunched	HD HD VD VD	Surveying methods Probe extensometers Surveying methods Horizontal inclinometers	Bell and Steward, 1977 (50) Steward and Mogygy, 1982 (47) Mohny, 1977 (51) Chassie, 1984 (52)
Theory and principle of reinforced earth walls, Vicksburg, MS	Alcoa Fil high strength panels, vertical	4-ply strip	HD TSW TSB	Visual observation Earth pressure cells Earth pressure cells	Al-Hussaini, 1977 (53) Al-Hussaini and Petry, 1976 (54)
New York Route 22 Columbia County, NY	Wrapped, 1:1 slope reinforce gunite	Non-woven needlepunched	HD ND ND	Probe extensometers Inclinometers Settlement devices	Douglas, 1982 (55) Chassie, 1984 (52)
I-70 Test Wall Glenwood Canyon, CO	Wrapped 1:1 slope reinforce gunite	Non-woven needlepunched	HD HD HD VD	Surveying methods Probe extensometers Inclinometers Visual observation Horizontal inclinometers Liquid level gages Probe extensometers Earth pressure cells Piezometers	Chassie, 1984 (52) Bell et al., 1983 (56) Barrett, 1985
Rte. 84 Lahoma, CA	Wrapped 1.5:1 to 1:1 slope compacted straw	Geogrid	HD HD HD VD	Surveying methods Extensometers Inclinometers Surveying methods	Forsyth and Bieber, 1984 (58)
Tarque Verde Rd. Tucson, AZ	Full height pre-cast concrete panels	Geogrid	HD TSW TSB LR LR T	Visual observation Earth pressure cells Earth pressure cells Resistance strain gages Induction coils Resistance thermometers	Berg et al., 1986 (59) Anderson, 1986 (60) Rock Mountain Construction, 1985
Jasper Asphalt Plant Operation Lithonia, GA	Cruciform pre-cast concrete panels	Geogrid	HD TSW TSB LR LR	Visual observation Earth pressure cells Earth pressure cells Resistance strain gages Induction coils	Berg et al., 1986 (59) Dixie Contractor, 1985
Gaspe Peninsula Seavall, Quebec	Concrete wave deflector panels	Geogrid	HD HD VD TSW LR PP T	Visual observation Fixed embankment extensometers Visual observation Earth pressure cells Strain gages Pneumatic piezometers Thermocouples	Berg et al., 1987 (59)

Table 13. Instrumented polymeric reinforced fill walls and slopes. (continued)

Project	Facing	Reinforcement Type	Measurement ¹	Instrumentation	Reference
Cascade Dam Cascade, MI	Wrapped wood facing attached after construct.	Geogrid	HD HD VD TSM LR LR	Visual observation Fixed embankment extensometers Visual observation Load cells at tie rod locations Resistance strain gages Induction coils	Christopher, 1987 (22)
Devon Geogrid Test Fill Devon, Alberta	1:1 slope	Geogrid	HD HD VD LR LR PP T T	Probe extensometers Inclinometers Probe extensometers Horizontal inclinometers Resistance strain gages Induction coils Pneumatic piezometers Prest gages Thermocouples	Scott et al., 1987 (61)
I-210 over Southern Pacific R.R. Lake Charles, LA	1:1 slope	Geogrid	HD LR PP	Inclinometers Strain gages Piezometers	Scott et al., 1987 (61)
I-20 over Chicago Mill Box Culvert, Tululah, LA	1:1 slope	Geogrid	HD LR	Inclinometers Strain gages	Scott et al., 1987 (61)
NATO Advanced Research Workshop Kingston, Ontario	1:1 slope	Geogrid	HD VD TSM LR T	Visual observation Visual observation Earth pressure cells Strain gages Thermocouples	Scott et al., 1987 (61)

¹HD = Horizontal Deformation
VD = Vertical Deformation
TSM = Total Stress at Base or Face of Wall
TSB = Total Stress Within Backfill
LR = Load Distribution in Reinforcement
PP = Pore Pressure Response Below Wall
T = Temperature

4. Drainage behavior of the backfill.
5. 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 1 inch (25 mm) for rigid reinforcement and 2 inches (51 mm) 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 one half inch per 10 feet (4.2 mm per m) of wall height for either system. Post construction horizontal movements are anticipated to be very small. Post construction vertical movements should be estimated from foundation settlement analyses, and measurements of actual foundation settlement during and after construction should be made.

Drainage can be monitored visually by observing outflow points during storm events or through open stand pipe piezometers installed near the base and in the back of the reinforced soil section.

8.3 COMPREHENSIVE MONITORING PROGRAM

Comprehensive studies involve monitoring of surficial behavior as well as internal behavior of the reinforced soil. Planning and execution of such programs are discussed in the following sections.

8.4 PLANNING MONITORING PROGRAMS

A well defined systematic plan should be used for developing all monitoring programs, whether limited or comprehensive. Table 14 provides the key steps that should be followed in developing such a program. Dunicliff⁽⁴⁹⁾ includes a checklist for use in ensuring that all steps in the planning process have been taken. All steps should be followed, and if possible, completed before instrumentation work commences in the field. Based on Dunicliff's discussion of these steps, each can be adapted for monitoring reinforced soil structures.

a. Purpose of the 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. Purposes of monitoring programs are discussed in section 8.1.

Table 14. Systematic approach to planning monitoring programs using geotechnical instrumentation. (49)

1. Define purpose of the monitoring program.
2. Define the project conditions.
3. Predict mechanisms that control behavior.
4. Select the parameters to be monitored.
5. Predict magnitudes of change.
6. Devise remedial action, should measurements exceed warning levels.
7. Assign monitoring tasks for design, construction and operation phases.
8. Select instruments, based on reliability and simplicity.
9. Select instrument locations.
10. Plan recording of factors that may influence measured data.
11. Establish procedures for ensuring reading correctness.
12. Prepare budget.
13. Write instrument procurement specifications.
14. Plan installation.
15. Plan regular calibration and maintenance.
16. Plan data collection, processing, presentation, interpretation, reporting and implementation.
17. Write contractual arrangements for field instrumentation services.
18. Update budget.

b. Define the Project Conditions

The engineer responsible for planning the monitoring program must be thoroughly familiar with the project, including the reinforced structure layout, subsurface conditions, groundwater conditions and construction methods. Environmental conditions that might affect the performance of either the structure or the instrumentation must also be considered.

c. Predict Mechanisms that Control Behavior

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. The more flexible the reinforcement, the more likely horizontal stress levels will approach K_a as opposed to K_0 for rigid reinforcement. Likewise, stiffer wall facings are anticipated to result in higher lateral stress levels closer to the face than with flexible wall facings.

d. Select the Parameters to be Monitored

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 reinforced structures, important parameters that should be considered include:

1. Horizontal movements of the face.
2. Vertical movements of the surface of the overall structure.
3. Local movements or deterioration of the facing elements.
4. Drainage behavior of the backfill.
5. Performance of any structure supported by the reinforced soil, such as approach slabs for bridge abutments or footings.
6. Horizontal movements within the overall structure.
7. Vertical movements within the overall structure.
8. Lateral earth pressure at the back of facing elements.
9. Vertical stress distribution at the base of the structure.

10. Stresses in the reinforcement, with special attention to the magnitude and location of the maximum stress.
11. Stress distribution in the reinforcement due to surcharge loads.
12. Relationship between settlement and stress-strain distribution.
13. Stress relaxation in the reinforcement with time.
14. Total horizontal stress within the backfill and at the back of the reinforced wall section.
15. Aging condition of reinforcement such as corrosion losses. ⁽¹⁵⁾
16. Pore pressure response below structure.
17. Temperature (often a cause of real changes in other parameters, and also may affect instrument readings).
18. Rainfall (often a cause of real changes in other parameters).
19. Barometric pressure may affect readings of earth pressure and pore pressure measuring instruments.

e. Predict Magnitudes of Change

From the design methods in chapters 3, 4, and 5, the predicted magnitudes for each parameter should be established. Not only should the anticipated value be calculated, but predictions should be made to establish the required range and accuracy of each instrument. Stress-strain relations for the materials need to be established as well as the anticipated deformation response of the structure.

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.

f. Devise Remedial Actions, Should Measurements Exceed Warning Levels

As indicated in the previous section, maximum levels may be required to provide a warning, should they be exceeded. 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.

g. Assign Monitoring Tasks for Design, Construction, and Operation Phases

A chart for assigning monitoring tasks is included as table 15.

Several of the tasks involve the participation of more than one party. In cases where the owner is also the designer, there will be no design consultant. Instrumentation specialists may be employees of the owner or the design consultant or may be consultants with special expertise in geotechnical instrumentation. All tasks assigned to instrumentation specialists should be under the supervision of one individual.

When assigning tasks, the party with the greatest vested interest in the data should be given direct line responsibility for producing it accurately. Reliability and patience, perseverance, a background in the fundamentals of geotechnical engineering, mechanical and electrical ability, attention to detail, and a high degree of motivation are the basic requirements for qualities needed in instrumentation personnel.

Further discussion of table 15 and guidance for assigning tasks are contained in reference 49.

Table 15. Chart of task assignment.

