MECHANICALLY STABILIZED EARTH SYSTEM INSPECTION MANUAL

GEOTECHNICAL ENGINEERING MANUAL
GEM-16
Revision #3
AUGUST 2015
PREFACE

The purpose of this manual is to provide New York State Department of Transportation Construction Inspectors with a quick and easy-to-use set of inspection guidelines for the installation of Mechanically Stabilized Earth Systems (MSES) that utilize inextensible metallic reinforcements i.e., reinforcing strips, bar mats or wire mesh. The enclosed check lists are intended to serve as reminders to inspectors of important aspects for the successful construction of MSES.

Prior to using this manual, inspectors should become familiar with the general concepts of MSES. This information can be found in Chapters 1, 2 & 3 of the Federal Highway Administration Publication No. FHWA-NHI-00-043, titled “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines”, which is included as Appendix 1 of this manual. In addition, it is imperative that Engineers-In-Charge obtain from the Contractor and review copies of the MSES designers/suppliers Construction Manual. The MSES Designer/Supplier’s Construction Manual contains detailed information for the construction of a particular MSES and provides the necessary guidance during the construction inspection process.

The user of this manual is encouraged to make copies of the enclosed checklists as needed.
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Chapters 1, 2 & 3 of:
Federal Highway Administration Publication No. FHWA-NHI-00-043
"Mechanically Stabilized Earth Walls and Reinforced Slopes
Design and Construction Guidelines"
   1. Introduction ............................................................................................................ A1
   2. Systems and Project Evaluation ............................................................................. A8
   3. Soil Reinforcement Principles and System Design Properties ............................. A50
INTRODUCTION

The New York State Department of Transportation first used Mechanically Stabilized Earth System (MSES) in 1978. At that time, the Reinforced Earth Company and, to a much lesser extent, VSL Retained Earth, enjoyed the distinction of being the only designers/suppliers of this type of system for the Department. They were relatively new and expanding companies with a sole source product and always had personnel available on-site to ensure that their system was constructed properly. MSES were also very cost effective, usually representing a cost saving of 20 to 40 percent over conventional reinforced concrete walls.

Today, the situation is much different. The Department still reaps the benefits of cost-effective and well-designed MSES by both the Reinforced Earth Company and VSL Retained Earth. However, today's economy, and an increase in the number of designers/suppliers of MSES, has made it necessary for these companies to shift their priorities to be more competitive. Because of this priority shift, we believe the construction support that was once enjoyed has declined and it has become increasingly more important for the Department to become more educated and take a more active role in the construction control of MSES. To that end, this Inspection Manual has been developed in an effort to make the job of construction inspection easier.
I. GLOSSARY OF TERMS

1. **Backfill** – Any suitable material, meeting the requirements of Section 554, Mechanically Stabilized Earth System, §554-2.09 of the Standard Specification, which, when placed in conjunction with the reinforcing strips or mesh and the facing panels, comprise the reinforced volume.

2. **Connectors** – Galvanized metal tie strips or welded clevis loops cast into the back of a facing panel to which the reinforcing strips or mesh are attached.

3. **Coping** – Precast or cast-in-place concrete cap, which is placed on top of the MSES facing panels.

4. **Facing Panels** – Precast-reinforced-concrete units which are part of the reinforced volume that forms the outside face of the MSES and are attached by means of the connectors to the reinforcing strips or mesh.

5. **Fasteners** – Bolts, washer, and nuts or connecting rods used to attach the reinforcing strips or mesh to the connectors.

6. **Joint Filler** – Material used to fill the vertical, angled and horizontal joints between the facing panels, consisting of either polyether foam or geotextile for all joints to prevent soil migration and resin-bonded corkboard or rubber bearing pads for the horizontal and angled joints for bearing.

7. **Leveling Pad** – A concrete pad or footing, usually unreinforced, which serves as a flat starting surface for placing the initial coursed of facing panels.

8. **Reinforced Volume** – A system of facing panels, reinforcing strips or mesh and backfill, which, when constructed together according to specification, form one coherent mass.

9. **Reinforcing Mesh** – A system of longitudinal and cross galvanize wires or bars, spaced and welded together at specified intervals, forming mats of specified length, which are attached to connectors and internally reinforce and stabilize the backfill.

10. **Reinforcing Strips** – Galvanized or epoxy-coated ribbed steel strips of a specified length, width and thickness, which are attached to connectors and internally reinforce and stabilize the backfill.
III. CHECKLISTS

A. **Preconstruction**

Review and become familiar with the State plans and specifications and the MSES working drawings, including the construction sequencing.

Check to make sure that all required manuals and directives for the construction of the wall(s) have been supplied. This includes:

- Standard Specification, Section 554, Mechanically Stabilized Earth System,
- MSES Construction Manual, supplied by the MSES designer/supplier,
- The current NYSDOT Geotechnical Control Procedure (GCP) issued for the Control of Granular Material, supplied by the Regional Geotechnical Engineer,
- The current NYSDOT Directive for the Acceptance/Rejection Procedure MSES Backfill, supplied by the Regional Geotechnical Engineer.

Preliminary evaluation for potential source(s) of backfill material for electrochemical properties may be undertaken by the State at the discretion of the Regional Geotechnical Engineer.

**Preconstruction Meeting:**

A preconstruction meeting with the Regional Geotechnical Engineer and/or the Geotechnical Engineering Bureau representative is encouraged. This meeting should be used by the Engineer-In-Charge and his inspectors to discuss and clarify any items of concern about the proposed MSES.
B. **Materials, Handling and Storage**

Ensure that the Contractor has an area to store and/or stockpile materials supplied by the MSES designer/supplier. This area should be clean, dry and capable of being covered.

Ensure that all component materials necessary to construct the wall(s) supplied by the MSES designer/supplier have been shipped to the site. This includes:

- Facing panels, marked as to their type and stamped as being inspected by the NYSDOT Materials Bureau. Facing panels which are not being directly unloaded form a truck and placed in the structure must be restacked and stored in accordance with the Construction Manual.

- Reinforcing strips or reinforcing mesh, bundled, labeled and of the length specified by the designer/supplier. Reinforcing strips and reinforcing mesh are to be stored off the ground.

- Fasteners, bundled or boxed and labeled.

- Joint fillers, consisting of:
  
  For all Joints:
  - Polyether foam strips, properly sized and shipped in bags.
    - or -
  - Geotextile, shipped in rolls.

  In addition, for bearing surfaces:
  - Preformed bearing pads, boxed and labeled.
    - or -
  - Resin-bonded corkboard, properly sized and shipped on pallets.

Check the items that must be fabricated by the Contractor as shown in the appropriate Construction Manual. Make sure there are sufficient quantities to perform the work.
C. **Construction**

1. **Site Preparation:**

   - Concrete leveling pad area is excavated to the proper dimensions and compacted according to Section 554, Mechanically Stabilized Earth System, of the Standard Specifications.

   - Underdrains, where required, are installed.

   - Concrete leveling pad is poured within tolerance.

   - Concrete leveling pad is cured properly.

   - Control line for the front face of the MSES is established on the leveling pad.
2. *Initial Course Construction:*

Proper layout and setting other initial course of panels for any MSES is crucial. Extra effort by the Contractor at the beginning of wall construction should result in trouble-free and rapid construction of a wall. The inspector should ensure that the Contractor is paying close attention to vertical and horizontal alignments, batter and centering when placing each panel.

Initial course construction is straightforward and consists of setting a series of alternating half-height and full-height panels along the proper alignment and to the proper batter. The general construction procedure is as follows and is repeated throughout the initial course construction:

Setting a Half-Height Panel:
- Set the first half-height panel at its proper location.
- Align the panel with the control line.
- Space the next half-height panel the proper lateral distance from the previous half-height panel using the spacing tool.
- Spacing tool left in place, ensuring proper distance.
- Batter of the half-height panel is set with wedges.

Setting a Full-Height Panel:
- Remove the spacing tool.
- Center the full-height panel between the half-height panels to ensure equal vertical or angled joint spacing.
- Batter of the full-height panel is set with wedges.
- Clamp the full-height panel to the half-height panels on each side.

For ease of inspection, checklists have been provided which break the initial course construction into Phases 1, 2 and 3. This is done so that there are breaks at critical junctures to ensure the construction of the wall is proceeding properly.
## Initial Course Construction – Phase 1 Checklist

<table>
<thead>
<tr>
<th>Construction Activity</th>
<th>Half-Height Panel Number*</th>
<th>Full-Height Panel Number*</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Set Half-Height Panel</strong></td>
<td>1</td>
<td>2</td>
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<tr>
<td>• Align the panel with the control line.</td>
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<tr>
<td>• Space half-height panel proper distance from the previous half-height panel using spacer bar.</td>
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<td>-</td>
</tr>
<tr>
<td>• Spacer bar left in place.</td>
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<td>-</td>
</tr>
<tr>
<td>• Batter is set with wedges.</td>
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<td>-</td>
</tr>
</tbody>
</table>

| Full-Height Panel Number*       | 1        | 2          |
|----------------------------------|---------------------------|
| • Remove spacer bar.             | -        | -          |
| • Center full-height panel between the half-height panels to ensure equal vertical joints. | -        | -          |
| • Batter is set with wedges.     | -        | -          |
| • Clamp full-height panel to the half-height panels. | -        | -          |

* Numbers in the Table represent the order of placement of the half- and full-height panels.

Upon completion of Phase 1, the horizontal alignment of the completed wall section is checked and adjusted accordingly.
<table>
<thead>
<tr>
<th>Construction Activity</th>
<th>Half-Height Panel Number*</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
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<tr>
<td>Set Half-Height Panel</td>
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<tr>
<td>● Align half-height panel with control line.</td>
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<td>● Space half-height panel proper distance from the previous half-height panel using spacer bar.</td>
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<td>● Spacer bar left in place.</td>
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<td>● Batter is set with wedges.</td>
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<td>Set Full-Height Panel</td>
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<td>● Remove spacer bar.</td>
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<tr>
<td>● Center full-height panel between the half-height panels to ensure equal vertical joints.</td>
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<td>● Batter is set with wedges.</td>
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<tr>
<td>● Clamp full-height panel to the half-height panels.</td>
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</table>

* Numbers in the Table represent the order of placement of the half- and full-height panels.

Upon completion of Phase 2, the horizontal alignment of the completed wall section is checked and adjusted accordingly. Bracing of panels begins (See Construction Manual). Backfilling of panels begins (See Checklist, p. 15).
## Initial Course Construction – Phase 3 Checklist

<table>
<thead>
<tr>
<th>Construction Activity</th>
<th>Half-Height Panel Number*</th>
<th>13</th>
<th>14</th>
<th>15</th>
<th>Repeat to end</th>
</tr>
</thead>
<tbody>
<tr>
<td>Set Half-Height Panel</td>
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<tr>
<td>● Align the panel with the control line.</td>
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<tr>
<td>● Space half-height panel proper distance from the previous half-height panel using spacer bar.</td>
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<tr>
<td>● Spacer bar left in place.</td>
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<tr>
<td>● Batter is set with wedges.</td>
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<tr>
<td>Set Full-Height Panel</td>
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<tr>
<td>● Remove spacer bar.</td>
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<tr>
<td>● Center full-height panel between the half-height panels to ensure equal vertical joints.</td>
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<td>● Batter is set with wedges.</td>
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<td>● Clamp full-height panel to the half-height panels.</td>
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</tbody>
</table>

* Numbers in the Table represent the order of placement of the half- and full-height panels.

During Phase 3, the horizontal alignment of the completed wall section should be checked frequently and adjusted accordingly until the entire initial course has been completed. Bracing of panels continues (See Construction Manual). Backfilling of panels continues (See Checklist, p. 15).
3. **Backfilling – Phase 1:**

- Backfill is properly evaluated and documented in accordance with the current NYSDOT Directive titled “Acceptance/Rejection Procedure, MSES Backfill”.
- Backfill is properly stockpile and documented in accordance with the current NYSDOT Geotechnical Control Procedure titled “Procedures for Control of Granular Materials”.

Backfilling of the structure can commence, along with the remainder of the initial course construction, once 10 full-height panels have been installed, battered, aligned, braced, and the joints filled. The first backfill lift proceeds as follows:

- Backfill is placed up to the bottom row of facing-panel connectors and in the same direction as the wall construction. Backfill is sloped so that the toe of the slope originates at the back of the facing panel.
- Backfill is compacted as required. Large compaction equipment is kept a minimum of 3 ft. (1 m) away from the panels. Compaction within 3 ft. (1 m) of the wall is accomplished using small hand operated equipment.
- Check wall alignment, adjust accordingly and continue.

Once the first backfill lift has commenced, the reinforcing strips or mesh can be placed and the second backfill lift can be placed. These two operations can run concurrently, but must not interfere with one another.

- Reinforcing strips or mesh are placed on the compacted backfill and into the facing-panel connectors. The reinforcing strips or mesh must be perpendicular to the facing panels, unless otherwise specified.
- Reinforcing strips or mesh are attached to the facing-panel connectors in accordance with the Construction Manual.
- Backfill is placed up to the next row of facing-panel connectors and in the same direction as the wall construction. Backfill is spread parallel to and windrowed toward the facing panels and the free end of the reinforcing strips or mesh. Backfill is sloped so that the toe of the slope originates at the back of the facing panel.
- Backfill is compacted as required. Large compaction equipment is kept a minimum of 3 ft. (1 m) away from the panels. Compaction within 3 ft. (1 m) of the wall is accomplished using small hand operated equipment.
- Check the wall alignment, adjust accordingly and continue.
4. **Subsequent Course of Construction:**

Subsequent course construction can commence only after the backfill has been placed and compacted to the top of the half-height panels from the previous course. Subsequent course construction must begin at the wall end where the initial construction began. The construction procedure for subsequent courses is identical to that which was followed for initial course construction.

