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**Mechanically Stabilized Earth Structures – Part 1**

**By**

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**I. History**

**Ancient History of MSE Structures**

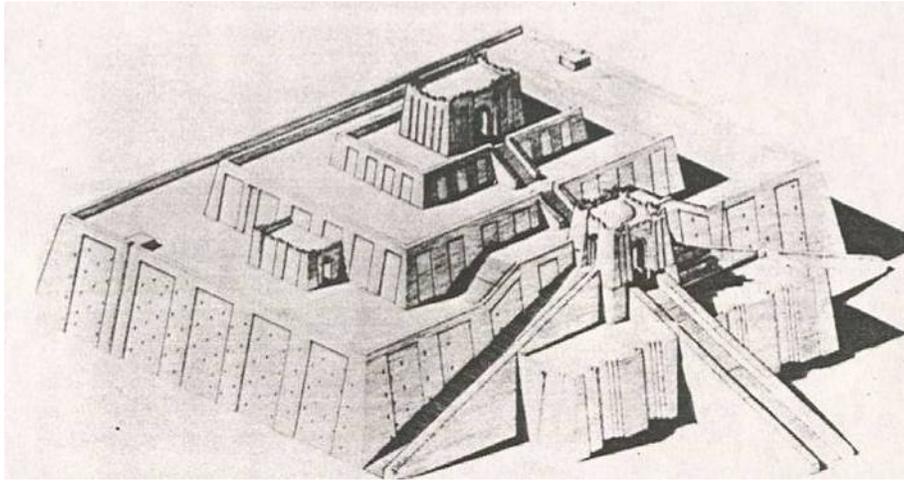
Reinforced soil technology is not modern as the use of tensile inclusions in soils has been around for several thousand years. Author Mark Twain stated, “*The ancients have stolen all our best ideas.*” Ancient civilizations used native material such as straw, tree branches, and plant material to reinforce the earth for centuries. Large religious towers called ziggurats were built by the Babylonian’s about 2,500 to 5,000 years ago in what is now modern day Iraq. The ziggurats had walls faced with clay bricks in an asphalt mortar with sun dried bricks behind. Layers of reed matting were laid as horizontal reinforcing sheets.

Another ether example of an ancient reinforced soil includes the Great Wall of China (2,000 BC) in which portions of the wall are reinforced with tree branches. Some early civilizations used sticks and branches to reinforce mud dwellings and some Roman Levees in Italy along Tiber River were reinforced with Reed mats 2,000 years ago.

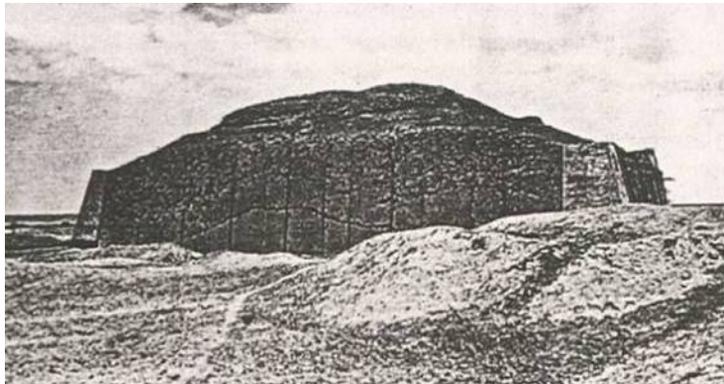


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**Ziggurat of Ur in Mesopotamia About 2500 B.C. (artist rendition)**



**Ziggurat of Ur in Mesopotamia (current day Iraq) about 2500 B.C.**



**Modern History of MSE Structures**

In 1822 tests were performed on a 4-ft high wall constructed with soil reinforced with horizontal layers of sacking material. Reinforcement was not connected to the facing, however stability was reported to have increased by 12-percent. This led to using reinforced soil in European military structures during the 1850's.

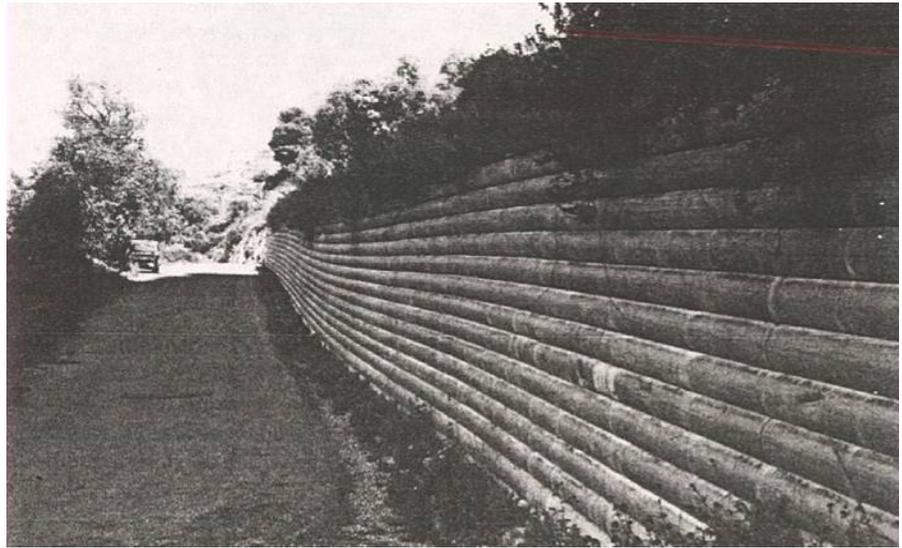
The technique of reinforcing soil was "rediscovered" for retaining walls by Henri Vidal in France in the early 1960's during a beach vacation when Vidal was placing pine needles into sand to make it stand vertically. Vidal's method incorporated discrete steel strips embedded within the soil mass. Since then, other types of reinforcement materials, classified as either inextensible or extensible, have been used to reinforce earth. *Inextensible reinforcement* (steel) is defined as a material that deforms considerably less than the surrounding soil at failure whereas *extensible reinforcement* (polyester or HPDE geogrids or polyester geotextiles) are materials that deform as much as the surrounding soil.