Task	<u>Responsible Party</u>			
	Owner	Design Consultant	Instru- mentation Specialist	Construc- tion Contractor
Plan monitoring program				
Procure instruments and make factory calibrations				
Install instruments				
Maintain and calibrate instruments on regular schedule				
Establish and update data collection schedule				
Collect data				
Process and present data				
Interpret and report data				
Decide on implementation of results				

h. Select Instruments

Instrumentation should be selected on the basis of reliability and simplicity. Lowest cost of an instrument should never be allowed to dominate the selection, and the least expensive instrument is not likely to result in minimum overall cost. Before selecting the instruments, the engineer should be thoroughly familiar with the operation, accuracy, and reliability of both the instrument and its readout. The effects of the environment on the instrument should also be well understood and protection requirements evaluated.

Limitations in the skill of available personnel for installing the instruments should also be identified. This may alter both the instrument selection and also the entire approach to the monitoring program. The influence of the instrumentation installation on construction must also be assessed, and instruments should be selected which result in minimum interference. Access for instrumentation installation and monitoring must also be considered.

Most of the instruments utilized will be conventional geotechnical type instrumentation. Table 16 provides an instrumentation selection chart with recommendations on the types of instruments that should be considered for each parameter identified in section d. The various instruments are described and evaluated in reference 49. The following additional factors and guidelines apply to selection of strain gauges for monitoring stress in the reinforcement. The selection is, of course, highly dependent on the reinforcement type.

1. Sensitivity of the instrumentation over a large range of strains (from large during construction to very small following construction).
2. Compatibility of gauges and attachment method to the type of reinforcement material (each type of reinforcement requires different considerations).
3. Sufficient redundancy to explain anomalous data.
4. Sufficient number of instruments along with preferential spacing to identify highly localized areas of maximum stress.

Table 16. Possible instruments for monitoring reinforced soil structures.

Parameters	Possible Instruments
Horizontal movements of face	Visual observation Surveying methods Horizontal control stations
Vertical movements of overall structure	Visual observation Surveying methods Benchmarks
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
Vertical movements within overall structure	Surveying methods Benchmarks Probe extensometers Horizontal inclinometers Liquid level gauges
Performance of structure supported by reinforced soil	Numerous possible instruments (depends on details of structure)
Lateral earth pressure at the back of facing elements	Earth pressure cells Strain gauges at connections Load cells at connections
Stress distribution at base of structure	Earth pressure cells
Stress in reinforcement	Resistance strain gauges Induction coil gauges Hydraulic strain gauges Vibrating wire strain gauges Multiple telltales

Table 16. Possible instruments for monitoring reinforced soil structures (continued).

<u>Parameters</u>	<u>Possible Instruments</u>
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: <ul style="list-style-type: none"> • 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

See also additional guidelines in section 8.4.h.

5. Measurement of both local (micro) and global (macro) strains in each gauged layer.
6. Perform calibration of gauged reinforcement using both unconfined tension and confined in-soil tension tests (e.g., pullout tests).
7. Placement of strain gauges on both top and bottom of the reinforcement to identify bending stresses (if this is not done, in the interests of economy, questionable results are likely to be obtained).

8. Evaluate temperature effects.
9. Exhibit care during initial placement of soil cover.
10. Monitor continuously during construction (not just at its beginning and end).

In the final selection, the instruments chosen must achieve the monitoring objectives. Where unproven instruments are used, sufficient backup should be provided to verify resulting data.

Figure 65 provides an example of a chart for summarizing the selected instruments.

i. Select Instrument Locations

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. However, in most cases, parameters are of interest over an entire section of the structure. Therefore, a large number of measurement points may be required to evaluate such parameters as distribution of stresses in the reinforcement and stress levels below the retaining structure. For example, accurate location of the locus of the maximum stress in the reinforced soil mass will require a significant number of gauge points, usually spaced on the order of 1 ft (305 mm) apart in the critical zone. Reduction in the number of gauge points will make interpretation difficult, if not impossible, and may compromise the objectives of the program.

Parameter	Range Required	Accuracy Required	Instrument	Supplier	Quantity	Readout and Other Components	Cost
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Figure 65. Example format for instrument summary chart.

Figures 66 and 67 provide examples of possible instrumentation layouts for a wall and a slope, respectively, and are based on comprehensive monitoring programs conducted during preparation of this manual. The details for this field study are summarized in volume II, Summary of Research and Systems Information. The instrumentation program was developed to evaluate all of the parameters listed in section 8.4.d for the range of reinforcement types and soil conditions currently used for reinforced backfill walls and slopes. It should be noted that figures 66 and 67 are only examples, and it must be stressed that many other configurations are possible.

j. Plan Recording of Factors that May Influence Measured Data

It is very important to maintain records with special emphasis on factors that might affect instrumentation results. Items that should be included during the construction phase are:

1. An "installation record sheet" for each instrument (section 8.4.n).
2. Construction details and progress (especially delays).
3. Visual observations of unusual behavior of the structure.
4. Activities around instrumentation locations.
5. Environmental factors such as temperature, rainfall, snow, sun, shade, etc.

After construction is complete, visual observations and environmental factors should be included.

A special section for recording such items should be included on field data sheets (section 8.5.d) to emphasize their importance.

k. Establish Procedures for Ensuring Reading Correctness

Methods of determining whether an instrument is functioning correctly must be established. Anomalous readings in themselves do not necessarily mean that the instrument is not functioning, but may actually be an indication of unusual behavior. Reading correctness can best be established through redundancy in the instrumentation, both in the number of instruments and in using more than one type of instrument.

For example, strain gauges should be installed on both sides of the reinforcement so that bending strains can be eliminated from the calculation of lateral stresses. If resistance strain gauges are used, several options can be considered for the electrical circuit. The two gauges could be connected at the measuring point with two additional compensating gauges at 90° to the principal gauges and connected in a full bridge. (This is the preferred method for metal strip and bar type reinforcement, but it should

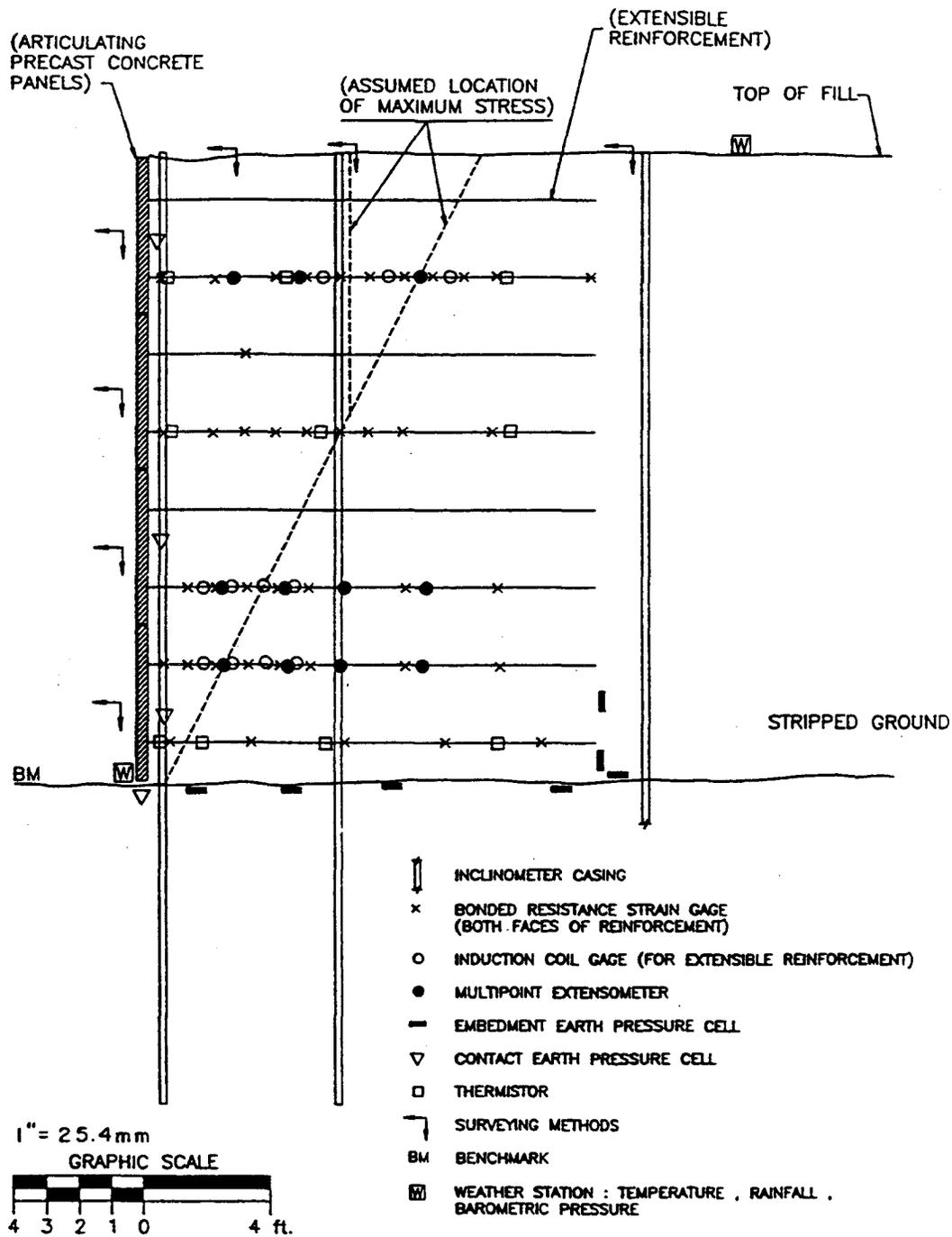


Figure 66. Possible layout of instrumentation for comprehensive monitoring of reinforced backfill walls.