- **Set a full-height panel:**
  - Remove the two clamps securing the two initial course full-height panels to the half-height panel.
  - Joint filler is placed on the horizontal surface where the next panel is to be placed.
  - Place the full-height panel onto the underlying panel, centering the panel to ensure equal joints.
  - Panel batter is set with the wedges.
  - Joint filler is placed into the horizontal and angled joints.
  - Clamp the panel to the two adjacent initial course panels.
5. **Subsequent Backfilling and Wall Completion:**

Backfilling of subsequent courses can only commence once that course has been completed. The backfill procedure followed during initial course construction applies here. That is:

- Backfill is placed up to the next row of facing panel connectors and in the same direction as the wall construction. Backfill is not placed against the panels themselves, but is spread parallel to and windrowed toward the facing panels and the free end of the reinforcing strips or mesh.
- Backfill is compacted as required.
- Check the wall alignment and adjust accordingly.
- Reinforcing strips or mesh are placed on the compacted backfill and aligned with the panel connectors.
- Reinforcing strips or mesh are attached to the connectors as recommended in the Construction Manual.
- Backfill is placed to the top of the full-height panels from the previous course.
- Toe of the backfill is placed approximately 3 ft. (1 m) from the facing panel.
- Backfill is spread parallel to the facing panels and windrowed toward the facing panels and the free ends of the reinforcing strips.
- Backfill is compacted as required.
- Bracing from the initial course is removed after the second course has been placed.
- Backfilling in front of the wall or berm construction commences. Backfilling must be completed before wall reaches 50 percent of its height or 15 ft. (4.5 m), whichever is less.
- Hardwood wedges are removed. This should be done prior to there being no more than three courses of panels above the level of any wedge.

This procedure continues until the wall is complete.

- Remaining hardwood wedges and clamps are removed from wall.
- Coping is installed.
APPENDIX
CHAPTER 1
INTRODUCTION

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

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

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

This manual has been developed in support of FHWA educational programs on the design and
construction monitoring of MSEW retaining structures and RSS construction. Its principal
function is to serve as a reference source to the materials presented. The manual serves as
FHWA's primary technical guideline on the use of this technology on transportation facilities.

a. Scope
The manual addresses in a comprehensive manner the following areas:
  • Overview of MSE development and the cost, advantages, and disadvantages of using
    MSE structures.
  • Available MSE systems and applications to transportation facilities.
  • Basic soil-reinforcement interaction.
  • Design of routine and complex MSE walls.
  • Design of steepened RSS.
  • Design of steepened RSS over soft foundations.
  • Specifications and contracting approaches for both MSE walls and RSS construction.
  • Construction monitoring and inspection.
  • Design examples as case histories with detailed cost savings documented.
• A separate companion Manual addresses the long-term degradation of metallic and polymeric reinforcements. Sections of the Degradation manual address the background of full-scale, long-term evaluation programs and the procedures required to develop, implement, and evaluate them. These procedures have been developed to provide practical information on this topic for MSE users for non corrosion or polymer specialists, who are interested in developing long-term monitoring programs for these types of structures.

As an integral part of this Manual, several student exercises and workshop problems are included with solutions that demonstrate individual design aspects.

b. Source Documents
The majority of the material presented in this Manual was abstracted from FHWA RD89-043 "Reinforced Soil Structures, Volume I Design and Construction Guidelines", 1996 AASHTO Specifications, both Division I, Design and Division II, Construction, and direct input from the AASHTO Bridge T-15 Technical Committee as part of their effort to update Section 5.8 of the AASHTO Bridge Specifications which resulted in the 1997, 1998, 1999 and 2000 AASHTO Interims.

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

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

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

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

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

Geosynthetics is a generic term that encompasses flexible polymeric materials used in geotechnical engineering such as geotextiles, geomembranes, geonets, and grids (also known as geogrids).
Facing is a component of the reinforced soil system used to prevent the soil from raveling out between the rows of reinforcement. Common facings include precast concrete panels, dry cast modular blocks, metal sheets and plates, gabions, welded wire mesh, shotcrete, wood lagging and panels, and wrapped sheets of geosynthetics. The facing also plays a minor structural role in the stability of the structure. For RSS structures it usually consists of some type of erosion control material.

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

Reinforced backfill is the fill material in which the reinforcements are placed. Generic cross sections of a mechanically stabilized soil mass in its geotechnical environment is shown in figures 1 and 4.

![Mechanically Stabilized Earth Mass - Principal Elements](image)

Figure 1. Generic cross section of a MSE structure.

1.2 HISTORICAL DEVELOPMENT
Retaining structures are essential elements of every highway design. Retaining structures are used not only for bridge abutments and wing walls but also for slope stabilization and to minimize right-of-way for embankments. For many years, retaining structures were almost exclusively made of reinforced concrete and were designed as gravity or cantilever walls which are essentially rigid structures and cannot accommodate significant differential settlements unless founded on deep foundations. With increasing height of soil to be retained and poor subsoil conditions, the cost of reinforced concrete retaining walls increases rapidly.
Mechanically Stabilized Earth Walls (MSEW) and Reinforced Soil Slopes (RSS) are cost-effective soil-retaining structures that can tolerate much larger settlements than reinforced concrete walls. By placing tensile reinforcing elements (inclusions) in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/reinforcement system is essentially self supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the inclusions can also withstand bending from shear stresses, providing additional stability to the system.

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

The modern methods of soil reinforcement for retaining wall construction were pioneered by the French architect and engineer Henri Vidal in the early 1960s. His research led to the invention and development of Reinforced Earth®, a system in which steel strip reinforcement is used. The first wall to use this technology in the United States was built in 1972 on California State Highway 39, northeast of Los Angeles. In the last 25 years, more than 23,000 Reinforced Earth structures representing over 70 million m² (750 million ft²) of wall facing have been completed in 37 countries. More than 8,000 walls have been built in the United States since 1972. The highest wall constructed in the United States was on the order of 30 meters (98 feet).

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

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

For the first 20 years of use in the United States an articulating precast facing unit 2 to 2.25 m² (21 to 24 ft²) generally square in shape, was the facing unit of choice. More recently, larger precast units of up to 5 m² (54 ft²) have been used as have much smaller dry-cast units, generally in conjunction with geosynthetic reinforcements.
The use of geotextiles in MSE walls and RSS started after the beneficial effect of reinforcement with geotextiles was noticed in highway embankments over weak subgrades. The first geotextile reinforced wall was constructed in France in 1971, and the first structure of this type in the United States was constructed in 1974. Since about 1980, the use of geotextiles in reinforced soil has increased significantly.

Geogrids for soil reinforcement were developed around 1980. The first use of geogrid in earth reinforcement was in 1981. Extensive use of geogrid products in the United States started in about 1983, and they now comprise a growing portion of the market.

The first reported use of reinforced steepened slopes is believed to be the west embankment for the great wall of China. The introduction and economy of geosynthetic reinforcements has made the use of steepened slopes economically attractive. A survey of usage in the mid 1980s identified several hundred completed projects. At least an order of magnitude more RSS structures have been constructed since that study. The highest constructed RSS structure in the U.S. to date has been 43 m (141 ft).

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

**Current Usage**

It is believed that MSE walls have been constructed in every State in the United States. Major users include transportation agencies in Georgia, Florida, Texas, Pennsylvania, New York, and California, which rank among the largest road building States.

It is estimated that more than 700,000 m² (7,500,000 ft²) of MSE retaining walls with precast facing are constructed on average every year in the United States, which may represent more than half of all retaining wall usage for transportation applications.

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

Recently, modular block dry cast facing units have gained acceptance due to their lower cost and nationwide availability. These small concrete units are generally mated with grid reinforcement, and the wall system is referred to as modular block wall (MBW). It has been reported that more than 200,000 m² (2,000,000 ft²) of MBW walls have been constructed yearly in the United States when considering all types of transportation related applications. The current yearly usage for transportation-related applications is estimated at about 50 projects per year.

The use of RSS structures has expanded dramatically in the last decade, and it is estimated that several hundred RSS structures have been constructed in the United States. Currently, 70 to 100
RSS projects are being constructed yearly in connection with transportation related projects in the United States, with an estimated projected vertical face area of 130,000 m\(^2\)/year (1,400,000 ft\(^2\)/yr).

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

<table>
<thead>
<tr>
<th>System Name</th>
<th>Reinforcement Detail</th>
<th>Typical Face Panel Detail</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reinforced Earth</td>
<td>Galvanized Ribbed Steel Strips: 4 mm thick, 50 mm wide. Epoxy-coated strips also available.</td>
<td>Facing panels are cruciform shaped precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.</td>
</tr>
<tr>
<td>The Reinforced Earth Company</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2010 Corporate Ridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>McLean, VA 22102</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Retained Earth</td>
<td>Rectangular grid of W11 or W20 plain steel bars, 610 x 150 mm grid. Each mesh may have 4, 5 or 6 longitudinal bars. Epoxy-coated meshes also available.</td>
<td>Hexagonal and square precast concrete 1.5 x 1.5 m x 140 mm thick. Half size panels used at top and bottom.</td>
</tr>
<tr>
<td>Foster Geotechnical</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1600 Hotel Circle North</td>
<td></td>
<td></td>
</tr>
<tr>
<td>San Diego, CA 92108-2803</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mechanically Stabilized Embankment</td>
<td>Rectangular grid, nine 9.5 mm diameter plain steel bars on 610 x 150 mm grid. Two bar mats per panel (connected to the panel at four points).</td>
<td>Precast concrete; rectangular 3.81 m long, 610 mm high, 200 mm thick.</td>
</tr>
<tr>
<td>Dept. of Transportation,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Division of Engineering Services</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5900 Folsom Blvd.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P.O. Box 19128</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sacramento, CA 95819</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ARES</td>
<td>HDPE Geogrid</td>
<td>Precast concrete panel; rectangular 2.74 m wide, 1.52 m high, 140 mm thick.</td>
</tr>
<tr>
<td>Tensar Earth Technologies</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5883 Glenridge Drive, Suite 200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Atlanta, GA 30328</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Welded Wire Wall</td>
<td>Welded steel wire mesh, grid 50 x 150 mm of W4.5 x W3.5, W9.5 x W4, W9.5 x W4, and W12 x W5 in 2.43 m wide mats.</td>
<td>Welded steel wire mesh, wrap around with additional backing mat 6.35 mm wire screen at the soil face (with geotextile or shotcrete, if desired).</td>
</tr>
<tr>
<td>Hilfiker Retaining Walls</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P.O. Drawer L Eureka, CA 95501</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinforced Soil Embankment</td>
<td>15 cm x 61 cm welded wire mesh: W9.5 to W20 - 8.8 to 12.8 mm diameter.</td>
<td>Precast concrete unit 3.8 m long, 610 mm high.</td>
</tr>
<tr>
<td>Hilfiker Retaining Walls,</td>
<td></td>
<td></td>
</tr>
<tr>
<td>P.O. Drawer L Eureka, CA 95501</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ISOGRID</td>
<td>Rectangular grid of W11 x W11 4 bars per grid</td>
<td>Diamond shaped precast concrete units, 1.5 by 2.5 m, 140 mm thick.</td>
</tr>
<tr>
<td>Neel Co.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6520 Deepford Street</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Springfield, VA 22150</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MESA</td>
<td>HDPE Geogrid</td>
<td>MESA HP (high performance), DOT OR Standard units (203 mm high by 457 mm long face, 275 mm nominal depth). (dry cast concrete)</td>
</tr>
<tr>
<td>Tensar Earth Technologies, Inc.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5883 Glenridge Drive, Suite 200</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Atlanta, GA 30328</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PYRAMID</td>
<td>Galvanized WWM, size varies with design requirements or Grid of PVC coated, Polyester yarn (Matrex Geogrid)</td>
<td>Pyramid unit (200 mm high by 400 mm long face, 250 mm nominal depth) (dry cast concrete)</td>
</tr>
<tr>
<td>The Reinforced Earths Company</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2010 Corporate Ridge</td>
<td></td>
<td></td>
</tr>
<tr>
<td>McLean, VA 22102</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maccaverri Terramesh System</td>
<td>Continuous sheets of galvanized double twisted woven wire mesh with PVC coating.</td>
<td>Rock filled gabion baskets laced to reinforcement.</td>
</tr>
<tr>
<td>Maccaverri Gabions, Inc.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>43A Governor Lane Blvd.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Williamsport, MD 21795</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Strengthened Earth</td>
<td>Rectangular grid, W7, W9.5 and W14, transverse bars at 230 and 450 mm.</td>
<td>Precast concrete units, rectangular or wing shaped 1.82 m x 2.13 m x 140 mm.</td>
</tr>
<tr>
<td>Gifford-Hill &amp; Co.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2515 McKinney Ave.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dallas, Texas 75201</td>
<td></td>
<td></td>
</tr>
<tr>
<td>MSE Plus</td>
<td>Rectangular grid with W11 to W24 longitudinal bars and W11 transverse. Mesh may have 4 to 6 longitudinal bars spaced at 200 mm.</td>
<td>Rectangular precast concrete panels 1.5 m high, 1.82 m wide with a thickness of 152 or 178 mm</td>
</tr>
<tr>
<td>SSL</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4740 Scotts Valley Drive</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Scotts Valley, CA 95066</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Key System - Inextensible Keystone</td>
<td>Galvanized welded wire ladder mat of W7.5 to W17 bars with crossbars at 150 mm to 600 mm</td>
<td>KeySystem concrete facing unit is 203 mm high x 457 mm wide x 305 mm deep (dry cast concrete)</td>
</tr>
<tr>
<td>Retaining Wall Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4444 W. 78th Street</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Minneapolis, MN 55435</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Ref: Federal Highway Administration
Publication No. FHWA-NHI-00-043
"Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines"*
Table 1. Summary of reinforcement and face panel details for selected MSE wall systems (cont).

<table>
<thead>
<tr>
<th>System Name</th>
<th>Reinforcement Detail</th>
<th>Typical Face Panel Detail</th>
</tr>
</thead>
<tbody>
<tr>
<td>KeySystem - Extensible Keystone Retaining Wall Systems 4444 W. 78th Street Minneapolis, MN 55435</td>
<td>Stratagrid high tenacity knit polyester geogrid soil reinforcement by Strata Systems, Inc. PVC coated</td>
<td>Keystone Standard and Compac concrete facing units are 203 mm high x 457 mm wide x 457 mm or 305 mm deep (dry cast concrete)</td>
</tr>
<tr>
<td>Versa-Lok Retaining Wall Systems 634 Highwy 36 Blvd. Oakdale, MN 55128</td>
<td>PVC coated PET or HDPE geogrids</td>
<td>Versa-Lok concrete unit 152 mm high x 406 mm long x 305 mm deep (dry cast concrete)</td>
</tr>
<tr>
<td>Anchor Wall Systems 5959 Baker Road Minnetonka, MN 55345</td>
<td>PVC coated PET geogrid</td>
<td>Anchor Vertica concrete unit 200 mm high x 450 mm long x 300 mm deep and Anchor Vertica Pro which is 500 mm deep (dry cast concrete)</td>
</tr>
</tbody>
</table>

Additional facing types are possible with most systems.

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

| Amoco Fabrics and Fibers Co. 260 The Bluff Austelle, GA 30168 | BBA Nonwovens - Reemay, Inc. 70 Old Hickory Blvd. Old Hickory, TN 37138 | Carthage Mills 4243 Hunt Road Cincinnati, OH 45242 |
| Colbond Geosynthetics (Akzo) 95 Sand Hill Road Enka, NC 28728 | Contech Construction Products 1001 Grove Street Middletown, OH 45044 | Huesker, Inc. 11107 A S. Commerce Blvd. Charlotte, NC 28241 |
| LINQ Industrial Fabrics, Inc. 2550 West 5th North Street Summerville, SC 29483 | Luckenhaus North America 841 Main Street Spartanburg, SC 29302 | TC Mirafi 365 S. Holland Drive Pendegrass, GA 30567 |
| Nicolon Corporation 3500 Parkway Lane, Suite 500 Norcross, GA 30092 | Strata Systems, Inc. 425 Trible Gap Road Cummings, GA 30130 | Synthetic Industries Construction Products Division 4019 Industry Drive Chattanooga, TN 37416 |
| Tenax Corporation 4800 East Monument Street Baltimore, MD 21205 | Tensar Earth Technologies 5883 Glenridge Drive, Suite 200 Atlanta, GA 30328 | TNS Advanced Technologies 681 Deyoung Road Greer, SC 29651 |

¹ List is from the Geosynthetic Materials Association membership list.

Ref: Federal Highway Administration Publication No. FHWA-NHI-00-043 "Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines"
CHAPTER 2
SYSTEMS AND PROJECT EVALUATION

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

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

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

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

Some additional successful uses of MSE walls include:

- Temporary structures, which have been especially cost-effective for temporary detours necessary for highway reconstruction projects.
- Reinforced soil dikes, which have been used for containment structures for water and waste impoundments around oil and liquid natural gas storage tanks. (The use of reinforced soil containment dikes is economical and can also result in savings of land because a vertical face can be used, which reduces construction time).
- Dams and seawalls, including increasing the height of existing dams.
- Bulk materials storage using sloped walls.

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

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

There are two primary purposes for using reinforcement in engineered slopes.
- To increase the stability of the slope, particularly if a steeper than safe unreinforced slope is desirable or after a failure has occurred as shown in figure 4a.
- To provide improved compaction at the edges of a slope, thus decreasing the tendency for surface sloughing as shown in figure 4b.

The principal purpose for using reinforcement is to construct an RSS embankment at an angle steeper than could otherwise be safely constructed with the same soil. The increase in stability allows for construction of steepened slopes on firm foundations for new highways and as replacements for flatter unreinforced slopes and retaining walls. Roadways can also be widened over existing flatter slopes without encroaching on existing right-of-ways. In the case of repairing a slope failure, the new slope will be safer, and reusing the slide debris rather than importing higher quality backfill may result in substantial cost savings. These applications are illustrated in figure 5.

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

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

Compaction aids placed as intermediate layers between reinforcement in steepened slopes may also be used to provide improved face stability and to reduce layers of more expensive primary reinforcement as shown in figure 4a.
Figure 2. MSE walls, urban applications.
Figure 3. MSE wall applications, abutments, and marine.
Figure 4. Slope reinforcement using geosynthetics to provide slope stability.
Other applications of reinforced slopes have included:
- Upstream/downstream face improvements to increased height of dams.
- Permanent levees.
- Temporary flood control structures.
- Decreased bridge spans.
- Temporary road widening for detours.
- Prevention of surface sloughing during periods of saturation.
- Embankment construction with wet, fine-grained soils.

2.2 ADVANTAGES AND DISADVANTAGES

a. Advantages of Mechanically Stabilized Earth (MSE) Walls
MSE walls have many advantages compared with conventional reinforced concrete and concrete gravity retaining walls. MSE walls:
- Use simple and rapid construction procedures and do not require large construction equipment.
- Do not require experienced craftsmen with special skills for construction.
- Require less site preparation than other alternatives.
- Need less space in front of the structure for construction operations.
- Reduce right-of-way acquisition.
- Do not need rigid, unyielding foundation support because MSE structures are tolerant to deformations.
- Are cost effective.
- Are technically feasible to heights in excess of 25 m (80 ft).

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

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

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

**b. Advantages of Reinforced Soil Slopes (RSS)**

The economic advantages of constructing a safe, steeper RSS than would normally be possible are the resulting material and rights-of-way savings. It also may be possible to decrease the quality of materials required for construction. For example, in repair of landslides it is possible to reuse the slide debris rather than to import higher quality backfill.

Right-of-way savings can be a substantial benefit, especially for road widening projects in urban areas where acquiring new right-of-way is always expensive and, in some cases, unobtainable. RSS also provide an economical alternative to retaining walls. In some cases, reinforced slopes can be constructed at about one-half the cost of MSEW structures.

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

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

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

- Require a relatively large space behind the wall or outward face to obtain enough wall width for internal and external stability.
- MSEW require select granular fill. (At sites where there is a lack of granular soils, the cost of importing suitable fill material may render the system uneconomical). Requirements for RSS are typically less restrictive.
- Suitable design criteria are required to address corrosion of steel reinforcing elements, deterioration of certain types of exposed facing elements such as geosynthetics by ultra violet rays, and potential degradation of polymer reinforcement in the ground.
- Since design and construction practice of all reinforced systems are still evolving, specifications and contracting practices have not been fully standardized, especially for RSS.
- The design of soil-reinforced systems often requires a shared design responsibility between material suppliers and owners and greater input from agencies geotechnical specialists in a domain often dominated by structural engineers.

2.3 RELATIVE COSTS
Site specific costs of a soil-reinforced structure are a function of many factors, including cut-fill requirements, wall/slope size and type, in-situ soil type, available backfill materials, facing finish, temporary or permanent application. It has been found that MSE walls with precast concrete facings are usually less expensive than reinforced concrete retaining walls for heights **greater** than about 3 m (10 ft) and average foundation conditions. Modular block walls (MBW) are competitive with concrete walls at heights of **less** than 4.5 m (15 ft).

In general, the use of MSE walls results in savings on the order of 25 to 50 percent and possibly more in comparison with a conventional reinforced concrete retaining structure, especially when the latter is supported on a deep foundation system (poor foundation condition). A substantial savings is obtained by elimination of the deep foundations, which is usually possible because reinforced soil structures can accommodate relatively large total and differential settlements. Other cost saving features include ease of construction and speed of construction. A comparison of wall material and erection costs for several reinforced soil retaining walls and other retaining wall systems, based on a survey of state and federal transportation agencies, is shown in figure 6. Typical total costs for MSE walls range from $200 to $400 per m² ($19 to $37 per ft²) of face, generally as function of height, size of project and cost of select fill.
The actual cost of a specific MSEW structure will depend on the cost of each of its principal components. For segmental precast concrete faced structures, typical relative costs are:

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

![Figure 6. Cost comparison for retaining walls.](after 23)
The additional cost for panel architectural finish treatment ranges from $5 to $15 per m² ($0.50 to $1.50 per ft²) depending on the complexity of the finish. Traffic barrier costs average $550 per linear meter ($170 per linear foot). In addition, consideration must be given to the cost of excavation which may be somewhat greater than for other systems. MBW faced walls at heights less than 4.5 m (15 ft) are typically less expensive by 10 percent or more.

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

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

The actual bid cost of a specific RSS structure depends on the cost of each of its principal components. Based on limited data, typical relative costs are:
- Reinforcement - 45 to 65 percent of total cost
- Backfill - 30 to 45 percent of total cost
- Face treatment - 5 to 10 percent of total cost

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

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

Figure 7 provides a rapid, first-order assessment of cost items for comparing a flatter unreinforced slope with a steeper reinforced slope.
2.4 DESCRIPTION OF MSE/RSS SYSTEMS

a. Systems Differentiation

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

Trademarks, such as Reinforced Earth®, Retained Earth®, Genesis® etc., describe systems with some present or past proprietary features or unique components marketed by nationwide commercial suppliers. Other trademark names appear yearly to differentiate systems marketed by

Ref: Federal Highway Administration
Publication No. FHWA-NHI-00-043
*Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines*
competing commercial entities that may include proprietary or novel components or for special applications.

A system for either MSEW or RSS structures is defined as a complete supplied package that includes design, specifications and all prefabricated materials of construction necessary for the complete construction of a soil reinforced structure. Often technical assistance during the planning and construction phase is also included. Components marketed by commercial entities for integration by the owner in a coherent system are not classified as systems.

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

Reinforcement Geometry
Three types of reinforcement geometry can be considered:

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

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

- **Metallic reinforcements.** Typically of mild steel. The steel is usually galvanized or may be epoxy coated.
- **Nonmetallic reinforcements.** Generally polymeric materials consisting of polypropylene, polyethylene, or polyester.

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

Reinforcement Extensibility
There are two classes of extensibility:

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

c. Facing Systems
The types of facing elements used in the different MSE systems control their aesthetics because they are the only visible parts of the completed structure. A wide range of finishes and colors can be provided in the facing. In addition, the facing provides protection against backfill sloughing and erosion, and provides in certain cases drainage paths. The type of facing influences settlement tolerances. Major facing types are:

- **Segmental precast concrete panels** summarized in table 1 and illustrated in figure 8. The precast concrete panels have a minimum thickness of 140 mm (5-½ inches) and are of a cruciform, square, rectangular, diamond, or hexagonal geometry. Temperature and tensile reinforcement are required but will vary with the size of the panel. Vertically adjacent units are usually connected with shear pins.

- **Dry cast modular block wall (MBW) units.** These are relatively small, squat concrete units that have been specially designed and manufactured for retaining wall applications. The mass of these units commonly ranges from 15 to 50 kg (30 to 110 lbs), with units of 35 to 50 kg (75 to 110 lbs) routinely used for highway projects. Unit heights typically range from 100 to 200 mm (4 to 8 inches) for the various manufacturers. Exposed face length usually varies from 200 to 450 mm (8 to 18 inches). Nominal width (dimension perpendicular to the wall face) of units typically ranges between 200 and 600 mm (8 and 24 inches). Units may be manufactured solid or with cores. Full height cores are filled with aggregate during erection. Units are normally dry-stacked (i.e. without mortar) and in a running bond configuration. Vertically adjacent units may be connected with shear pins, lips, or keys. They are referred to by trademarked names such as Keystone®, Versa-Lok®, Allan® etc. They are illustrated in figure 9.

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

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

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

- **Geosynthetic Facing.** Various types of geotextile reinforcement are looped around at the facing to form the exposed face of the retaining wall. These faces are susceptible to ultraviolet light degradation, vandalism (e.g. target practice) and damage due to fire. Alternately, a geosynthetic grid used for soil reinforcement can be looped around to form the face of the completed retaining structure in a similar manner to welded wire mesh and
fabric facing. Vegetation can grow through the grid structure and can provide both ultraviolet light protection for the geogrid and a pleasing appearance.

- **Postconstruction Facing.** For wrapped faced walls, the facing – whether geotextile, geogrid, or wire mesh – can be attached after construction of the wall by shotcreting, guniting, cast-in-place concrete or attaching prefabricated facing panels made of concrete, wood, or other materials. This multi-staging facing approach adds cost but is advantageous where significant settlement is anticipated.
Figure 8. MSE wall surface treatments.
Figure 9. Examples of commercially available MBW units (from NCMA Design Manual for Segmental Retaining Walls).
Precast elements can be cast in several shapes and provided with facing textures to match environmental requirements and blend aesthetically into the environment. Retaining structures using precast concrete elements as the facings can have surface finishes similar to any reinforced concrete structure.

Retaining structures with metal facings have the disadvantage of shorter life because of corrosion, unless provision is made to compensate for it.

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

Dry cast segmental block MBW facings may raise some concerns as to durability in aggressive freeze-thaw environments when produced with water absorption capacity significantly higher than that of wet-cast concrete. Historical data provide little insight as their usage history is less than two decades. Further, because the cement is not completely hydrated during the dry cast process, (as is often evidenced by efflorescence on the surface of units), a highly alkaline regime may establish itself at or near the face area, and may limit the use of some geosynthetic products as reinforcements. Freeze-thaw durability is enhanced for products produced at higher compressive strengths and low water absorption ratios. The current specifications in Chapter 8 have been developed to address this issue.