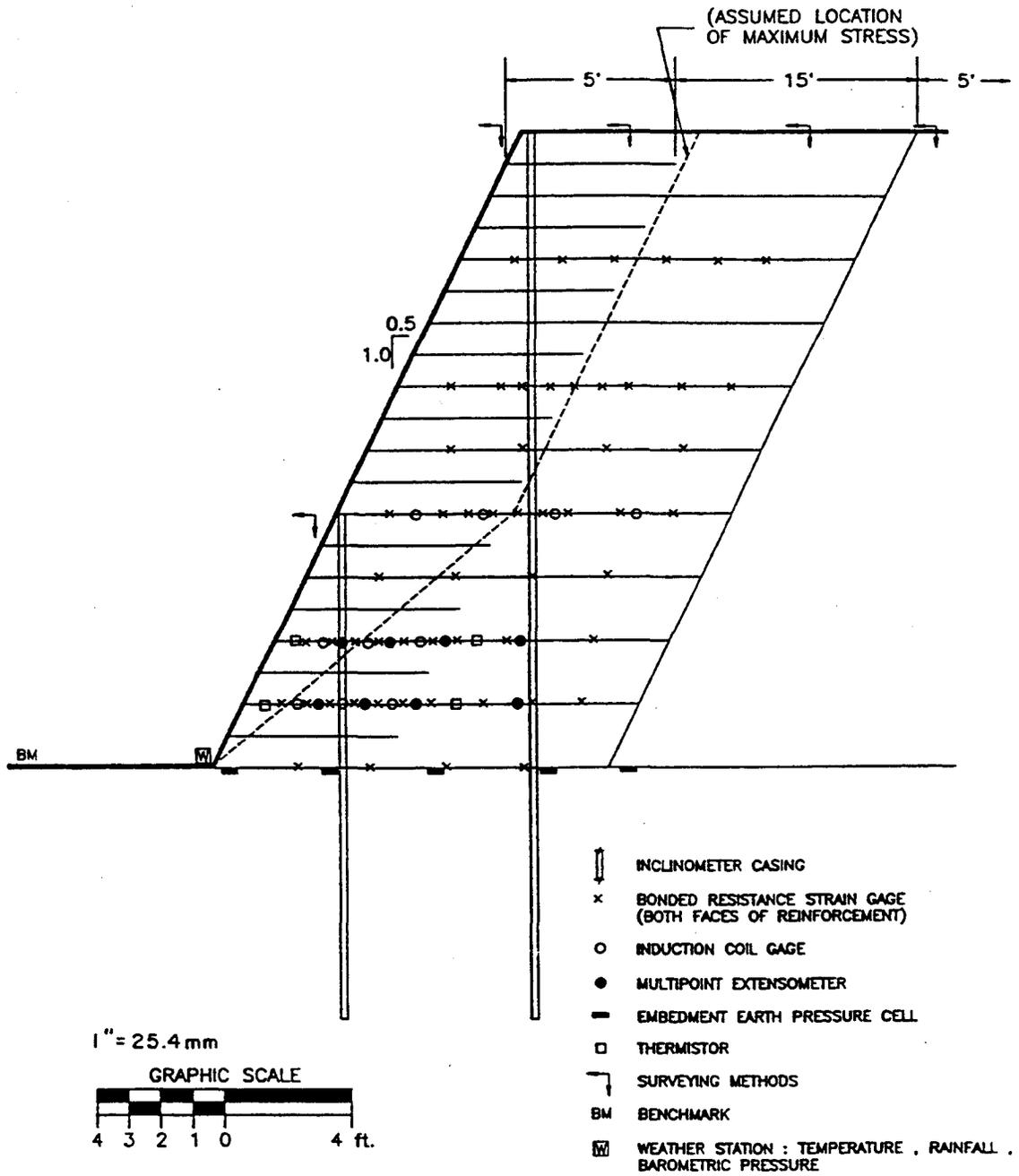


Figure 67. Possible layout of instrumentation for comprehensive monitoring of reinforced embankment slopes.

not be used for biaxial geosynthetic reinforcements.) Alternatively, the gauges can be attached separately as two three wire quarter bridges at the measuring point and connected separately or in series to the readout. (For a series connection, the measured strain should be factored by 2.) By allowing for independent gauge readings, the quarter bridge option provides redundancy and a means of judging reading correctness. Axial and bending strains can be determined separately, compared with data at neighboring gage locations, and evaluated accordingly.

Visual observations and optical surveys can also be used to support readings. Data consistency can also be used to examine correctness. An example would be consistency of strain measurements in the reinforcement between strain gauge data and multiple telltale data.

An effective way of examining correctness is to look at repeatability. Therefore, repeat readings should be taken whenever possible over short periods of time to evaluate consistent response of the instrument and reading method.

1. Prepare Budget

It is necessary at this time in planning to prepare a budget to make sure that sufficient funds are available to meet the needs of the program. It is much easier to modify the program at this stage than after the program has been completed. It is very important that the costs of all tasks listed in table 15 are carefully estimated.

m. Write Instrumentation Procurement Specifications

Procurement of other than the most simple geotechnical instruments should not be considered as a routine construction procurement item, because if valid measurements are to be made, extreme attention must be paid to quality and details. The cost is usually minor.

The "low-bid" method should never be used unless regulations allow for no alternative. One of the following two methods is recommended:

- The owner or design consultant procures the instruments directly, negotiating prices with suppliers.
- The owner enters an estimate of procurement cost in the construction contract bid schedule (this is called an "allowance item") and subsequently selects appropriate instruments for procurement by the contractor. Price is negotiated between the owner and suppliers of instruments, and the construction contractor is reimbursed at actual cost plus a handling fee.

In cases where neither of these methods can be used and the "low-bid" method with an "or equal" provision is unavoidable, a clear, concise, complete, and correct specification must be written. The specification should cover all salient features to guard against supply of an undesirable substitution, following the guidelines given in reference 49.

While writing instrument procurement specifications, one should determine the requirements for factory calibrations, and "acceptance tests" (section 8.5.a) should be planned to ensure correct functioning when instruments are first received by the user.

n. Plan Installation

Step-by-step installation procedures should be prepared, well in advance of scheduled installation dates, for installing all instruments. Detailed guidelines for installation of instruments are given in reference 49. The manufacturer's instruction manual and the designer's knowledge of the specific site conditions must be incorporated into these procedures. Included in the installation procedures should be a listing of required materials and tools. "Installation record sheets" should be prepared for documenting as-built installation details. Installation record sheets should include a record of appropriate items listed in table 17.

Procedures should ensure that the presence of the instruments do not alter the very quantities that instruments are intended to measure. Training programs for installation personnel should be established, access needs for installation should be planned, as should procedures for protecting instruments from damage and vandalism.

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 effect his operation or schedule.

Table 17. Possible content of installation record sheet. (49)

1. Project name.
2. Instrument type and number, including readout unit.
3. Planned location in plan and elevation.
4. Planned orientation.
5. Planned lengths, widths, diameters, depths, and volumes of backfill around instrument.
6. Personnel responsible for installation.
7. Plant and equipment used, including diameter and depth of any drill casing used.
8. Date and time of start and completion.
9. Spaces for necessary measurements or readings required during installation to ensure that all previous steps have been followed correctly, including acceptance tests.
10. A log of appropriate subsurface data.
11. Type of backfill used around instrument.
12. As-built location in plan and elevation.
13. As-built orientation.
14. As-built lengths, widths, diameters, depths, and volumes of backfill around instrument.
15. Weather conditions.
16. A space for notes, including problems encountered, delays, unusual features of the installation, and any events that may have a bearing on instrument behavior.

o. Plan Regular Calibration and Maintenance

Factory calibrations, and acceptance tests when instruments are first received by the user, will have been planned as described in section m. The third phase of calibration is calibration during service life.

Portable readout units are especially vulnerable to changes in calibration, often resulting from mishandling and lack of regular maintenance. They can sometimes be checked and/or recalibrated by following the acceptance test procedure. When this is insufficient, calibrations can often be made at local commercial calibration houses, using equipment traceable to the National Institute of Standards and Technology. Calibration frequency depends, of course, on the specific instrument, but as a general rule, the user should arrange for regular calibrations on a frequent rather than infrequent schedule.

Many users have experienced the dilemma of discovering that changes in calibration have occurred, and are, therefore, unsure of data correctness since the last calibration date. Frequent calibrations minimize this dilemma. A sticker on each instrument should indicate the last and next calibration dates.

Detailed maintenance requirements vary with each instrument and should be stated in the manufacturer's instruction manual. The manual should include a troubleshooting guide, cleaning, drying, lubricating, and disassembly instructions, and recommended maintenance frequency. If batteries are required, service and charging instructions should be given in the manual.