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

d. Reinforcement Types
Most, although not all systems using precast concrete panels use steel reinforcements which are typically galvanized but may be epoxy coated. Two types of steel reinforcements are in current use:

59. Steel strips. The currently commercially available strips are ribbed top and bottom, 50 mm (2 inches) wide and 4 mm (5/32-inch) thick. Smooth strips 60 to 120 mm (2 1/2 to 4¾-inch) wide, 3 to 4 mm ( to 5/32-inch) thick have been used.
Steel grids. Welded wire grid using 2 to 6 W7.5 to W24 longitudinal wire spaced at either 150 or 200 mm (6 or 8 inches). The transverse wire may vary from W11 to W20 and are spaced based on design requirements from 230 to 600 mm (9 to 24 inches). Welded steel wire mesh spaced at 50 by 50 mm (2 by 2-inch) of thinner wire has been used in conjunction with a welded wire facing. Some MBW systems use steel grids with 2 longitudinal wires.

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

1. High Density Polyethylene (HDPE) geogrid. These are of uniaxial manufacture and are available in up to 6 styles differing in strength.
2. PVC coated polyester (PET) geogrid. Available from a number of manufacturers. They are characterized by bundled high tenacity PET fibers in the longitudinal load carrying direction. For longevity the PET is supplied as a high molecular weight fiber and is further characterized by a low carboxyl end group number.
3. Geotextiles. High strength geotextiles can be used principally in connection with reinforced soil slope (RSS) construction. Both polyester (PET) and polypropylene (PP) geotextiles have been used.

**e. Reinforced Backfill Materials**

**MSEW Structures**

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

From a reinforcement capacity point of view, lower quality backfills could be used for MSEW structures; however, a high quality granular backfill has the advantages of being free draining, providing better durability for metallic reinforcement, and requiring less reinforcement. There are also significant handling, placement and compaction advantages in using granular soils. These include an increased rate of wall erection and improved maintenance of wall alignment tolerances.

**RSS Structures**

Reinforced Soil Slopes are normally not constructed with rigid facing elements. Slopes constructed with a flexible face can thus readily tolerate minor distortions that could result from settlement, freezing and thawing, or wet-drying of the backfill. As a result, any soil meeting the requirements for embankment construction could be used in a reinforced slope system. However, a higher quality material offers less durability concerns for the reinforcement, and is easier to handle, place and compact, which speeds up construction.
f. Miscellaneous Materials of Construction
Walls using precast concrete panels require bearing pads in their horizontal joints that provide some compressibility and movement between panels and preclude concrete to concrete contact. These materials are either neoprene, SBR rubber or HDPE.

All joints are covered with a polypropylene (PP) geotextile strip to prevent the migration of fines from the backfill. The compressibility of the horizontal joint material should be a function of the wall height. Walls with heights greater than 15 m (50 ft) may require thicker or more compressible joints to accommodate the larger vertical loads due to the weight of panels in the lower third of the structure.

2.5 SITE EVALUATION

a. Site Exploration
The feasibility of using an MSEW, RSS or any other type of earth retention system depends on the existing topography, subsurface conditions, and soil/rock properties. It is necessary to perform a comprehensive subsurface exploration program to evaluate site stability, settlement potential, need for drainage, etc., before repairing a slope or designing a new retaining wall or bridge abutment.

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

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

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

b. Field Reconnaissance
Preliminary subsurface investigation or reconnaissance should consist of collecting any existing data relating to subsurface conditions and making a field visit to obtain data on:
- Limits and intervals for topographic cross sections.
- Access conditions for work forces and equipment.
- Surface drainage patterns, seepage, and vegetation characteristics.
- Surface geologic features, including rock outcrops and landforms, and existing cuts or excavations that may provide information on subsurface conditions.
• The extent, nature, and locations of existing or proposed below-grade utilities and substructures that may have an impact on the exploration or subsequent construction.
• Available right-of-way.
• Areas of potential instability such as deep deposits of weak cohesive and organic soils, slide debris, high groundwater table, bedrock outcrops, etc.

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

c. Subsurface Exploration
The subsurface exploration program generally consists of soil soundings, borings, and test pits. The type and extent of the exploration should be decided after review of the preliminary data obtained from the field reconnaissance, and in consultation with a geotechnical engineer or an engineering geologist. The exploration must be sufficient to evaluate the geologic and subsurface profile in the area of construction. For guidance on the extent and type of required investigation, the 1988 AASHTO "Manual on Foundation Investigations", should be reviewed.

The following minimum guidelines are recommended for the subsurface exploration for potential MSE applications:
• Soil borings should be performed at intervals of:
  - 30 m (100 ft) along the alignment of the soil-reinforced structure
  - 45 m (150 ft) along the back of the reinforced soil structure
  The width of the MSE wall or slope structure may be assumed as 0.8 times the anticipated height.
• The boring depth should be controlled by the general subsurface conditions. Where bedrock is encountered within a reasonable depth, rock cores should be obtained for a length of about 3 m (10 ft). This coring will be useful to distinguish between solid rock and boulders. Deeper coring may be necessary to better characterize rock slopes behind new retaining structures. In areas of soil profile, the borings should extend at least to a depth equal to twice the height of the wall/slope. If subsoil conditions within this depth are found to be weak and unsuitable for the anticipated pressures from the structure height, then the borings must be extended until reasonably strong soils are encountered.
• In each boring, soil samples should be obtained at 1.5-m depth intervals and at changes in strata for visual identification, classification, and laboratory testing. Methods of sampling may follow AASHTO T 206 or AASHTO T 207 (Standard Penetration Test and Thin-Walled Shelby Tube Sampling, respectively), depending on the type of soil. In granular soils, the Standard Penetration Test can be used to obtain disturbed samples. In cohesive soils, undisturbed samples should be obtained by thin-walled sampling procedures. In each
boring, careful observation should be made for the prevailing water table, which should be observed not only at the time of sampling but also at later times to obtain a good record of prevailing water table conditions. If necessary, piezometers should be installed in a few borings to observe long-term water levels.

- Both the Standard Penetration Test and the Cone Penetration Test, ASTM D-3441, provide data on the strengths and density of soils. In some situations, it may be desirable to perform in situ tests using a dilatometer, pressuremeter, or similar means to determine soil modulus values.
- Adequate bulk samples of available soils should be obtained and evaluated as indicted in the following testing section to determine the suitability of the soil for use as backfill in the MSE structures. Such materials should be obtained from all areas from which preliminary reconnaissance indicates that borrow materials will be used.
- Test-pit explorations should be performed in areas showing instability or to explore further availability of the borrow materials for backfill. The locations and number of test pits should be decided for each specific site, based on the preliminary reconnaissance data.

The development and implementation of an adequate subsurface investigation program is a key element for ensuring successful project implementation. Causes for distress experienced in projects are often traced to inadequate subsurface exploration programs, that did not disclose local or significant areas of soft soils, causing significant local differential settlement and distress to the facing panels. In a few documented extreme cases, such foundation weakness caused complete foundation failures leading to catastrophic collapses. Where the select backfill is to be obtained from on-site sources, the extent and quality must be fully explored to minimize contractor claims for changed conditions.

d. Laboratory Testing
Soil samples should be visually examined and appropriate tests performed for classification according to the Unified Soil Classification System (ASTM D 2488-69). These tests permit the engineer to decide what further field or laboratory tests will best describe the engineering behavior of the soil at a given project site. Index testing includes determination of moisture content, Atterberg limits, compressive strength, and gradation. The dry unit weight of representative undisturbed samples should also be determined.

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

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

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

- pH.
- Electrical resistivity.
- Salt content including sulfate, sulfides, and chlorides.

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

2.6 PROJECT EVALUATION

a. Structure Selection Factors
The major factors that influence the selection of an MSE/RSS alternative for any project include:

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

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

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

RSS embankments have been constructed with a variety of geosynthetic reinforcements and treatments of the outward face. These factors again may create an initial difficulty in adequate
technical evaluation. A number of geosynthetic reinforcement suppliers provide design services as well as technical assistance during construction.

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

**b. Geologic and Topographic Conditions**

MSE structures are particularly well-suited where a "fill type" wall must be constructed or where side-hill fills are indicated. Under these latter conditions, the volume of excavation may be small, and the general economy of this type of construction is not jeopardized.

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

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

Where these conditions are not satisfied, ground improvement techniques must be considered to increase the bearing capacity at the foundation level. These techniques include but are not limited to:

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

Where marginal to adequate foundation strength is available, preliminary settlement analyses should be made to determine the potential for differential settlement, both longitudinally along a proposed structure as well as transverse to the face. This second-order feasibility evaluation is useful in determining the appropriate type of facing systems for MSE walls and in planning appropriate construction staging to accommodate the settlement.

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

The performance of reinforced soil slopes generally is not affected by differential longitudinal settlements.
c. Environmental Conditions
The primary environmental condition affecting reinforcement type selection and potential performance of MSE structures is the aggressiveness of the in situ ground regime that can cause deterioration to the reinforcement. Post construction changes must be considered where de-icing salts or fertilizers are subsequently used.

For steel reinforcements, in situ regimes containing chloride and sulfate salts generally in excess of 200 PPM accelerate the corrosive process as do acidic regimes characterized by a pH of less than 5. Alkaline regimes characterized by pH > 10 will cause accelerated loss of galvanization. Under these conditions, bare steel reinforcements could be considered.

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

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

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

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

d. Size and nature of structure
Theoretically there is no upper limit to the height of MSEW that can be constructed. Structures in excess of 25 m (80 ft) have been successfully constructed with steel reinforcements although such heights for transportation-related structures are rare. RSS embankments have been constructed to greater heights.

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

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

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

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

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

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

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

f. Questionable Applications
The current AASHTO Interim Specifications for Highway Bridges, indicates that MSE walls should not be used under the following conditions:

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

2.7 ESTABLISHMENT OF PROJECT CRITERIA

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

The process consists of the following successive steps:
• Consider all possible alternatives.
• Choose a system (MSEW or RSS).
• Consider facing options.
• Develop performance criteria (Loads, design heights, embedment, settlement tolerances, foundation capacity, effect on adjoining structures, etc.).
• Consider effect of site on corrosion/degradation of reinforcements.

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

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

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

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

For permanent applications, considerations should be given to MSE walls with precast concrete panels. They are constructed with a vertical face and cannot accommodate small, uniform front batters. Currently, the size of panels commercially produced varies from 1.8 to 4.5 m² (20 to 50 ft²). Full height panels may be considered for walls up to 4 to 5 m (13 to 16 ft) in height on foundations that are not expected to settle. The precast concrete panels can be manufactured with a variety of surface textures and geometrics, as shown in figure 8.
MBW facings are available in a variety of shapes and textures as shown in figure 9. They range in facial area from 0.05 to 0.1 m\(^2\) (0.5 to 1 ft\(^2\)). An integral feature of this type of facing is a front batter ranging from nominal to 15 degrees. Project geometric constraints, i.e., the bottom of wall and top of wall horizontal limits, may limit the amount of permissible batter and, thus, the types of MBW units that may be used. Note that the toe of these walls step back as the foundation elevation steps up, due to the stacking arrangement and automatic batter.

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

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

For RSS structures, the choice of slope facing may be controlled by climatic and regional factors. For structures of less than 10 m (33 ft) height with slopes of 1:1 or flatter, a vegetative "green slope" can be usually constructed using an erosion control mat or mesh and local grasses. Where vegetation cannot be successfully established and/or significant run-off may occur, armored slopes using natural or manufactured materials may be the only choice to reduce future maintenance. For additional guidance see chapter 6, section 6.5.

**c. Performance Criteria**

Performance criteria for MSE structures with respect to design requirements are governed by design practice or codes such as contained in Article 5.8 of 1996 AASHTO Specifications for Highway Bridges. These requirements consider the required margins of safety with respect to failure modes. They are equal for all types of MSEW structures. No specific AASHTO guidance is presently available for RSS structures.

With respect to lateral wall displacements, no method is presently available to definitely predict lateral displacements, most of which occur during construction. The horizontal movements depend on compaction effects, reinforcement extensibility, reinforcement length, reinforcement-to-panel connection details, and details of the facing system. A rough estimate of probable lateral displacements of simple structures that may occur during construction can be made based on the reinforcement length to wall-height ratio and reinforcement extensibility as shown in figure 10.

This figure indicates that increasing the length-to-height ratio of reinforcements from its theoretical lower limit of 0.5H to 0.7H, decreases the deformation by 50 percent. It further suggests that the anticipated construction deformation of MSE structures constructed with polymeric reinforcements (extensible) is approximately three times greater than if constructed with metallic reinforcements (inextensible).
Performance criteria are both site and structure-dependent. Structure-dependent criteria consist of safety factors or a consistent set of load and resistance factors as well as tolerable movement criteria of the specific MSE structure selected.

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

- **External Stability**
  - Sliding: F.S. ≥ 1.5 (MSEW); 1.3 (RSS)
  - Eccentricity e, at Base: ≤ L/6 in soil L/4 in rock
  - Bearing Capacity: F.S. ≥ 2.5
  - Deep Seated Stability: F.S. ≥ 1.3
  - Compound Stability: F.S. ≥ 1.3
  - Seismic Stability: F.S. ≥ 75% of static F.S. (All failure modes)

- **Internal Stability**
  - Pullout Resistance: F.S. ≥ 1.5 (MSEW and RSS)
  - Internal Stability for RSS: F.S. ≥ 1.3
  - Allowable Tensile Strength
    - for steel strip reinforcement: 0.55 F_y
    - for steel grid reinforcement: 0.48 F_y (connected to concrete panels or blocks)
    - for geosynthetic reinforcements: T_a - See design life, below
\[ \delta_{\text{max}} = \delta_R \cdot \frac{H}{250} \text{ (INEXTENSIBLE)} \]
\[ \delta_{\text{max}} = \delta_R \cdot \frac{H}{75} \text{ (EXTENSIBLE)} \]

WHERE: \( \delta_{\text{max}} \) = MAXIMUM DISPLACEMENT IN UNITS OF H
\( H = \) HEIGHT OF WALL IN M.
\( \delta_R = \) EMPIRICALLY DERIVED RELATIVE DISPLACEMENT COEFFICIENT.

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

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

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

Figure 10. Empirical curve for estimating probable anticipated lateral displacement during construction of MSE walls (FHWA RD-89-043).
A number of site specific project criteria need to be established at the inception of design:

- **Design limits and wall height.** The length and height required to meet project geometric requirements must be established to determine the type of structure and external loading configurations.

- **Alignment limits.** The horizontal (perpendicular to wall face) limits of bottom and top of wall alignment must be established as alignments vary with batter of wall system. The alignment constraints may limit the type and maximum batter, particularly with MBW units, of wall facing.

- **Length of reinforcement.** A minimum reinforcement length of 0.7H is recommended for MSE walls. Longer lengths are required for structures subject to surcharge loads. Shorter lengths can be used in special situations.

- **External loads.** The external loads may be soil surcharges required by the geometry, adjoining footing loads, line loads as from traffic, and/or traffic impact loads. Traffic line loads and impact loads are applicable where the traffic lane is located horizontally from the face of the wall within a distance less than one half the wall height. The magnitude of the minimum traffic loads outlined in Articles 3.20.3 and 5.8 of current AASHTO, is a uniform load equivalent to 0.6 m (2 ft) of soil over the traffic lanes.

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

<table>
<thead>
<tr>
<th>Slope in Front of Wall</th>
<th>Minimum to Top of Leveling Pad</th>
</tr>
</thead>
<tbody>
<tr>
<td>horizontal (walls)</td>
<td>H/20</td>
</tr>
<tr>
<td>horizontal (abutments)</td>
<td>H/10</td>
</tr>
<tr>
<td>3H:1V</td>
<td>H/10</td>
</tr>
<tr>
<td>2H:1V</td>
<td>H/7</td>
</tr>
<tr>
<td>3H:2V</td>
<td>H/5</td>
</tr>
</tbody>
</table>

Larger values may be required, depending on depth of frost penetration, shrinkage and swelling of foundation soils, seismic activity, and scour. Minimum in any case is 0.5 m, except for structures founded on rock at the surface, where no embedment may be used. Alternately, frost-susceptible soils could be overexcavated and replaced with non frost susceptible backfill, hence reducing the overall wall height.

A minimum horizontal bench 1.2 m (4 ft) wide as measured from the face shall be provided in front of walls founded on slopes.

*Ref:* Federal Highway Administration  
Publication No. FHWA-NHI-00-043  
“Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines”
For walls constructed along rivers and streams where the depth of scour has been reliably determined, a minimum embedment of 0.6 m (2 ft) below this depth is recommended. Embedment is not required for RSS unless dictated by stability requirements.

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

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

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

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

Square panels generally adapt to larger longitudinal differential settlements better than long rectangular panels of the same surface area. Guidance on minimum joint width and limiting differential settlements that can be tolerated is presented in table 3, for panels with a surface area typically less than 4.5 m² (50 ft²).

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

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

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

<table>
<thead>
<tr>
<th>Joint Width Limiting</th>
<th>Differential Settlement</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 mm</td>
<td>1/100</td>
</tr>
<tr>
<td>13 mm</td>
<td>1/200</td>
</tr>
<tr>
<td>6 mm</td>
<td>1/300</td>
</tr>
</tbody>
</table>

**d. Design Life**

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

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

The quality of in-service performance is an important consideration in the design of permanent retaining walls. Permanent walls shall be designed to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life. For RSS structures, similar minimum design life ranges should be adopted.

**2.8 CONSTRUCTION SEQUENCE**

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

**a. Construction of MSEW systems with precast facings**

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

- **Preparation of subgrade.** This step involves removal of unsuitable materials from the area to be occupied by the retaining structure. All organic matter, vegetation, slide debris and other unstable materials should be stripped off and the subgrade compacted.
In unstable foundation areas, ground improvement methods, such as dynamic compaction, stone columns, wick drains, or other foundation stabilization/improvement methods would be constructed prior to wall erection.

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

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

- **Erection of the first row of facing panels on the prepared leveling pad.** Facings may consist of either precast concrete panels, metal facing panels, or dry cast modular blocks.

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

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

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

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

- **Placement of the backfill over the reinforcing elements to the level of the next reinforcement layer and compaction of the backfill.** The previously outlined steps are repeated for each successive layer.
• **Construction of traffic barriers and copings.** This final construction sequence is undertaken after the final panels have been placed, and the backfill has been completed to its final grade.

A complete sequence is illustrated in figures 11 through 13.

b. **Construction of MSE systems with Flexible Facings**

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

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

• **Placement of first reinforcing layer.** Reinforcement with anisotropic strength properties (i.e., many geosynthetics) should be placed with the principal strength direction perpendicular to face of structure. It is often convenient to unroll the reinforcement with the roll or machine direction parallel to the face. If this is done, then the cross machine tensile strength must be greater than the design tension requirements.

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

  Overlap adjacent sheets a minimum of 150 mm (6 inches) along the edges perpendicular to the face. Alternatively, with geogrid or wire mesh reinforcement, the edges may be butted and clipped or tied together.
Figure 11. Erection of Precast Panels.
Figure 12. Fill spreading and reinforcement connection.
 Face Construction. Place the geosynthetic layers using face forms as shown in figure 14. For temporary support of forms at the face, form holders should be placed at the base of each layer at 1.20 m (4 ft) horizontal intervals. Details of temporary form work are shown in figure 15. These supports are essential for achieving good compaction. When using geogrids or wire mesh, it may be necessary to use a geotextile to retain the backfill material at the wall face.

When compacting backfill within 1 m (3 ft) of the wall face, a hand-operated vibratory compactor is recommended.
The return-type method or successive layer tie method as shown in figure 15 can be used for facing support. In the return method, the reinforcement is folded at the face over the backfill material, with a minimum return length of 1.25 m (4 ft) to ensure adequate pullout resistance. Consistency in face construction and compaction is essential to produce a wrapped facing with satisfactory appearance.

Apply facing treatment (shotcrete, precast facing panels, etc.). Figure 16 shows some alternative facing systems for flexible faced walls and slopes.

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

- Site preparation.
- Construct subsurface drainage (if indicated).
- Place reinforcement layer.
- Place and compact backfill on reinforcement.
- Construct face. Details of the available methods are outlined in chapter 6, construction.
- Place additional reinforcement and backfill.
- Construct surface drainage features.

Key stages of construction are illustrated in figure 17, and the complete sequence is fully outlined in Chapter 6.

2.9 PROPRIETARY ASPECTS

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

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

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

b. Special Applications
A number of patents may be in force for specific MSE construction methods under water, specific types of traffic barriers constructed over MSE walls, and facing attachments to temporary facings.
Figure 14. Lift construction sequence for geosynthetic faced MSE walls.
Figure 15. Typical geosynthetic face construction.
Figure 16. Types of geosynthetic reinforced soil wall facing.
Figure 17. Reinforced slope construction: a) georgic and fill replacement; b) soil fill erosion control mat placement; and c) finished, vegetated 1:1 slope.
CHAPTER 3
SOIL REINFORCEMENT PRINCIPLES AND SYSTEM DESIGN PROPERTIES

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

The objectives of this chapter are to develop:
- An understanding of soil-reinforcement interaction.
- Introduce normalized pullout capacity concepts.
- Develop design soil parameters for select backfill, retained fill and foundation bearing capacity.
- Establish structural design properties.

3.1 OVERVIEW

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

3.2 REINFORCED SOIL CONCEPTS

A reinforced soil mass is somewhat analogous to reinforced concrete in that the mechanical properties of the mass are improved by reinforcement placed parallel to the principal strain direction to compensate for soil's lack of tensile resistance. The improved tensile properties are a result of the interaction between the reinforcement and the soil. The composite material has the following characteristics:
• Stress transfer between the soil and reinforcement takes place continuously along the reinforcement.
• Reinforcements are distributed throughout the soil mass with a degree of regularity and must not be localized.

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

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

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

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

Mode of Reinforcement Action
The primary function of reinforcements is to restrain soil deformations. In so doing, stresses are transferred from the soil to the reinforcement. These stresses are carried by the reinforcement in two ways: in tension or in shear and bending.

Tension is the most common mode of action of tensile reinforcements. All "longitudinal" reinforcing elements (i.e., reinforcing elements aligned in the direction of soil extension) are generally subjected to high tensile stresses. Tensile stresses are also developed in flexible reinforcements that cross shear planes.

Shear and Bending. "Transverse" reinforcing elements that have some rigidity, can withstand shear stress and bending moments.
Figure 18. Stress transfer mechanisms for soil reinforcement.
3.3 SOIL REINFORCEMENT INTERACTION USING NORMALIZED CONCEPTS

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

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

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

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

The long-term pullout performance (i.e., displacement under constant design load) is predominantly controlled by the creep characteristics of the soil and the reinforcement material. Soil reinforcement systems will generally not be used with cohesive soils susceptible to creep. Therefore, creep is primarily an issue of the type of reinforcement. Table 4 provides, for generic reinforcement types, the basic aspects of pullout performance in terms of the main load transfer mechanism, relative soil-to-reinforcement displacement required to fully mobilize the pullout resistance, and creep potential of the reinforcement in granular (and low plasticity cohesive) soils.
**Table 4. Basic aspects of reinforcement pullout performance in granular and cohesive soils of low plasticity.**

<table>
<thead>
<tr>
<th>Generic Reinforcement Type</th>
<th>Major Load Transfer Mechanism</th>
<th>Range of Displacement at Specimen Front</th>
<th>Long Term Deformation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inextensible strips</td>
<td>Smooth</td>
<td>1.2 mm</td>
<td>Noncreeping</td>
</tr>
<tr>
<td></td>
<td>Ribbed</td>
<td>12 mm</td>
<td></td>
</tr>
<tr>
<td>Extensible composite plastic strips</td>
<td>Frictional</td>
<td>Dependent on reinforcement extensibility</td>
<td>Dependent on reinforcement structure and polymer creep</td>
</tr>
<tr>
<td>Extensible sheets</td>
<td>Geotextiles</td>
<td>Frictional</td>
<td>Dependent on reinforcement extensibility (25 to 100 mm)</td>
</tr>
<tr>
<td>Inextensible grids</td>
<td>bar mats</td>
<td>Passive + frictional</td>
<td>12 to 50 mm</td>
</tr>
<tr>
<td></td>
<td>welded wire meshes</td>
<td>Frictional + passive</td>
<td>12 to 50 mm</td>
</tr>
<tr>
<td>Extensible grids</td>
<td>Geogrids</td>
<td>Frictional + passive</td>
<td>Dependent on extensibility (25 to 50 mm)</td>
</tr>
<tr>
<td></td>
<td>woven wire meshes</td>
<td>Frictional + passive</td>
<td>25 to 50 mm</td>
</tr>
</tbody>
</table>

**b. Estimate of the Reinforcement Pullout Capacity in RSS and MSE Structures**

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

For design and comparison purposes, a normalized definition of pullout resistance will be used throughout the manual. The pullout resistance, $P_r$, of the reinforcement per unit width of reinforcement is given by:
\[ P_r = F^* \cdot \alpha \cdot \sigma' \cdot L_e \cdot C \]  

(1)

where: \( L_e \cdot C \) = the total surface area per unit width of the reinforcement in the resistive zone behind the failure surface

\( L_e \) = the embedment or adherence length in the resisting zone behind the failure surface

\( C \) = the reinforcement effective unit perimeter; e.g., \( C = 2 \) for strips, grids, and sheets

\( F^* \) = the pullout resistance (or friction-bearing-interaction) factor

\( \alpha \) = a scale effect correction factor to account for a non linear stress reduction over the embedded length of highly extensible reinforcements, based on laboratory data (generally 1.0 for metallic reinforcements and 0.6 to 1.0 for geosynthetic reinforcements, see table 5).

\( \sigma' \) = the effective vertical stress at the soil-reinforcement interfaces.

The correction factor \( \alpha \) depends, therefore, primarily upon the strain softening of the compacted granular backfill material, the extensibility and the length of the reinforcement. For inextensible reinforcement, \( \alpha \) is approximately 1, but it can be substantially smaller than 1 for extensible reinforcements. The \( \alpha \) factor (a scale correction factor) can be obtained from pullout tests on reinforcements with different lengths as presented in appendix A or derived using analytical or numerical load transfer models which have been "calibrated" through numerical test simulations. In the absence of test data, \( \alpha = 0.8 \) for geogrids and \( \alpha = 0.6 \) for geotextiles (extensible sheets) is recommended (see table 5).

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

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

or,

\[ F^* = F_q \cdot \alpha_\beta + \tan \rho \]  

(2)

where: \( F_q \) = the embedment (or surcharge) bearing capacity factor

\( \alpha_\beta \) = a bearing factor for passive resistance which is based on the thickness per unit width of the bearing member.

\( \rho \) = the soil-reinforcement interaction friction angle.
The pullout capacity parameters for equation 2 are summarized in table 5 and figure 19 for the soil reinforcement systems considered in this manual.