Planned maintenance procedures should include readout units, field terminals, and embedded components. Detailed guidelines are included in reference 49.

p. Plan Data Collection, Processing, Presentation, Interpretation, Reporting, and Implementation

Many consulting engineering firms have files filled with large quantities of partially processed and undigested data because sufficient time or funds were not available for these tasks. Careful attention should be given to these efforts and the required time should not be underestimated.

The following steps are required:⁽⁴⁹⁾

1. Plan data collection:

- Prepare preliminary detailed procedures for collection of initial and subsequent data.
- Prepare field data sheets.
- Plan staff training.
- Plan data collection schedule and duration, in accordance with the purpose of the monitoring program.
- Plan access needs.

2. Plan data processing and presentation:

- Determine need for automatic data processing.
- Prepare preliminary detailed procedures for data processing and presentation.
- Prepare calculation sheets.
- Plan data plot format.
- Plan staff training.

3. Plan data interpretation:
 - Prepare preliminary detailed procedures for data interpretation.
 4. Plan reporting of conclusions:
 - Define reporting requirements, contents, frequency.
 5. Plan implementation:
 - Verify that all steps associated with remedial actions have been planned, including contractual authority and communication channels.
- g. Write Contractual Arrangements for Field Instrumentation Services

Field instrumentation services include instrument installation, regular calibration and maintenance, and data collection, processing, presentation, interpretation, and reporting. Contractual arrangements for the selection of personnel to provide these services may govern success or failure of a monitoring program.

Geotechnical instrumentation field work should not be considered a routine construction item, because successful measurements require extreme dedication to detail throughout all phases of the work. The "low-bid" method should never be used unless regulations allow for no alternative, and one of the following two methods is recommended:

- The owner or design consultant performs field instrumentation work that requires special skill, if necessary retaining the services of a consulting firm specializing in instrumentation. Supporting work is performed by the construction contractor.
- The owner enters an estimate of specialist field instrumentation service costs in the construction contract bid schedule. This is called an "allowance item". Subsequently, the owner and construction contractor select an appropriate specialist consulting firm, which is retained as an "assigned subcontractor" by the construction contractor to perform field instrumentation work that requires specialist skill. Charges for specialist work are negotiated between the owner and consulting firm, and the construction contractor is reimbursed at actual cost plus a handling fee. Supporting work is performed by the construction contractor.

One of these two methods is essential for data processing, presentation, and interpretation. They are also preferable for installation, calibration, maintenance, and data collection. Where regulations do not allow either of these methods to be used, and where the "low-bid" method is unavoidable, a clear, concise, complete, and correct specification should be written to maximize the quality of field services, following the guidelines given in reference 49.

r. Update Budget

A finalized budget should be prepared at this time, taking into consideration all the tasks listed in table 15.

8.5 EXECUTING MONITORING PROGRAMS

The key steps that should be followed in executing a monitoring program include: (49)

- Procure instruments.
- Install instruments.
- Calibrate and maintain instruments on a regular schedule.
- Collect data.
- Process and present data.
- Interpret data.
- Report conclusions.

Based on the discussion of these steps by Dunningcliff, each will be subsequently reviewed with respect to monitoring reinforced soil structures. (49)

a. Procure Instruments

Procurement specifications will have been written, as described in section 8.4.m. During manufacture, instruments should be calibrated, inspected, and tested, and appropriate certificates transmitted to the user.

On receipt by the user, "acceptance tests" should be made to ensure correct functioning because, if an instrument is not working perfectly at this stage, it is unlikely to work at all when installed in the reinforced soil structure. Whenever possible, acceptance tests should include a verification of calibration data provided by the manufacturer, by checking two or three points within the measurement range. When comprehensive acceptance tests are not possible, simple tests should be performed to verify that instruments appear to be working correctly.

b. Install Instruments

Installation of instruments requires a special effort, and equipment that has an excellent record of performance can be rendered unreliable if a single essential but apparently minor requirement is overlooked during the installation.

Before starting installation work, field personnel should study and understand the written step-by-step installation procedure described in section 8.4.n. However, even the best written procedure cannot provide for every field condition that may affect the results, and even slavish attention to the procedure cannot guarantee success. The installer must therefore have a background in the fundamentals of geotechnical engineering as well as knowledge of the intricacies of the device being installed. Sometimes the installer must consciously depart from the written procedure.

c. Calibrate and Maintain Instruments on a Regular Schedule

Readout units should be calibrated and maintained on a regular schedule. Field terminals should be maintained, and maintenance should include any embedded instrument components for which access is available.

d. Collect Data

Special care should be taken when making initial readings, because most data are referenced to these readings, and engineering interpretations are based on changes rather than on absolute values.

Data should be recorded on field data sheets, specifically prepared for each project and instrument. Field data sheets should include:

- Project name.
- Instrument type.
- Date.
- Time.
- Observer.
- Readout unit number.
- Instrument number.
- Readings.
- Remarks.
- Data correctness checks.
- Visual observations.
- Other causal data including weather, temperature, and construction activities.

Readings should be compared immediately with the previous set of readings, a copy of which should be taken into the field. Special paper, available from suppliers of weatherproof field books, can be used to allow writing in wet conditions. One or more field

data sheets will be used for each date, with later transcription of data to one calculation sheet for each instrument. Raw data should be copied and the copy and original stored in separate safe places to guard against loss.

Data collection personnel should take the first step in determining whether the instrument is functioning correctly, by comparing the latest readings with the previous readings. Any significant changes can then be identified immediately, and if warning levels (section 8.4.e) have been reached, supervisory personnel should be informed. Data collection personnel should record factors that may influence measured data and should be on the lookout for damage, deterioration, or malfunction of instruments.

The frequency of data collection should be related to construction activity, to the rate at which the readings are changing, and to the requirements of data interpretation. Too many readings overload the processing and interpretation capacity, whereas too few may cause important events to be missed and prevent timely actions from being taken. Good judgment in selecting an appropriate frequency is vital if these extremes are to be avoided.

e. Process and Present Data

The first aim of data processing and presentation is to provide a rapid assessment in order to evaluate data correctness and to detect changes requiring immediate action, and this should usually be the responsibility of data collection personnel. The second aim is to summarize and present the data in order to show trends and to compare observed with predicted behavior for determination of the appropriate action to be taken. Specially prepared data forms will usually be required, and a computer will often be used to minimize data processing effort.

Data should always be plotted, normally versus time. Plots of predicted behavior and causal data are often included on the same axes.

f. Interpret Data

Monitoring programs have failed because the data generated were 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.

g. Report Conclusions

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 initial communication may be verbal, but should be confirmed in writing. 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.

If important knowledge gaps have been filled, the conclusions should be disseminated to the profession in a technical publication.

CHAPTER 9

SPECIFICATIONS

9.1 INTRODUCTION

A successful reinforced soil project will require sound, well prepared specifications to provide 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 the owner's representative, usually at a detriment to the project.

This chapter provides guidance on items which should be included in reinforced soil specifications. Two detailed specifications are included at the end. The first specification was recently developed by AASHTO-AGC-ARTBA, Task Force 27. This specification is specifically intended to cover all galvanized steel strip or mesh stabilized earth systems utilizing discrete concrete facing panels. The second specification is an example of a special provision for a polymer reinforced wall system with flexible facing (obtained from Washington State Department of Transportation) which includes most of the items that should be considered. Caution is advised when using either of these specifications as a guideline, as only the concerns for the specific type of system are addressed.

9.2 ITEMS TO BE INCLUDED IN REINFORCED SOIL SPECIFICATIONS⁽¹³⁾

a. Materials

Facing Elements - Pertinent standard (AASHTO, ASTM) mechanical properties (concrete compressive strength, shotcrete compressive strength, tensile strength of polymers), fabrication method (precast, cast-in-place concrete, shotcrete and gunite), finish (texture, color for precast concrete), protective coating or additives (wire mesh-epoxy coating or galvanizing, carbon black in polymers and polymer coating such as bitumen), product identification, handling, storage and shipping, tolerances (dimensions and finish), acceptance/rejection criteria.

Reinforcing Elements - Pertinent standards (ASTM, AASHTO), strength, other properties (mechanical, hydraulic and durability for polymers), protective coatings (epoxy, PVC, and galvanizing plus grout requirements for nails), shape dimensions, product identification, handling, storage and shipping, acceptance/rejection criteria.

Connection and Alignment Devices - Pertinent standards (ASTM, AASHTO), strength, protective coatings, shape and dimensions, tolerances.

Joint Materials - Strength and modulus properties. Materials description and dimensions.

Reinforced Backfill - Gradation, plasticity index, strength, soundness, electrochemical properties and acceptance procedures.

b. Construction

Excavation - Pertinent pay items (generally outside reinforced soil specification), excavation limits, and construction sequencing.

Foundation Preparation - Limits of treatment, excavation of unsuitable material, backfill, compaction procedure, leveling pad construction.

Reinforced Soil System Erection - Materials supplier technical assistance, handling and placement procedures for reinforcing elements, facing form work and shoring requirements, sequence of erection (facing elements, reinforced backfill, facing elements), tolerances for wall alignment (horizontal, vertical), orientation of reinforcing elements, construction of special features (corners, wall ends, headers).

Reinforced Backfill Placement - Effects on panel placements (damage, misalignment), compaction requirements such as lift thickness, moisture content, density), end of day grading.

Method of Measure/Basis of Payment - Lump sum or unit prices (reinforced soil system materials, erection, leveling pad, backfill).