A significant number of laboratory pullout tests have been performed for many commonly used reinforcement backfill combinations and correlated to representative field pullout tests. Therefore, the need for additional laboratory and/or field pullout tests, should be limited to reinforcement/backfill combinations, where this data is sparse or non-existent. Where applicable, laboratory pullout tests should be made in a device consisting of a test box with the following minimum dimensions: 760 mm (30 inches) wide, 1210 mm (48 inches) long, and 450 mm (18 inches) deep. The reinforcement samples should be horizontally embedded between two, 150-mm (6-inch) layers of soil. The reinforcement specimen should be pulled horizontally out the front of the box through a split removable door. The test normal load should be applied vertically to the sample by pressurizing an air bag placed between a cover plate and a reaction plate resting on the soil. The pullout movement should be approximately 1.0 mm (0.04-inch) per minute and monitored using dial gauges mounted to the front of the specimen. Note that this test procedure provides a short-term pullout capacity and does not account for soil or reinforcement creep deformations, which may be of significance in RSS structures utilizing fine grained backfills.

When using laboratory pullout tests to determine design parameters, vertical stress variations and reinforcement element configurations for the actual project should be used. Tests should be performed on samples with a minimum embedded length of 600 mm (24 inches). The pullout resistance is the greater of the peak pullout resistance value prior to or the value achieved at a maximum deformation of 20 mm (¾-inch) as measured at the front of the embedded section for inextensible reinforcements and 15 mm (5/8-inch) as measured at the end of the embedded sample for extensible reinforcements. This allowable deflection criteria is based on a need to limit the structure deformations, which are necessary to develop sufficient pullout capacity.
### Table 5. Summary of pullout capacity design parameters.

<table>
<thead>
<tr>
<th>Reinforcement Type</th>
<th>S&lt;sub&gt;opt&lt;/sub&gt;</th>
<th>Grid Spacing</th>
<th>Tan ρ</th>
<th>F&lt;sub&gt;q&lt;/sub&gt;</th>
<th>α&lt;sub&gt;α&lt;/sub&gt;</th>
<th>A Default Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inextensible strips</td>
<td>NA</td>
<td>NA</td>
<td>Obtain Tan ρ from tests, or use default values</td>
<td>NA</td>
<td>NA</td>
<td>1.0</td>
</tr>
<tr>
<td>Inextensible grids (bar mats and welded wire)</td>
<td>t(F&lt;sub&gt;q&lt;/sub&gt;)/(2Tanφ)</td>
<td>S&lt;sub&gt;i&lt;/sub&gt; ≤ S&lt;sub&gt;opt&lt;/sub&gt;</td>
<td>Obtain Tan ρ from tests</td>
<td>NA</td>
<td>NA</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>t(F&lt;sub&gt;q&lt;/sub&gt;)/(2Tanφ)</td>
<td>S&lt;sub&gt;i&lt;/sub&gt; &gt; S&lt;sub&gt;opt&lt;/sub&gt;</td>
<td>NA</td>
<td>Obtain F&lt;sub&gt;q&lt;/sub&gt; from tests, or use default values</td>
<td>t/(2S&lt;sub&gt;i&lt;/sub&gt;)</td>
<td>1.0</td>
</tr>
<tr>
<td>Extensible grids:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(Min. grid opening)/d&lt;sub&gt;50&lt;/sub&gt; &gt; 1</td>
<td>t(F&lt;sub&gt;q&lt;/sub&gt;)/(2Tanφ)</td>
<td>S&lt;sub&gt;i&lt;/sub&gt; ≤ S&lt;sub&gt;opt&lt;/sub&gt;</td>
<td>Obtain Tan ρ from tests</td>
<td>NA</td>
<td>NA</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td>t(F&lt;sub&gt;q&lt;/sub&gt;)/(2Tanφ)</td>
<td>S&lt;sub&gt;i&lt;/sub&gt; &gt; S&lt;sub&gt;opt&lt;/sub&gt;</td>
<td>NA</td>
<td>Obtain F&lt;sub&gt;q&lt;/sub&gt; from tests, or use default values</td>
<td>(f&lt;sub&gt;b&lt;/sub&gt;t)/(2S&lt;sub&gt;i&lt;/sub&gt;)</td>
<td>0.8</td>
</tr>
<tr>
<td>(Min. grid opening)/d&lt;sub&gt;50&lt;/sub&gt; &lt; 1</td>
<td>NA</td>
<td>Obtain Tan ρ from tests</td>
<td>NA</td>
<td>NA</td>
<td>0.8</td>
<td></td>
</tr>
<tr>
<td>Extensible sheets</td>
<td>NA</td>
<td>Obtain Tan ρ from tests</td>
<td>NA</td>
<td>NA</td>
<td>0.6</td>
<td></td>
</tr>
</tbody>
</table>

**NOTES:**

It is acceptable to use the empirical values provided in or referenced by this table to determine F<sup>α</sup> in the absence of product and backfill specific test data, provided granular backfill as specified in Article 7.3.6.3 of Division II of 1996 AASHTO Standard Specifications for Highway Bridges is used and C<sub>u</sub> $\geq$ 4. For backfill outside these limits, tests must be run.

Pullout testing to determine α is recommended if α shown in table is less than 1.0. These values of α represent highly extensible geosynthetics.

For grids where Tan ρ is applicable, apply Tan ρ to the entire surface area of the reinforcement sheet (i.e., soil and grid), not just the surface area of the grid elements.

NA means "not applicable." ϕ is the soil friction angle. ρ is the interface friction angle mobilized along the reinforcement. S<sub>opt</sub> is the optimum transverse grid element spacing to mobilize maximum pullout resistance as obtained from pullout tests (typically 150 mm or greater). S<sub>i</sub> is the spacing of the transverse grid elements. t is the thickness of the transverse elements. F<sub>q</sub> is the embedment (or surcharge) bearing capacity factor. α<sub>α</sub> is a structural geometric factor for passive resistance. f<sub>b</sub> is the fraction of the transverse member on which bearing can be fully developed (typically ranging from 0.6 to 1.0) as obtained from an evaluation of the bearing surface shape. d<sub>50</sub> is the backfill grain size at 50% passing by weight. α is the scale effect correction factor. Definition of the geometric variables are illustrated in figure 19.
Figure 19. Definition of grid dimensions for calculating pullout capacity.
Long-term pullout tests to assess soil/reinforcement creep behavior should be conducted when silt or clay reinforced backfill is being used. Soil properties and reinforcement type will determine if the allowable pullout resistance is governed by creep deformations. The placement and compaction procedures for both short-term and long-term pullout tests should simulate field conditions. The allowable deformation criteria in the previous paragraph should be applied.

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

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

In the absence of site specific pullout testing data, it is reasonable to use these semi empirical relationships in conjunction with the standard specifications for backfill to provide a conservative evaluation of pullout resistance.

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

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

$$F^* = \tan \varphi \text{ at a depth of 6 m (20 ft) and below}$$

where $C_u$ is the uniformity coefficient of the backfill ($D_{60}/D_{10}$). If the specific $C_u$ for the wall backfill is unknown at design time a $C_u$ of 4 should be assumed (i.e., $F^* = 1.8$ at the top of the wall), for backfills meeting the requirements of section 3.4 of this chapter.

For steel grid reinforcements with transverse spacing $S_t > 150$ mm (6 inches) (see figure 19), $F^*$ is a function of a bearing or embedment factor ($F_q$), applied over the contributing bearing $\alpha$, as follows:

$$F^* = F_q \alpha = 40 \alpha = 40 (t/2S_t) = 20 (t/S_t) \text{ at the top of the structure}$$

$$F^* = F_q \alpha = 20 \alpha = 20 (t/2S_t) = 10 (t/S_t) \text{ at a depth of 6 m (20 ft) and below}$$

where $t$ is the thickness of the transverse bar. $S_t$ shall be uniform throughout the length of the reinforcement rather than having transverse grid members concentrated only in the resistant zone. For sloping backfills see figure 30 in Chapter 4.

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

**Ref:** Federal Highway Administration

Publication No. FHWA-NHI-00-043

"Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines"
\[ F^* = \frac{2}{3} \tan \varphi \] (7)

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

c. Interface Shear

The interface shear between sheet type geosynthetics (geotextiles, geogrids and geocomposite drains) and the soil is often lower than the friction angle of the soil itself and can form a slip plane. Therefore the interface friction coefficient \( \tan \rho \) must be determined in order to evaluate sliding along the geosynthetic interface with the reinforced fill and, if appropriate, the foundation or retained fill soil. The interface friction angle \( \rho \) is determined from soil geosynthetic direct shear tests in accordance with ASTM D 5321. In the absence of test results, the interface friction coefficient can be conservatively taken as \( \frac{2}{3} \tan \varphi \) for geotextiles, geogrids and geonet type drainage composites. Other geosynthetics such as geomembranes and some geocomposite drain cores may have much lower interface values and tests should accordingly be performed.

3.4 ESTABLISHMENT OF ENGINEERING PROPERTIES BASED ON SITE EXPLORATION AND TESTING

a. Foundation Soils

Determination of engineering properties for foundation soils should be focused on establishment of bearing capacity, settlement potential, and position of groundwater levels. For bearing capacity determinations, frictional and cohesive parameters (\( \varphi, c \)) as well as unit weights (\( \gamma_T \)) and groundwater position are normally required in order to calculate bearing capacity in accordance with Article 4.4.7 for soil and 4.4.8 for rock in 1996 AASHTO Standard Specifications for Highway Bridges. The effects of load inclination and footing shape may be omitted and the minimum Factor of Safety may be taken as 2.5 for Group I loading.

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

Major foundation weakness and compressibility may require the consideration of ground improvement techniques to achieve adequate bearing capacity, or limiting total or differential settlement. Techniques successfully used, include surcharging with or without wick drains, stone
columns, dynamic compaction, and the use of lightweight fill to reduce settlement. Additional information on ground improvement techniques can be found in the FHWA’s Ground Improvement Manual DP116. As an alternate, MSE structures with faces constructed of geosynthetic wraps, welded wire mesh or gabion baskets, which will tolerate significant differential settlement, could be constructed and permanent facings such as concrete panels attached after the settlement has occurred. Of particular concern, are situations where the MSEW structure may terminate adjacent to a rigidly supported structure such as a pile supported abutment at the end of a retained approach fill.

_Evaluation of these foundation related issues are typically beyond the scope of services provided by wall/slope system suppliers. Evaluations of this type are the responsibility of agency engineers or consultant geotechnical designers._

b. Reinforced Backfill Soil

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

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

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

<table>
<thead>
<tr>
<th>U.S. Sieve Size</th>
<th>Percent Passing&lt;sup&gt;(a)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td>102 mm (4 in)&lt;sup&gt;(a,b)&lt;/sup&gt;</td>
<td>100</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>0-60</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0-15</td>
</tr>
</tbody>
</table>

<sup>(a)</sup> Ref: Federal Highway Administration

Publication No. FHWA-NHI-00-043

“Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines”
Plasticity Index (PI) shall not exceed 6.
(a) In order to apply default F* values, Cu, should be greater than or equal to 4.
(b) As a result of recent research on construction survivability of geosynthetics and epoxy coated reinforcements, it is recommended that the maximum particle size for these materials be reduced to 19 mm (¾-inch) for geosynthetics, and epoxy and PVC coated reinforcements unless tests are or have been performed to evaluate the extent of construction damage anticipated for the specific fill material and reinforcement combination.

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

The fill material must be free of organic matter and other deleterious substances, as these materials not only enhance corrosion but also result in excessive settlements. The compaction specifications should include a specified lift thickness and allowable range of moisture content with reference to optimum. The compaction requirements of backfill are different in close proximity to the wall facing (within 1.5 to 2 m). Lighter compaction equipment is used near the wall face to prevent buildup of high lateral pressures from the compaction and to prevent facing panel movement. Because of the use of this lighter equipment, a backfill material of good quality in terms of both friction and drainage, such as crushed stone is recommended close to the face of the wall to provide adequate strength and tolerable settlement in this zone. It should be noted that granular fill containing even a few percent fines may not be free draining and drainage requirements should always be carefully evaluated.

For RSS structures, less select backfill can be used as facings are typically flexible and can tolerate some distortion during construction. Even so, a high quality embankment fill meeting the following gradation requirements to facilitate compaction and minimize reinforcement requirements is recommended. The following guidelines are provided as recommended backfill requirements for RSS construction:

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Percent Passing</th>
</tr>
</thead>
<tbody>
<tr>
<td>20 mm*</td>
<td>100</td>
</tr>
<tr>
<td>4.76 mm (No. 4)</td>
<td>100 - 20</td>
</tr>
<tr>
<td>0.425 mm (No. 40)</td>
<td>0 - 60</td>
</tr>
<tr>
<td>0.075 mm (No. 200)</td>
<td>0 - 50</td>
</tr>
</tbody>
</table>

Plasticity Index (PI) # 20 (AASHTO T-90)
Soundness: Magnesium sulfate soundness loss less than 30% after 4 cycles, based on AASHTO T-104 or equivalent sodium sulfate soundness of less than 15 percent after 5 cycles.

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

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

The reinforced fill criteria outlined above represent materials that have been successfully used throughout the United States and resulted in excellent structure performance. Peak shear strength parameters are used in the analysis. For MSE walls, a lower bound frictional strength of 34 degrees would be consistent with the specified fill, although some nearly uniform fine sands meeting the specifications limits may exhibit friction angles of 31 to 32 degrees. Higher values may be used if substantiated by laboratory direct shear or triaxial test results for the site specific material used or proposed. However, extreme caution is advised for use of friction angles above 40 degrees for design due to a lack of field performance data and questions concerning mobilization of shear strength above that value.

Fill materials outside of these gradation and plasticity index requirements have been used successfully; however, problems including significant distortion and structural failure have also been observed. While there may be a significant savings in using lower quality backfill, property values must be carefully evaluated with respect to influence on both internal and external stability. For MSE walls constructed with reinforced fill containing more than 15% passing a 0.075 mm (#200) sieve and/or the PI exceeds 6, both total and effective shear strength parameters should be evaluated in order to obtain an accurate assessment of horizontal stresses, sliding, compound failure (behind and through the reinforced zone) and the influence of drainage on the analysis. Both long-term and short-term pullout tests as well as soil/reinforcement interface friction tests should be performed. Settlement characteristics must be carefully evaluated, especially in relation to downdrag stresses imposed on connections at the face and settlement of supported structures. Drainage requirements at the back, face and beneath the reinforced zone must be carefully evaluated (e.g., use flow nets to evaluate influence of seepage forces and hydrostatic pressure).

Electrochemical tests should be performed on the backfill to obtain data for evaluating degradation of reinforcements and facing connections. Moisture and density control during construction must be carefully controlled in order to obtain strength and interaction values. Deformation during construction also must be carefully monitored and maintained within defined design limits. Performance monitoring is also recommended for backfill soils that fall outside of the requirements listed above, as detailed in chapter 9.

For RSS structures, where a considerably greater percentage of fines (minus #200 sieve) is permitted, lower bound values of frictional strength equal to 28 to 30 degrees would be reasonable for the backfill requirements listed. A significant economy could again be achieved if laboratory direct shear or triaxial test results on the proposed fill are performed, justifying a higher value. Likewise, soils outside the gradation range listed should be carefully evaluated and monitored.

Ref: Federal Highway Administration
Publication No. FHWA-NHI-00-043
"Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines"
c. Retained Fill
The key engineering properties required are strength and unit weight based on evaluation and testing of subsurface data. Friction angles (φ) and unit weight (γT) may be determined from either drained direct shear tests or consolidated drained triaxial tests. If undisturbed samples cannot be obtained, friction angles may be obtained from in-situ tests or by correlations with index properties. The strength properties are required for the determination of the coefficients of earth pressure used in design. In addition, the position of groundwater levels above the proposed base of construction must be determined in order to plan an appropriate drainage scheme. For most retained fills lower bound frictional strength values of 28 to 30 degrees are reasonable for granular and low plasticity cohesive soils. For highly plastic retained fills (PI>40), even lower values would be indicated and should be evaluated for both drained and undrained conditions.

d. Electrochemical Properties
The design of buried steel elements of MSE structures is predicated on backfills exhibiting minimum or maximum electrochemical index properties and then designing the structure for maximum corrosion rates associated with these properties. These recommended index properties and their corresponding limits are shown in table 6.

Reinforced fill soils must meet the indicated criteria to be qualified for use in MSE construction using steel reinforcements. Where geosynthetic reinforcements are planned, the limits for electrochemical criteria would vary depending on the polymer. Tentative limits, based on current research are shown in table 7.

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Criteria</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Resistivity</td>
<td>&gt;3000 ohm-cm</td>
<td>AASHTO T-288-91</td>
</tr>
<tr>
<td>pH</td>
<td>&gt;5&lt;10</td>
<td>AASHTO T-289-91</td>
</tr>
<tr>
<td>Chlorides</td>
<td>&lt;100 PPM</td>
<td>AASHTO T-291-91</td>
</tr>
<tr>
<td>Sulfates</td>
<td>&lt;200 PPM</td>
<td>AASHTO T-290-91</td>
</tr>
<tr>
<td>Organic Content</td>
<td>1% max.</td>
<td>AASHTO T-267-86</td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Base Polymer</th>
<th>Property</th>
<th>Criteria</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyester (PET)</td>
<td>pH</td>
<td>&gt;3&lt;9</td>
<td>AASHTO T-289-91</td>
</tr>
<tr>
<td>Polyolefin (PP &amp; HDPE)</td>
<td>pH</td>
<td>&gt;3</td>
<td>AASHTO T-289-91</td>
</tr>
</tbody>
</table>
3.5 ESTABLISHMENT OF STRUCTURAL DESIGN PROPERTIES
The structural design properties of reinforcement materials are a function of geometric characteristics, strength and stiffness, durability, and material type. The two most commonly used reinforcement materials, steel and geosynthetics, must be considered separately as follows:

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

The coverage ratio $R_c$ is used to relate the force per unit width of discrete reinforcement to the force per unit width required across the entire structure.

$$ R_c = \frac{b}{S} $$  \hspace{1cm} (8)

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

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

b. Strength Properties

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

$$ E_c = E_n - E_R $$  \hspace{1cm} (9)

where $E_c$ is the thickness of the reinforcement at the end of the design life, $E_n$ the nominal thickness at construction, and $E_R$ the sacrificial thickness of metal expected to be lost by uniform corrosion during the service life of the structure.
The allowable tensile force per unit width of reinforcement, $T_a$, is obtained as follows:

\[
T_a = \frac{0.55 \, F_y A_c}{b}
\]

for steel strips (10)

and

\[
T_a = \frac{0.48 \, F_y A_c}{b}
\]

for steel grids connected to concrete panels or blocks (11)

(Note: 0.55 $F_y$ may be used for steel grids with flexible facings)

where:

- $b =$ the gross width of the strip, sheet or grid
- $F_y =$ yield stress of steel
- $A_c =$ design cross section area of the steel, defined as the original cross section area minus corrosion losses anticipated to occur during the design life of the wall.

The allowable tensile stress for steel reinforcements and connections for permanent structures is developed in accordance with Article 10.32, in particular table 10.32.1A of AASHTO Standard Specifications for Highway Bridges. These requirements result in an allowable tensile stress for steel strip reinforcement, in the wall backfill away from the wall face connections, of 0.55 $F_y$. The 0.55 factor applied to $F_y$ for permanent structures accounts for uncertainties in structure geometry, fill properties, externally applied loads, the potential for local overstress due to load nonuniformities, and uncertainties in long-term reinforcement strength and is equivalent to a factor of safety of 1.82 (i.e. 1/0.55). For grid reinforcing members connected to a rigid facing element (e.g., a concrete panel or block), the allowable tensile stress is reduced to a 0.48 $F_y$ providing an implied factor of safety of 2.08 to account for the greater potential for local overstress due to load nonuniformities for steel grids than for steel strips or bars. Transverse and longitudinal grid members are sized in accordance with ASTM A-185. For temporary structures (i.e., design lives of 3 years or less), AASHTO permits an increase to the allowable tensile stress by 40 percent.

The quantities needed for determination of $A_c$ for steel strips and grids are shown in figure 20. Typical dimensions for common steel reinforcements are provided in appendix D. The use of hardened and otherwise low strain (very high strength) steels may increase the potential for catastrophic failure, therefore, a lower allowable material stress may be warranted with such materials.

For metallic reinforcement, the life of the structure will depend on the corrosion resistance of the reinforcement. Practically all the metallic reinforcements used in construction of embankments and walls, whether they are strips, bar mats, or wire mesh, are made of galvanized mild steel. Woven meshes with PVC coatings provide some corrosion protection, provided the coating is not
significantly damaged during construction. Epoxy coatings can be used for corrosion protection, but are susceptible to construction damage, which can significantly reduce its effectiveness. When PVC or epoxy coatings are used, the maximum particle size of the backfill should be restricted to 19 mm (¾-inch) or less to reduce the potential for construction damage. For a more detailed discussion of requirements, refer to the Corrosion/Degradation document.

Several State transportation departments have used resin-bonded epoxy coated steel reinforcing elements. The effectiveness of these coatings in MSEW structures has not been sufficiently demonstrated and their widespread use cannot be presently endorsed. If used a minimum coating thickness of 0.41 mm (16 mils) is recommended applied in accordance with ASTM A-884 for grid reinforcement and AASHTO M-284 for strip reinforcement. Their in-ground life is presently estimated at 20 years. Where other metals, such as aluminum alloys or stainless steel have been used, corrosion, unexpectedly, has been a severe problem, and their use has been discontinued.

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

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

The majority of MSE walls constructed to date have used galvanized steel and backfill materials with low corrosive potential. A minimum galvanization coating of 0.61 kg/m² (2.0 oz/ft²) or 86 μm (3.4 mils) thickness applied in accordance with AASHTO M 111 (ASTM A 123) for strip type reinforcements or ASTM D 641 for bar mat or grid type steel reinforcements is required, per AASHTO Standard Specifications for Highway Bridges. The zinc coating provides a sacrificial anode that corrodes while protecting the base metal. Galvanization also assists in preventing the formation of pits in the base metal during the first years of aggressive corrosion. After the zinc is oxidized (consumed), corrosion of the base metal starts.
Figure 20. Parameters for metal reinforcement strength calculations.

\[ A_b = b E_b \]
\[ E_b = \text{strip thickness corrected for corrosion loss.} \]

\[ A_o = (\text{No. of longitudinal bars}) \times \pi \times \frac{D^*^2}{4} \]
\[ D^* = \text{diameter of bar or wire corrected for corrosion loss.} \]
\[ b = \text{unit width of reinforcement (if reinforcement is continuous count number of bars for reinforcement width of 1 unit).} \]

\[ T_{\text{max}} = T_o R_o = \frac{FS A_o F_y R_o}{b} \]

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

\( FS \) = factor of safety

\( F_y \) = yield strength of steel

\( R_o \) = reinforcement coverage ratio = \( \frac{b}{S_h} \)

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

\( T_{\text{max}} \) = maximum load applied to reinforcement (load/unit wall width).
The corrosion rates presented below are suitable for conservative design. **These rates assume a mildly corrosive backfill material having the controlled electrochemical property limits** that are discussed under electrochemical properties in this chapter.

**Corrosion Rates** - mildly corrosive backfill

For zinc/side

- 15 μm/year (0.6 mils/yr) (first 2 years)
- 4 μm/year (0.16 mils/yr) (thereafter)

For residual carbon steel/side

- 12 μm/year (0.5 mils/yr) (thereafter)

Based on these rates, complete corrosion of galvanization with the minimum required thickness of 86 μm (3.4 mils) (AASHTO M 111) is estimated to occur during the first 16 years and a carbon steel thickness or diameter loss of 1.42 mm to 2.02 mm (0.055 in to 0.08 in) would be anticipated over the remaining years of a 75 to 100 year design life, respectively. The designer of an MSE structure should also consider the potential for changes in the reinforced backfill environment during the structure's service life. In certain parts of the United States, it can be expected that deicing salts might cause such an environment change. For this problem, the depth of chloride infiltration and concentration are of concern.

For permanent structures directly supporting roadways exposed to deicing salts, limited data indicate that the upper 2.5 m (8 ft) of the reinforced backfill (as measured from the roadway surface) are affected by higher corrosion rates not presently defined. Under these conditions, it is recommended that a 30 mil (minimum) geomembrane be placed below the road base and tied into a drainage system to mitigate the penetration of the deicing salts in lieu of higher corrosion rates as shown in the Design Details section in Chapter 4.

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

- Structures exposed to a marine or other chloride-rich environment. (Excluding locations where de-icing salts are used.) For marine saltwater structures, carbon steel losses on the order of 80 μm (3.2 mils) per side should be anticipated in the first few years, reducing to 17 to 20 μm (0.67 to 0.7 mils) thereafter. Zinc losses are likely to be quite rapid as compared to losses in backfills meeting the MSE electrochemical criteria. Total loss of zinc (86 am) should be anticipated in the first year.

- Structures exposed to stray currents, such as from nearby underground power lines, and structures supporting or located adjacent to electrical railways. Each of these situations creates a special set of conditions that should be specifically analyzed by a corrosion specialist.
**Geosynthetic Reinforcement**

Selection of Tₐ for geosynthetic reinforcement is more complex than for steel. The tensile properties of geosynthetics are affected by environmental factors such as creep, installation damage, aging, temperature, and confining stress. Furthermore, characteristics of geosynthetic products manufactured with the same base polymer can vary widely, and the details of polymer behavior for in-ground use are not completely understood. Ideally, Tₐ should be determined by thorough consideration of allowable elongation, creep potential and all possible strength degradation mechanisms.

Polymeric reinforcement, although not susceptible to corrosion, may degrade due to physicochemical activity in the soil such as hydrolysis, oxidation, and environmental stress cracking depending on polymer type. In addition, these materials are susceptible to installation damage and the effects of high temperature at the facing and connections. Temperatures can be as high as 50°C compared with the normal range of in-ground temperature of 12°C in cold and temperate climates to 30°C in arid desert climates.

Degradation most commonly occurs from mechanical damage, long-term time dependent degradation caused by stress (creep), deterioration from exposure to ultraviolet light, and chemical or biological interaction with the surrounding environment. Because of varying polymer types, quality, additives and product geometry, each geosynthetic is different in its resistance to aging and attack by different chemical and biological agents. Therefore, each product must be investigated individually.

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

Polyolefin products (PP and HDPE) are susceptible to aging strength losses due to oxidation (contact with oxygen) and or high temperatures. The level of oxygen in reinforced fills is a function of soil porosity, ground water location and other factors, and has been found to be slightly less than oxygen levels in the atmosphere (21 percent). Therefore, oxidation of geosynthetics in the ground may proceed at an equal rate than those used above ground. Oxidation is accelerated by the presence of transition metals (Fe, Cu, Mn, Co, Cr) in the backfill as found in acid sulphate soils, slag fills, other industrial wastes or mine tailings containing transition metals. It should be noted that the resistance of polyolefin geosynthetics to oxidation is primarily a function of the proprietary antioxidant package added to the base resin, which differs for each product brand, even when formulated with the same base resin.

The relative resistance of polymers to these identified regimes is shown in table 8 and a choice can be made, therefore, consistent with the in-ground regimes indicated. Most geosynthetic reinforcement is buried, and therefore ultraviolet (UV) stability is only of concern during
construction and when the geosynthetic is used to wrap the wall or slope face. If used in exposed locations, the geosynthetic should be protected with coatings or facing units to prevent deterioration. Vegetative covers can also be considered in the case of open weave geotextiles or geogrids. Thick geosynthetics with ultraviolet stabilizers can be left exposed for several years or more without protection; however, long-term maintenance should be anticipated because of both UV deterioration and possible vandalism.