9.3 GUIDE SPECIFICATION FOR REINFORCED FILL SYSTEMS USING GALVANIZED STEEL STRIP OR MESH REINFORCEMENT

a. Description

This work shall consist of mechanically stabilized walls and abutments constructed in accordance with these specifications and in reasonably close conformity with the lines, grades, and dimensions shown on the plans or established by the engineer. Design details for these earth retaining structures such as specified strip or mesh length, concrete panel thickness, 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.

b. 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 documents shall not be used without written consent from the engineer.

Reinforced Concrete Facing Panels - The panels shall be fabricated in accordance with Section 4 of AASHTO, Division II, with the following exceptions and additions.

- (1) The Portland cement concrete shall conform to Class A, (AE) with a 4,000 lb/in² (psi) (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 1,000 psi (6.9 MPa). The units may be shipped after reaching a minimum compressive strength of 3,400 psi (23.4 MPa). At the option of the contractor, the units may be installed after the concrete reaches a minimum compressive strength of 3,400 psi (23.4 MPa).
- (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 4.25 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 1/4 in (6.4 mm). The panels shall be cast on a flat area. The coil embeds, tie strip guide, or other galvanized 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 of panel connection devices within 1 in (25.4 mm), except for coil and loop imbeds which shall be 3/16 in (4.8 mm). All other dimensions within 3/16 in (4.8 mm).

- Panel Squareness - Squareness as determined by the difference between the two diagonals shall not exceed 1/2 in (12.7 mm).
 - Panel Surface Finish - Surface defects on smooth formed surfaces measured over a length of 5 ft (1.52 m) shall not exceed 1/8 in (3.2 mm). Surface defects on the textured-finish surfaces measured over a length of 5 ft (1.52 m) shall not exceed 5/16 in (7.9 mm).
- (7) Steel - In accordance with section 5.
- (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 6 in (152-mm) by 12 in (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 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 which 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 4,000 psi (27.6 MPa). If the compressive strength test result is less than 4,000 psi (27.6 MPa), then the 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 4,150 psi (28.6 MPa).
- The average of any six consecutive compressive strength test results shall exceed 4,250 psi (29.3 MPa).
- No individual compressive strength test result shall fall below 3,600 psi (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 of 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 and shall be welded into the finished mesh fabric in accordance with ASTM A-185. 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) Coil Embeds/Loop Imbeds - Shall be fabricated of cold drawn steel wire conforming to ASTM 510, UNS G-10350, or ASTM A-82. Loop imbeds shall be welded in accordance with ASTM A-185. Both shall be galvanized in accordance with ASTM B-633 or equal.
- (5) Coil Embed Grease - The cavity of each coil embed shall be completely filled with no-oxide type grease or equal.
- (6) Coil Bolt - The coil bolts shall have 2 in (51 mm) of thread. They shall be cast of 80-55-06 ductile iron conforming to ASTM A-536. Galvanization shall conform to ASTM B-633 or equal.
- (7) 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.
- (8) 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 and galvanized in accordance with ASTM A-123.

Joint Materials - Installed to the dimensions and accordance with ASTM A-153 thicknesses in accordance with the plans or approved shop drawings.

- (1) Provide flexible foam strips for filler for vertical joints between panels, and in horizontal joints where pads are used.
- (2) Provide either preformed cord conforming to AASHTO M-153 Type II 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/cm³ in accordance with ASTM 1505.
- (3) Cover all joints between panels on the back side of the wall with a geotextile fabric. The minimum width and lap of the fabric shall be as follows:

Vertical and horizontal joints: 12 in (305-mm);
lap 4 in (102 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>U.S. Sieve Size</u>	<u>Percent Passing</u>
4 in (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, utilizing 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 3/4 in (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.
- (4) Electrochemical Requirements - The backfill materials shall meet the following criteria:

<u>Requirements</u>	<u>Test Methods</u>
Resistivity >3,000 ohm cm	California DOT 643
pH 5-10	California DOT 643
Chlorides <50 parts per million	California DOT 422
Sulfates <500 parts per million	California DOT 417

Concrete Leveling Pad - The concrete footing shall conform to AASHTO Division II, section 4.5 for Class B concrete.

Acceptance of Material - The contractor shall furnish the engineer a Certificate of Compliance certifying the above materials, excluding select backfill, 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.

c. 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.

Foundation Preparation - The foundation for the structure shall be graded level for a width equal to the length of reinforcement elements plus 1 ft (305 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 section (a). Materials of these specifications, except that the material shall have 100 percent passing the 3 in sieve (76 mm), 20 percent passing the No. 40 sieve, and 0 to 12 percent passing the No. 200 sieve.

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 3/4 in (19 mm) when measured with a 10 ft (3-m) straight edge. During construction, the maximum allowable offset in any panel joint shall be 3/4 in (19 mm). The overall vertical tolerance of the wall (top to bottom) shall not exceed 1/2 in (12.7 mm) per 10 ft (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 of reinforcing element. Any wall materials which 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 by the contractor at his 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 3/4 in (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 5 ft (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 have a placement 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 12 in (305 mm). The contractor shall decrease this lift thickness, if necessary, to obtain the specified density.

Compaction within 3 ft (0.91 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.

d. 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 foot (square meter) of wall face constructed.

Wall Erection - The unit of measurement for wall erection will be per square foot (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 construction, 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 feet (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 yards (cubic meters).

e. Payment

The quantities, determined as provided above, will be paid for at the contact 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:

<u>Pay Item</u>	<u>Pay Unit</u>
1. Wall materials	Square foot (meter)
2. Wall erection	Square foot (meter)
3. Concrete leveling pad	Linear foot (meter)
4. Select granular backfill	Cubic yard (meter)

9.4 CONSTRUCTION SPECIAL PROVISION FOR GEOTEXTILE RETAINING WALL (after Washington State DOT Requirements)

a. Description

The contractor shall construct geotextile retaining walls in accordance with the details shown in the plans, these special provisions, or as directed by the engineer.

b. Materials

Geotextiles and Thread for Sewing

The material shall be woven or nonwoven geotextile consisting 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 polymers shall be polyolephins, polyesters, or polyamides. The

material shall be free of defects and tears. The geotextile shall conform to the properties as indicated in tables 18 and 19. The geotextile shall be free from any treatment or coating which might adversely alter its physical properties after installation.

Thread used shall be high strength polypropylene, polyester, or Kevlar™ thread. Nylon threads will not be allowed. The thread used must also be resistant to ultraviolet radiation if the sewn seam is exposed at the wall face.

Geotextile Approval and Acceptance

The contractor shall submit to the engineer a manufacturer's certificate of compliance which shall include the following information:

Manufacturer's name and current address, full product name, and geotextile polymer type(s).

If more than one style, merge, or product code number (i.e., this number being representative of a geotextile whose properties are different from a geotextile with the same product name and different style, merge or product code number) has been produced under the same product name, the style, merge or product code number of the geotextile to be approved must also be specified. If the geotextile has not been previously tested for source approval, the contractor shall submit sample(s) of the geotextile for approval by the engineer. Source approval will be based on conformance to the applicable values from tables 18 and 19. Each sample shall have minimum dimensions of 1.5 yd (1.37 m) by the full roll width of the geotextile. A minimum of 6 yd² (5 m²) of geotextile shall be submitted to the engineer for testing. The geotextile machine direction shall be marked clearly on each sample submitted for testing. The machine direction is defined as the direction perpendicular to the axis of the geotextile roll.

The geotextile samples shall be cut from the geotextile roll with scissors, sharp knife, or other suitable method which produces a smooth geotextile edge and does not cause geotextile ripping or tearing. The samples shall not be taken from the outer wrap of the geotextile nor the inner wrap of the core.

If the geotextile seams are to be sewn at the factory, at least one sewn sample, with a minimum of 2 yd (1.83 m) of seam length per sample and with a minimum of 18 in (457 mm) of geotextile width on each side of the seam, shall also be submitted for each geotextile direction (i.e., machine or cross-machine direction) proposed to be sewn.

Table 18. Minimum properties required for geotextiles used in geotextile retaining walls.

<u>Geotextile Property</u>	<u>Test Method</u>	<u>Minimum Geotextile¹ Property Requirements</u>
Water permeability	ASTM D-4491	* _____ cm/sec
AOS	ASTM D-4751	* _____ mm/maximum
Grab Tensile Strength ² , min. in machine and cross machine direction	ASTM D-4632	* _____ lb (kN)
Burst Strength ²	ASTM D-3786	* _____ psi (kN/m ²)
Puncture Resistance ²	ASTM D-4833	* _____ lb (kN)
Tear Strength ² , min. in machine and cross machine direction	ASTM D-4533	* _____ lb (kN)
Ultraviolet (UV) Radiation Stability (% Strength Retained)	ASTM D-4355	* _____ %
Seam Breaking ³ Strength	ASTM D-4884	* _____ lb/in (kN/m)

*Project specific values

¹ All geotextile properties are minimum average roll values (i.e., the test results for any sampled roll in a lot shall meet or exceed the minimum values in the table).

² Based on constructability and survivability requirements.

³ Applies only to seams perpendicular to the wall face (must be equal to or greater than design strength required in table 19).

Table 19. Wide strip tensile strength required for the geotextile used in geotextile retaining walls.

Surcharge Conditions	Wall Locations	Distance from top of Wall	Minimum Wide Strip Tensile Strength ^f for Geotextile Polymer Type (ASTM D-4595)	
			Strength	Polymer
XXXXX	XX	XX	XX	XX

Note: These geotextile strengths are for a vertical geotextile layer spacing of _____. These geotextile strengths are minimum average roll values (i.e., the test results for any sampled roll in a lot shall meet or exceed the minimum values shown in the table).

Acceptance Samples

Samples will be randomly taken by the engineer at the job site to confirm that the geotextile meets the property values specified. The contractor shall provide a manufacturer's certificate of compliance to the Engineer which includes the following information about each geotextile roll to be used:

- . Manufacturer's name and current address.
- . Full product name.
- . Style, merge, or product code number.
- . Geotextile roll number.
- . Geotextile polymer type.
- . Certified test results.

Approval will be based on testing of samples from each lot. A "lot" shall be defined for the purposes of this specification as all geotextile rolls within the consignment (i.e., all rolls sent to the project site) which were manufactured at the same manufacturing plant, have the same product name, and have the same style, merge, or product code number. A minimum of 14 calendar days after the samples have arrived at the engineer's office will be required for this testing. If the results of the testing show that a geotextile lot, as defined, does not meet the properties required in tables 18 and 19, the roll or rolls which were sampled shall be rejected. Two additional rolls from the lot previously

tested will then be selected at random by the engineer for sampling and retesting. If the retesting shows that either or both rolls do not meet the required properties, the entire lot shall be rejected. All geotextile which has the defects, deterioration, or damage, as determined by the Engineer, will also be rejected. All rejected geotextile shall be replaced at no cost to the project.

If the geotextile samples tested for the purpose of source approval came from the same geotextile lot as proposed for use at the project site, acceptance will be by manufacturer's certificate of compliance only.

Approval of Seams

If the geotextile seams are to be sewn in the field, the contractor shall provide a section of sewn seams before the geotextile is installed which can be sampled by the engineer. The seam sewn for sampling shall be sewn using the same equipment and procedures as will be used to sew the production seams. The seams sewn for sampling must be at least 2 yd (1.83 m) in length. If the seams are sewn in the factory, the Engineer will obtain samples of the factory seam at random from any of the rolls to be used.

Shotcrete Wall Facing

(Appropriate shotcrete specifications including gradation requirements, proportioning concrete, and shotcrete testing).

c. Construction Requirements

Shipment and Storage of Geotextiles

During periods of shipment and storage, the geotextile shall be kept dry at all times and shall be stored off the ground. Under no circumstances, either during shipment or storage, shall the materials be exposed to sunlight, or other form of light which contains ultraviolet rays, for more than 5 calendar days.

Wall Construction

The base for the wall shall be graded to a smooth, uniform condition free from ruts, potholes, and protruding objects such as rocks or sticks. The geotextile shall be spread immediately ahead of the covering operation.

Wall construction shall begin at the lowest portion of the excavation and each layer shall be placed horizontally as shown in the plans. Each layer shall be completed entirely before the next layer is started. Geotextile splices transverse to the wall face will be allowed provided the minimum overlap is 2 ft (610 mm) or the splice is sewn together. Geotextile splices parallel to the

wall face will not be allowed. The geotextile shall be stretched out in the direction perpendicular to the wall face to ensure that no slack or wrinkles exist in the geotextile prior to backfilling.

Under no circumstances shall the geotextile be dragged through mud or over sharp objects which could damage the geotextile. The fill material shall be placed on the geotextile in such a manner that a minimum of 4 in (102 mm) of material will be between the vehicle or equipment tires or tracks and the geotextile at all times. Particles within the backfill material greater than 3 in (76 mm) in size shall be removed. Turning of vehicles on the first lift above the geotextile will not be permitted. End-dumping fill directly on the geotextile will not be permitted.

Should the geotextile be torn or punctured or the overlaps or sewn joints disturbed as evidenced by visible geotextile damage, subgrade pumping, intrusion, or distortion, the backfill around the damaged or displaced area shall be removed and the damaged area repaired or replaced by the contractor at no cost to the State. The repair shall consist of a patch of the same type of geotextile which replaces the ruptured area. All geotextile within 1 ft (305 mm) of the ruptured area shall be removed from the smooth geotextile edge in such a way as to not cause additional ripping or tearing. The patch shall be sewn onto the geotextile.

If geotextile seams are to be sewn in the field or at the factory, the seams shall consist of two parallel rows of stitching. The two rolls of stitching shall be 0.5 in (12.7 mm) apart with a tolerance of ± 0.25 in (6.4 mm) and shall not cross, except for restitching. The stitching shall be a lock-type stitch. The minimum seam allowance, i.e., the minimum distance from the geotextile edge to the stitch line nearest to that edge, shall be 1.5 in (38.1 mm) if a flat or prayer seam, Type SSa-2, is used. The minimum seam allowance for all other seam types shall be 1 in (25.4 mm). The seam, stitch type, and the equipment used to perform the stitching shall be as recommended by the manufacturer of the geotextile and as approved by the engineer.

The seams shall be sewn in such a manner that the seam can be inspected readily by the engineer. The seam strength will be tested and shall meet the requirements stated in this Special Provision.

A temporary form system shall be used to prevent sagging of the geotextile facing elements during construction. A typical example of a temporary form system and sequence of wall construction required when using this form are shown in the plans.

Pegs, pins, or the manufacturer's recommended method, in combination with the forming system shall be used as needed to hold the geotextile in place until the specified cover material is placed.

The wall backfill shall be placed and compacted in accordance with the wall construction sequence shown in the plans. The minimum compacted backfill lift thickness of the first lift above each geotextile layer shall be 4 in (102 mm). The maximum compacted lift thickness anywhere within the wall shall be 6 in (152 mm) or one half of the geotextile layer spacing, whichever is least.

Each layer shall be compacted to 95 percent of maximum density as per AASHTO T-99. The water content of the wall backfill shall not deviate above the optimum water content by more than 3 percent. Sheepsfoot rollers or other rollers with protrusions shall not be used. The required compaction shall be achieved with lightweight vibratory roller compactors approved by the engineer. Compaction within 3 ft (0.91 m) of the wall face, as well as large vibratory rollers shall be achieved using light mechanical tampers approved by the engineer and shall be done in a manner to cause no damage or distortion to the wall facing elements or reinforcing layer.

If corners must be constructed in the geotextile wall due to abrupt changes in alignment of the wall face as shown in the plans, the method used to construct the geotextile wall corner(s) shall be submitted to the engineer for approval at least 14 calendar days prior to beginning construction of the wall. The corner must provide a positive connection between the sections of the wall on each side of the corner such that the wall backfill material cannot spill out through the corner at any time during the design life of the wall. Furthermore, the corner must be constructed in such a manner that the wall can be constructed with the full geotextile embedment lengths shown in the plans in the vicinity of the corner.

The base of the excavation shall be completed to within ± 3 in (76 mm) of the staked elevations unless directed by the engineer. The external wall dimensions shall be placed within ± 2 in (51 mm) of that staked on the ground. Each layer and overlap distance shall be completed to within ± 1 in (25.4 mm) of that shown in the plans.

The maximum deviation of the face from the batter shown in the plans shall not be greater than 3 in (76 mm) for permanent walls and 5 in (127 mm) for temporary walls. The face batter measurement shall be made at the midpoint of each wall layer. Each wall layer depth shall be completed to within ± 1 in (25.4 mm) of that shown in the plans.

If the wall is to be a permanent structure, the entire wall face shall be coated with a reinforced shotcrete facing as detailed in the plans and as described in this Special Provision.

Placement of Shotcrete Wall Facing

(Includes qualification of craftsman, equipment, placing wire reinforcement, placing concrete, and curing specification).

d. Measurement

Geotextile retaining wall will be measured by the square foot (square meter) of face of completed wall. Shotcrete wall facing will be measured by the square foot (square meter) of the completed area of the facing.

e. Payment

The unit contract prices per square foot (square meter) for "Geotextile Retaining Wall" and "Shotcrete Wall Facing", per ton (tonne) for "Gravel Borrow Incl. Haul" and per cubic yard (cubic meter) for "Structure Excavation Class A" shall be full pay for furnishing all labor, tools, equipment, and materials necessary to complete the work in accordance with these specifications, including compaction of the backfill material and the temporary forming system.

Instructions for This Special Provision

This special provision for geotextile retaining walls does not provide a complete design of the geotextile wall. It does provide material and construction requirements which are true for all geotextile walls. The project designer is responsible for all geotextile wall designs. Therefore, the designer must provide the information needed to complete the geotextile wall design as presented in the plans attached with the special provisions. The information which must be provided by the designer is as follows:

1. Geotextile wall base width.
2. Geotextile wall embedment depth.
3. Geotextile wall face batter.
4. Geotextile layer vertical spacing.
5. Geotextile wall backfill material requirements.
6. Maximum slope of fill above the geotextile wall.
7. Minimum geotextile wide strip strength requirements.

The State should, of course, provide a wall plan and profile for each geotextile wall proposed for a given contract.

Please note that the unit of measure for the geotextile retaining wall and the shotcrete wall facing is per square foot (square meter) of wall face. This unit of measure should always be used.