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

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

Table 8. Anticipated resistance of polymers to specific environments.

<table>
<thead>
<tr>
<th>Soil Environment</th>
<th>Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>PET</td>
</tr>
<tr>
<td>Acid Sulphate Soils</td>
<td>NE</td>
</tr>
<tr>
<td>Organic Soils</td>
<td>NE</td>
</tr>
<tr>
<td>Saline Soils pH &lt; 9</td>
<td>NE</td>
</tr>
<tr>
<td>Calcareous Soils</td>
<td>?</td>
</tr>
<tr>
<td>Modified Soils/Lime, Cement</td>
<td>?</td>
</tr>
<tr>
<td>Sodic Soils, pH &gt; 9</td>
<td>?</td>
</tr>
<tr>
<td>Soils with Transition Metals</td>
<td>NE</td>
</tr>
</tbody>
</table>

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

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

\[
T_a = \frac{T_{ULT}}{RF \cdot FS} = \frac{T_{al}}{FS}
\]

where \(T_a\) is the design long-term reinforcement tension load for the limit state, \(T_{ULT}\) the ultimate geosynthetic tensile strength and \(RF\) is the product of all applicable reduction factors and \(FS\) the overall factor of safety. \(T_a\) is the long-term material strength or more specifically:
where:

\[ T_{al} = \frac{T_{ULT}}{RF_{CR} \cdot RF_{D} \cdot RF_{ID}} \]  

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

\[ T_{ULT} \] = Ultimate (or yield) tensile strength from wide strip test (ASTM D 4595) for geotextiles and wide strip (ASTM D 4595) or single rib test (GR1:GG1) for geogrids (note, that the same test shall be used for definition of the geogrid creep reduction factor), based on minimum average roll value (MARV) for the product.

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

<table>
<thead>
<tr>
<th>Polymer Type</th>
<th>Creep Reduction Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyester</td>
<td>2.5 to 1.6</td>
</tr>
<tr>
<td>Polypropylene</td>
<td>5 to 4.0</td>
</tr>
<tr>
<td>High Density Polyethylene</td>
<td>5 to 2.6</td>
</tr>
</tbody>
</table>

\[ RF_{D} \] = Durability reduction factor. It is dependent on the susceptibility of the geosynthetic to attack by microorganisms, chemicals, thermal oxidation, hydrolysis and stress cracking, and can vary typically from 1.1 to 2.0. The minimum reduction factor shall be 1.1.

\[ RF_{ID} \] = Installation Damage reduction factor. It can range from 1.05 to 3.0, depending on backfill gradation and product mass per unit weight. The minimum reduction factor shall be 1.1 to account for testing uncertainties.

\[ FS \] = Overall factor of safety to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. For permanent, MSEW structures only, a minimum factor of safety of 1.5 has been typically used (thus \( T_{a} = T_{al} / 1.5 \)).

For RSS structures, it is taken as 1.0, as the required factor of safety, is accounted in the stability analysis (thus \( T_{a} = T_{al} \)).

\( T_{al} \) is typically obtained directly from the manufacturer. It typically includes reduction factors but does not include a design or material factor of safety, \( FS \). The determination of reduction factors for each geosynthetic product require extensive field and/or laboratory testing, briefly summarized as follows:
**Creep Reduction Factor, RF\textsubscript{cr}**
The creep reduction factor is obtained from long term laboratory creep testing as detailed in appendix B. This reduction factor is required to limit the load in the reinforcement to a level known as the creep limit, that will preclude creep rupture over the life of the structure. Creep in itself does not degrade the strength of the polymer. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. The creep reduction factor is the ratio of the ultimate load to the extrapolated maximum sustainable load (i.e., creep limit) within the design life of the structure (e.g., several years for temporary structures, 75 to 100 years for permanent structures).

For temporary structures, the maximum sustainable load is defined at a time equal to the temporary life of the structure.

**Durability Reduction Factor, RF\textsubscript{D}**
The protocol for testing to obtain this reduction factor have been proposed and are detailed in FHWA RD-97-144. In general, it consists of oven aging polyolefins (PP and HDPE) samples to accelerate oxidation and measure their strength reduction, as a function of time, temperature and oxygen concentration. This high temperature data must then be extrapolated to a temperature consistent with field conditions. For polyesters (PET) the aging is conducted in an aqueous media at varying pH's and relatively high temperature to accelerate hydrolysis, with data extrapolated to a temperature consistent with field conditions.

For more detailed explanations, see the companion Corrosion/Degradation document. The following recommendations are stated in this companion document in regards to defining a RF\textsubscript{D} factor.

With respect to aging degradation, current research results suggest the following:

**Polyester geosynthetics**
PET geosynthetics are recommended for use in environments characterized by 3 < pH < 9, only. The following reduction factors for PET aging (RF\textsubscript{D}) are presently indicated for a 100 year design life in the absence of product specific testing:
Table 9. Aging reduction factors, PET.

<table>
<thead>
<tr>
<th>No.</th>
<th>Product*</th>
<th>Reduction factor, $R_{FD}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$5 \leq \text{pH} \leq 8$</td>
</tr>
<tr>
<td>1</td>
<td>Geotextiles $M_n&lt;20,000$, $40&lt;\text{CEG}&lt;50$</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td>Coated geogrids, Geotextiles $M_n&gt;25,000$, $\text{CEG}&lt;30$</td>
<td>1.15</td>
</tr>
</tbody>
</table>

$M_n =$ number average molecular weight  
$\text{CEG} =$ carboxyl end group

* Use of materials outside the indicated pH or molecular property range requires specific product testing.

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

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

At present, heat aging protocols for PP products, at full or reduced atmospheric oxygen, with subsequent numerical analysis are available for PP products which exhibit no initial cracks or crazes in their as manufactured state, typically monofilaments. For PP products with initial crazes or cracks, typically tape products, or HDPE, heat aging testing protocols may change the nature of the product and therefore may lead to erroneous results. Alternate testing protocols using oxygen pressure as a time accelerator are under study and development.

Since each product has a unique and proprietary blend of antioxidants, product specific testing is required to determine the effective life span of protection at the in ground oxygen content. Limited data suggests that certain antioxidants are effective for up to 100 years in maintaining strength for in-ground use.

Ref: Federal Highway Administration
Publication No. FHWA-NHI-00-043
"Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines"
Installation Damage Reduction Factor, RF_{ID}.
Protocols for field testing for this reduction factor is detailed in the companion Corrosion/Degradation document and in ASTM D-5818. The protocol requires that the geosynthetic material is subjected to a backfilling and compaction cycle, consistent with field practice. The ratio of the initial strength, to the strength of retrieved samples defines this reduction factor. For reinforcement applications a minimum weight of 270 g/m² (7.9 oz/yd²) for geotextiles is recommended to minimize installation damage. This roughly corresponds to a Class 1 geotextile as specified in AASHTO M-288-96.

The following recommendations are stated in this companion document in regards to defining a RF_{ID} factor. For more detailed explanations, see the companion Corrosion/Degradation document.

To account for installation damage losses of strength where full-scale product-specific testing is not available, Table 10 may be used with consideration of the project specified backfill characteristics. In absence of project specific data the largest indicated reduction factor for each geosynthetic type should be used.

**Table 10. Installation damage reduction factors.**

<table>
<thead>
<tr>
<th>No.</th>
<th>Geosynthetic Type</th>
<th>Type 1 Backfill</th>
<th>Type 2 Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Max. Size 102mm</td>
<td>Max. Size 20mm</td>
</tr>
<tr>
<td></td>
<td></td>
<td>D_{50} about 30mm</td>
<td>D_{90} about 0.7mm</td>
</tr>
<tr>
<td>1</td>
<td>HDPE uniaxial geogrid</td>
<td>1.20 - 1.45</td>
<td>1.00 - 1.20</td>
</tr>
<tr>
<td>2</td>
<td>PP biaxial geogrid</td>
<td>1.20 - 1.45</td>
<td>1.00 - 1.20</td>
</tr>
<tr>
<td>3</td>
<td>PVC coated PET geogrid</td>
<td>1.30 - 1.85</td>
<td>1.10 - 1.30</td>
</tr>
<tr>
<td>4</td>
<td>Acrylic coated PET geogrid</td>
<td>1.30 - 2.05</td>
<td>1.20 - 1.40</td>
</tr>
<tr>
<td>5</td>
<td>Woven geotextiles (PP&amp;PET)</td>
<td>1.40 - 2.20</td>
<td>1.10 - 1.40</td>
</tr>
<tr>
<td>6</td>
<td>Non woven geotextiles (PP&amp;PET)</td>
<td>1.40 - 2.50</td>
<td>1.10 - 1.40</td>
</tr>
<tr>
<td>7</td>
<td>Slit film woven PP geotextile</td>
<td>1.60 - 3.00</td>
<td>1.10 - 2.00</td>
</tr>
</tbody>
</table>

*(1) Minimum weight 270 g/m² (7.9 oz/yd²)*

**Durability Reduction Factor, RF_{D}, at Wall Face Unit.**
As noted in section 4.3.e Connection Strength, the long-term environmental aging factor (RF_{D}) may be significantly different than that used in computing the allowable reinforcement strength T_{a}. Of particular concern is the use of polyester geogrid and geotextile reinforcements with concrete facings because of the potential high pH environment. It is recommended that the use of polyesters be limited to a pH range of > 3 and < 9, as noted in table 7.

It is also noted in Section 4.3.e, that PET geogrids and geotextiles should not be cast into concrete for connections, due to potential chemical degradation. Use of PET reinforcements connected to dry-cast MBW units by laying the reinforcement between units may be subject to additional strength reductions.

**Ref:** Federal Highway Administration
Publication No. FHWA-NHI-00-043
"Mechanically Stabilized Earth Walls and Reinforced Slopes Design and Construction Guidelines*
An FHWA sponsored field monitoring study to examine pH conditions within and adjacent to MBW units has been concluded\(^{(35)}\). This study provided a large database of pH measurements of 25 MSEW structures in the United States.

The results indicated that the pH regime within the blocks in the connection zone is only occasionally above 9 and then for only the first few years. The pH subsequently decreases to the pH of the ambient backfill\(^{(35)}\). It therefore appears that for coated PET geogrids no further reduction is warranted. For geotextiles a small further reduction should be considered to account for a few years at a pH in excess of 9.

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

**Factor of Safety, FS.**

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

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

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

For preliminary design of permanent structures or for applications defined by the user as not having severe consequences should poor performance or failure occur, the allowable tensile strength \(T_a\), may be evaluated without product specific data, as:

\[
T_a = \frac{T_{ULT}}{7 \cdot FS}
\]  

\(^{(14)}\)
Further, this reduction factor RF = 7, should be limited to projects where the project environment meets the following requirements:

- Granular soils (sands, gravels) used in the reinforced volume.
- 4.5 ≤ pH ≤ 9
- Site temperature < 30°C
- Maximum backfill particle size of 19 mm
- Maximum MSEW height is 10 m (33 ft) and
- Maximum RSS height is 15 m (50 ft)

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

The total reduction factor of 7 has been established by multiplying lower bound partial reduction factors obtained from currently available test data, for products which meet the minimum requirements in table 11.

*It should be noted that the total Reduction Factor may be reduced significantly with appropriate test data. It is not uncommon for products with creep, installation damage and aging data, to develop total Reduction Factors in the range of 3 to 6.*

For temporary applications not having severe consequences should poor performance or failure occur, a default value for RF of 3 rather than 7 could be considered.

*Serviceability*

Serviceability requirements for geosynthetic reinforcements are met through the use of low allowable stress levels resulting from reduction factors combined with the inherent constraining effects of granular soils. With regard to strain limits on the reinforcement, methods for determination of strain vary widely with no present consensus on an appropriate analytical method capable of modeling strains in the structure. Measurements from instrumented field structures have consistently measured much lower strain levels in the reinforcement (typically less than 1 percent) than predicted by most current analytical methods. *Therefore, until an appropriate method of determination is agreed upon, it is recommended that strain limit requirements not be imposed on the reinforcement.*
Table 11. Minimum requirements for use of default reduction factors for primary geosynthetic reinforcement.

<table>
<thead>
<tr>
<th>Type</th>
<th>Property</th>
<th>Test Method</th>
<th>Criteria to Allow Use of Default RF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polypropylene</td>
<td>UV Oxidation Resistance</td>
<td>ASTM D-4355</td>
<td>Min. 70% strength retained after 500 hrs. in weatherometer</td>
</tr>
<tr>
<td>Polyethylene</td>
<td>UV Oxidation Resistance</td>
<td>ASTM D-4355</td>
<td>Min. 70% strength retained after 500 hrs. in weatherometer</td>
</tr>
<tr>
<td>Polyester</td>
<td>Hydrolysis Resistance</td>
<td>Inherent Viscosity Method</td>
<td>Min. Number (Mn) Molecular Weight of 25,000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(ASTM D-4603) with Correlation or Determine Directly Using Gel Permeation Chromatography</td>
<td></td>
</tr>
<tr>
<td>Polyester</td>
<td>Hydrolysis Resistance</td>
<td>GRI GG7</td>
<td>Max. Carboxyl End Group Number of 30</td>
</tr>
<tr>
<td>All Polymers</td>
<td>Survivability</td>
<td>Weight per Unit Area,</td>
<td>Min. 270 g/m² (7.9 oz/yd²)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ASTM D-5261</td>
<td></td>
</tr>
<tr>
<td>All Polymers</td>
<td>% Post Consumer Recycled Material by Weight</td>
<td>Certification of Material used</td>
<td>Maximum 0%</td>
</tr>
</tbody>
</table>