REFERENCES

1. AASHTO, "Standard Specifications for Highway Bridges," 13th ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1983.
2. Mitchell, J.K. and Villet, W.C.B., "Reinforcement of Earth Slopes and Embankments", NCHRP Report No. 290, Transportation Research Board, Washington, D.C., 1987.
3. Devata, M.S., "Geogrid Reinforced Embankments with Steep Slopes," Proc. of Conf. on Polymeric Grid Reinforcement, Sponsored by the Science and Engineering Research Council, p82, Telford, London, March, 1984.
4. The Reinforced Earth Company, Arlington, Virginia promotional publication, 1988.
5. Enka Systems, promotional publication.
6. Chang, J.C., Beaton, J.L., and Forsyth, R.A. "Design and Field Behavior of the Reinforced Earth Embankment - California Highway 39" presented at the Jan. 21-25, 1974 ASCE National Water Resources Engineering Meeting, Los Angeles, California, 1974.
7. Jewell, R.A., "Material Requirements for Geotextiles and Geogrids in Reinforced Slope Applications", Proc. 23rd Int. Man-Made Fibres Congress, Dornbirn, Austria, Sept., 1984.
8. Hueckel, S.M. and Kwasniewski, J., "Scale Model Tests on the Anchorage Values of Various Elements Buried in Sand," Proc. 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, 1961.
9. Schlosser, F., Juran, I. and Jacobsen, H.M., "Soil Reinforcement" General Report, 8th European Conf. on Soil Mechanics and Foundation Engineering, Helsinki, 1983.
10. Phan, T.L., Segrestine, P., Schlosser, F., and Long, N.T., "Stability Analysis of Reinforced Earth Walls by Two Slip Circle Methods", Proc. Int. Conf. on Soil Reinforcement, pp 119-123, Paris, 1979.
11. Shen, C.K., Bang, S., Romstad, J.M., Kulchin, L. and Denatale, J.S., "Field Measurements of an Earth Support System", J. Geot. Eng. Div., Vol. 107, GT12, ASCE, 1981.
12. Schlosser, F., "Analogies et Differences Dans le Comportement et le Calcul des Ouvrages de Soutènement en Terre Armee et par Clouge du Sol", Annals de L'Institut Technique du Batiment et des Travaux Publics, No. 418, 1983.

13. DiMaggio, J., "Mechanically Stabilized Earth: Walls and Slopes", Internal Memo Federal Highway Administration, Washington, D.C., 1988.
14. Weatherby, D.E., "Tiebacks", Report No. FHWA/RD-82/047, Federal Highway Administration, Washington, D.C., 1982
15. Elias, V., and Juran, I., "Soil Nailing", Report to Federal Highway Administration, Washington, D.C., 1988.
16. Bastick, M., Schlosser, F., Amar, S., and Canepa, Y., "Monitoring of a Prototype Reinforced Earth Wall with Short Strips," to be published in the Proc. of the 12th ICSMFE, Rio de Janeiro, 1989.
17. Terre Armee Internationale, Finite Element Study of Reinforced Earth Structures, Rosalie, Internal Report, 1983.
18. Meyerhof, G.G., "The Bearing Capacity of Foundations Under Eccentric and Inclined Loads", Proc. 3rd Int. Conf. on Soil Mechanics and Foundation Engineering, pp 440-445, Zurich, Switzerland, 1953.
19. Segrestin, P., Bastic, M.J., Seismic Design of Reinforced Earth Retaining Walls: The Contribution of Finite Element Analysis," Proc. of the Int. Symp. on Theory and Practice of Earth Reinforcement, Kyushu, Japan, Oct, 1988.
20. Seed, H.B., and Mitchell, J.K., "Earthquake Resistant Design of Reinforced Earth Walls", Internal Study for the Reinforced Earth Company, Progress Report, Berkeley, California, 1981.
21. Seed, H.B. and Whitman, R.V., "Design of Earth Retaining Structures for Dynamic Loads", Proc. of the ASCE Specialty Conf., Lateral Stresses and Earth Retaining Structures, Cornell University, pp 103-147, Ithaca, NY, June, 1970.
22. Christopher, B.R., Unpublished, STS Consultants, Ltd. Report on Task C Field Test Wall Construction - Prepared for FHWA, Washington, D.C., Contract No. DTFH61-84-C-00073, "Behavior of Reinforced Soil", 1987.
23. Wichter, L., Risseeuw, P., and Gay, G., "Large Scale Tests on the Bearing Behavior of Woven Reinforced Earth Wall", Proc. of the 3rd Int. Conf. on Geotextiles, Vol. IV, pp 1073-1078, Vienna, Austria, 1986.
24. Christopher, B.R., and Holtz, R.D., "Geotextile Design and Construction Guidelines Manual," Federal Highway Administration, Washington, DC, 1988.

25. Christopher, B.R. and Holtz, R.D., Geotextile Engineering Manual, STS Consultants, Ltd., report No. FHWA-TS-86/203, pp 1044, Washington, DC, 1984.
26. Schmertmann, G.R., Chouery-Curtis, V.E., Johnson, R.D., and Bonaparte, R., "Design Charts for Geogrid -Reinforced Soil Slopes", Proc. Geosynthetics, Vol. 1, pp 108-120, New Orleans, 1987.
27. Cheney, R.S., and Chassie, R.G., "Soils and Foundation Workshop Manual", FHWA, Washington, D.C., 1982.
28. Fukuoka, M., "Fabric Retaining Walls", Proc. 2nd Int. Conf. on Geotextiles, pp 575-580, Las Vegas, 1982.
29. Juran, I., Schlosser, F., Legeay, G., and Long, N.T., "Experimentation en Vraid Grandeur sur un mur Soumis a des Surcharges en Tete a Dunkerque", Proc. Int. Conf. on Soil Reinforcement: Reinforced Earth and Other Techniques, Vol II, Paris, 1979.
30. Murray, R.T., "Studies of the Behavior of Reinforced and Anchored Earth", Ph.D. Thesis, Harriot-Watt University, Edinburgh, 1983.
31. Juran, I., "Behavior of Reinforced Soil Structures", Partial Report No.2, FHWA Contract No. 61-84-C-00078, 1985.
32. Winkler, E., "Die Lehre von Elastizitat und Festigkeit", Prague, 1867.
33. Cheney, R.S., "Permanent Ground Anchors," Federal Highway Administration Report No. FHWA-DP-68-1R, November, 1984.
34. Jewell, R.A., Milligan, G.W., Sarsby, R.W., and Dubois, D., "Interaction Between Soil and Geogrids", Proc. Symp. on Polymer Grid Reinforcement in Civil Engineering, Science and Engineering Research Council and Netlon, Ltd., March, 1984.
35. Gouvenot, D., and Bustamante, M.G., "Mesures In-situ Sur Les Ouvrages Maritimes de Soutenement", Report, Annales de L'Institut Technique du Batiment et des Travaux Publique, Sept, 1979.
36. Juran, I., Beech, J., and Delaure, E., "Experimental Study of the Behavior of Nailed Soil Retaining Structures on Reduced Scale Models," Proc., Int. Symp. on In-situ Soil and Rock Reinforcements, Paris, 1984.
37. Cartier, G., and Gigan, J.P., "Experiments and Observations on Soil Nailing Structures," Proc., Seventh Conf. of the ECSMFE, Helsinki, Finland, 1983.

38. Gassler, G., and Gudehus, G., "Soil Nailing: Some Mechanical Aspects of In-situ Reinforced Earth," Proc., 10th ICSMFE, Vol. 3, pp 665-670, Stockholm, Sweden, 1981.
39. Juran, I., and Elias, V., "Soil Nailed Retaining Structures: Analysis of Case Histories", ASCE, Special Geotechnical Publication No. 12, pp 232-245, 1987.
40. Plumelle, C., "Experimentation en Vraie Grandeur d'une Paroi Clouee," Revue Francaise de Geotechnique, No. 40, pp 45-50, 1987.
41. Juran, I., Beech, J., "Analyse Theorique du Comportement d'un Soutenement en Sol Cloue", Proc. Int. Conf. on In-situ Reinforcement of Soils and Rocks, pp 301-307, Paris, 1984.
42. Juran, I., Baudrand, G., Farrag, F, and Elias, V., "Kinematical Limit Analysis Approach for the Design of Nailed Soil Retaining Structures," Proc. Int. Geotechnical Symp. on Theory and Practice of Earth Reinforcement, Fukuoka Kyushu, Japan, 1988.
43. Terzaghi, K., "Theoretical Soil Mechanics", John Wiley & Sons, Inc., New York, 1943.
44. Terzaghi, K., and Peck, R.B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., New York, 1948 and 1967.
45. Tschebotarioff, G.P., "Foundations, Retaining and Earth Structures", McGraw-Hill Book Company, 1951.
46. Stocker, M.F., Korber, G.W., Gassler, G., and Gudehus, G., "Soil Nailing", Proc. Int. Conf. on Soil Reinforcement, Vol. 2, pp 469-474, Paris, 1979.
47. Steward, J.E. and Mohny, J., "Trial Use, Results and Experience Using Geotextiles for Low-Volume Forest Roads," Proc., 2nd Int. Conf. on Geotextiles, Vol. 2 pp 335-340, Las Vegas, Nevada, August 1982.
48. Yako, M.A., and Christopher, B.R., "Polymerically Reinforced Retaining Walls and Slopes in North America", The Application of Polymeric Reinforcement in Soil Retaining Structures, P.M. Jarrett and A. McGown, eds. pp 239-282, 1987.
49. Dunnicliff, J., and Green, G.E., "Geotechnical Instrumentation for Monitoring Field Performance," J. Wiley & Sons, New York, 1988.

50. Bell, J.R., and Steward, J.E., "Construction and Observations of Fabric Retained Soil Walls," Proc. Int. Conf. on the Use of Fabrics in Geotechnics, Vol. 1, pp 123-128, Paris, France, April, 1977.
51. Mohny, J., "Fabric Retaining Wall, Olympic National Forest," Highway Focus, Federal Highway Administration, Vol. 9. No. 1, pp 88-103, May, 1977.
52. Chassie, R.G., "Geotextile Retaining Walls, Some Case History Examples," prepared for presentation at the NW Roads and Streets Conf., Corvallis, Oregon, February, 1984.
53. Al-Hussaini, M.M., "Field Experiment of Fabric Reinforced Earth Wall," Proc. Int. Conf. on the Use of Fabrics in Geotechnics, Vol. 1, pp 119-121, April, 1977.
54. Al-Hussaini, M.M., and Perry, E.B., "Effect of Horizontal Reinforcement on Stability of Earth Masses", Technical Report No. S-76-11, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1976.
55. Douglas, E.G., "Design and Construction of Fabric Reinforced Retaining Walls by New York State," Transportation Research Board, Transportation Research Record 872, pp 32-37, Washington, D.C., 1982.
56. Bell, J.R., Barrett, R.K. and Ruckman, A.C., "Geotextile Earth-Reinforced Retaining Wall Tests: Glenwood Canyon, Colorado," Transportation Research Board, Transportation Research Record 916, pp 59-69, Washington, D.C., 1983.
57. Barrett, R.K., "Geotextiles in Earth Reinforcement," Geotechnical Fabrics Report, pp 15-19, March/April, 1985.
58. Forsyth, R.A. and Bieber, D.A., "La Honda Slope Repair with Geogrid Reinforcement," Proc. of the Symp. on Grid Reinforcement in Civil Eng., Paper No. 2.2, London, U.K., 1984.
59. Berg, R.R., LaRochelle, P., Bonapart, R., and Tanguay, L., "Gaspe' Peninsula Reinforced Soil Seawall Case History," Proc. ASCE Symp. on Soil Improvement, Atlantic City, New Jersey, April, 1987.
60. Anderson, R.P., "Soil Reinforcement Objective: Polymer Geogrid Replaces Galvanized Metal Strips in Concrete-Faced Retaining Walls," Geotechnical Fabrics Report, January/February, 1986.

61. Scott, J.D., et al, "Design of the Devon Geogrid Test Fill," Proc. Geosynthetics '87 Conf. New Orleans, Louisiana, February, 1987.
62. Jaber, M., "Behavior of Reinforced Soil Walls in Centrifuge Model Tests", Dissertation submitted in partial satisfaction of the requirements for the degree of Doctor of Philosophy, University of California, Berkeley, Department of Civil Engineering, March, 1989.
63. Alimi, Bacot, J., Lareal, P., Long, N.T., and Schlosser, F., "Etude de l'Adherence Sols-armatures," Proc. 9th Int. Conf. on Soil Mechanics and Foundation Engineering, Vol. I, pp 11-14, Tokyo, 1977.
64. ASTM, "Soil and Rock; Building Stones," Annual Book of ASTM Standards, Section 4, Vol. 04.08, American Society for Testing and Materials, Philadelphia, PA, 1988.
65. Rowe, R.K., and Davis, E.H., "The Behavior of Anchor Plates in Sand," Geotechnique 32, No. 1, pp 25-41, 1982.
66. Bloomfield, R.A. "Proposed Design Modifications - VSL Retaining Earth," VSL Corporation, January, 1984.
67. Nielsen, M.R., "Pullout Resistance of Welded Wire Mats Embedded in Soil", Masters Thesis, Utah State University, Logan, UT, 1983.
68. Murray, R.T., "Studies of the Behavior of Reinforced and Anchored Earth," Ph.D. Thesis, Heriot-Watt University, Edinburgh, 1983.
69. Jewell, R.A., "Some Effects of Reinforcement on the Mechanical Behavior of Soils," Ph.D. Thesis, University of Cambridge, 1980.
70. Juran, I., and Chen, Ch.L., "Soil-Geotextile Pullout Interaction Properties: Testing and Interpretation", Transportation Research Board, 67th Annual Meeting, Paper No. 87-0159, 1988.
71. Littlejohn, G.S. and Bruce, D.A., "Rock Anchors State-of-the-Art," Part I: Design and Part II: Construction, Ground Engineering, May, 1975.
72. Peck, R.B., "A Study of the Comparative Behavior of Friction Piles", Highway Research Board Special Report No. 36, p 72, 1958.
73. Woodward, R.J., Lundgren, R., and Boitono, J.D., "Pile Loading Tests in Stiff Clays", Proc., 5th ICSMFE, Vol. 2, pp 177-184, 1961.

74. Louis, C., "Theory and Practice in Soil Nailing Temporary or Permanent Works," ASCE Annual Conference, Boston, Oct., 1986.
75. Louis, C., "Nouvelle Methode de Soutennement des Sols en Deblais," Revue Travaux No. 533, 1981.
76. Elias, V., and Juran, I., Draft Manual of Practice for Soil Nailing, Prepared for U.S. Department of Transportation, FHWA, Contract DTFH-61-85-C-00142, May, 1988.
77. LCPC-SETRA, "Reglis de Justification des Foundations err Pioux", 1985.
78. Singh, A., and Mitchell, J.K., "General Stress-Strain-Timi Function for Soils", 1968, J. SMFD., ASCE, Vol. 94, SMI, pp 21-46.
79. Bustamante, M., "Capacite de' Ancrage et Comportement des Tirants Injectes, Scelles dans une Angile Plastique," These' Docteur-Ingenieur ENPC, Paris.
80. Wilding, M.A. and Ward, I.M., "Tensile Creep and Recovery in Ultrahigh Modulus Linear Polyethylene", J. Polymer, 19, pp 969-976, 1978.
81. McGown, A., Andrawes, K.Z., and Kabir, M.H., "Load-Extention Testing of Geotextiles Confined in Soil", Proc. Second International Conference in Geotextiles, Vol. 3, pp 793-798, Las Vegas, Nevada, 1982.
82. John, N., Johnson, P., Ritson, R., and Petley, D., "Behavior of Fabric Reinforced Soil Walls, Proceedings of the Second International Conference on Geotextiles, Las Vegas, Nevada, IFAI, St. Paul, MN, 1982.
83. Jarrett, P.M. and McGown, A., "The Application of Polymeric Reinforcement in Soil Retaining Structures," Proc. NATO Advanced Research Workshop, Kluwer Academic Publishers, Kingston, Ontario, Canada, 1987.
84. Adib, Mazen, E., "Internal Lateral Earth Pressure in Earth Walls", Dissertation submitted in partial satisfaction of the requirements of the degree of Doctor of Philosophy, Department of Civil Engineering, University of California, Berkeley, 1988. Note: This research was supported primarily by the Institute for Transportation Studies, University of California, 1988.
85. Schmertmann, G., Chew, S.H., and Mitchell, J., "Finite Element Modeling of Reinforced Soil Wall Behavior," Geotechnical Engineering Report No. UCB/GT/89-01, University of California, Berkeley, September, 1989.

86. Bastick, M., Schlosser, F., Amar, S., and Canepa, Y., "Instrumentation d'un Mur Experimental en Terre Armee a Armatures Courtes", Proc. 12th ICSMFE, 1988. To be published.
87. Bastick, M., and Schlosser, F., "Comportement et Dimensionnement Dynamique des Ouvrages en Terre Armee", 1er Coll. Nat. de Genie Parasismique, Saint-Remy-les-Chevreuse France, January, 1986.
88. Tomlinson, M.J., "The Adhesion of Piles Driven in Clay Soils", Proc., 4th ICSMFE, Vol. 2, pp 61-71, 1957.
89. Ward, I.M., "The Orientatin of Polymers to Produce High Performance Materials," Proc. Symp. on Polymer Performance in Civil Engineering, London, 1984.
90. Juran, I., Shafiee, S., and Schlosser, F., "Numerical Study of Nailed Soil Retaining Structures," Proc., 11th ICSMFE, Vol. 4, pp 1713-1717, San Francisco, 1985.
91. Shafiee, S., "Simulation Numerique du Comportement des Sols Cloues: Interaction Sol-Renforcement et Comportement de L'ouvrage", Ph.D. Diss, ENPC, Paris, 1986.
92. Frank, R., Guenot, A., and Humbert, P., "Numerical Analysis of Contact in Geomechanics," Proc., 4th Int. Conf. on Numerical Methods in Geomechanics, Edmonton, 1982.
93. Zaman, M.M., Desai, C.S., and Brumm, E.C., "Interface Model for Dynamic Soil-Structure Interaction", Journal of Geotechnical Engineer Division, ASCE, Vol. 110, SM9, pp 1257-1273, September, 1984.