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The first MSE wall in United States using concrete panels and steel strip reinforcement was constructed in 1972 along Highway 39 in the San Gabriel Mountains, California. 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. In the early to mid 1980's many segmental block products were developed along with geosynthetic reinforcement that have greatly accelerated use of MSE walls and slopes over the past 30 years.

**First Highway Use of Modern MSE Earth Wall France between Nice and the Italian Border (1968)**



**First MSE wall constructed in the United States using steel reinforcement strips. This wall was constructed in 1972 along Highway 39 San Gabriel Mountains, California.**





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## II. Industry Design Methods and Programs

The two most popular design methodologies for MSE walls are the National Concrete Masonry Association (NCMA) and FHWA (Federal Highway Administration) methods. Reference to the design manuals are noted below:

- AASHTO - American Association of State Highway and Transportation Officials uses FHWA Publication No. “NHI-00-043, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes - Design and Construction Guidelines”, March 2001.
- NCMA - National Concrete Masonry Association uses the “NCMA Design Manual for Segmental Retaining Walls”, (First Edition 1993, Second Edition 1997 and Third Edition 2009)

### Basic Difference between NCMA and AASHTO

#### NCMA

L/H ratio  $\geq 60$  % of wall height.  
Uses Coulomb Earth pressure.  
Variable reinforcement lengths.  
Re-use of on-site soils (if possible).  
Uniform Loads – Limited Design.  
Reduced block embedment depths.  
Commercial & Private projects only.  
Simple Structures & Geometry.  
Uniform Surcharge Loading.  
Minimum design life of 75-years

#### AASHTO

L/H ratio  $\geq 70$  % of wall height.  
Uses Rankine Earth pressure.  
Uniform reinforcement lengths.  
Select fill in the reinforced zone.  
Uniform & Strip Loads - Full Design  
Minimum embedment of 2-feet.  
Public & Private projects.  
Complex Structures & Geometry.  
Uniform Surcharge, Strip & Footing Loads.  
Minimum design life of 75-years.

Design engineers should be aware and watch out for in-house design methods developed by block or reinforcement manufactures used in designing MSE walls. At this time and for nearly 20-years, the NCMA and AASHTO design methods have been and are considered the industry standards of practice in the United States.

The NCMA method will work but is somewhat limited as it does not properly address complex structures. Failure rate of MSE walls is estimated to be 4.6%, which is unacceptable for an engineered system. Failure of MSE walls investigated by the author *with respect to design issues* show that all used the NCMA method with fine grained soil quality, shorter reinforcement length, either no global stability or global stability not properly performed and active earth pressure coefficients ( $K_a$ ) less than the geotechnical engineers' recommendation.



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### **Design Software**

There are a number of software programs available for designing MSE, however in the US the programs MSEW and SrWall are considered to be the industry standard.

- |                               |               |
|-------------------------------|---------------|
| 1. MSEW (AASHTO, NCMA & LRFD) | 4. MESA-Pro   |
| 2. SRWall (NCMA)              | 5. MiraWall   |
| 3. AnchorWall                 | 6. RisiWall   |
| 4. ABWalls 2000               | 9. StrataWall |
| 5. KeyWall                    |               |

The author offers the following questions and answers that may be posed in regards to applicable design methodology (FHWA vs. NCMA) and computer programs that are available for MSE wall design. It is the authors' opinion that the MSEW program should be used for design.....

1. **Should program MSEW be used for design?**
  - o The MSEW software should be used in design as it addresses both the public sector (AASHTO/FHWA) and private sector (NCMA) design methods. It is always useful to compare designs, especially when the analysis is based on major assumptions (e.g., lateral earth pressures). Such assumptions may lead to paradoxical results which can be easily realized when comparing designs from two different methods. Also, the level of conservatism (or un-conservatism) can be assessed when comparing results. The MSEW program is a comprehensive tool that allows a designer to look at all the design aspects including global stability. It also allows the user to override values that do not correspond to certain MSE systems (it allows for adaptation of properties to specific block and reinforcement systems). Also, the FHWA-NHI document 2001 uses MSEW (3.0) as part of comparing hand calculations with computational analysis; it makes the computations in MSEW transparent.
2. **Should FHWA criteria be used for design since it can handle complex geometries, e.g. strip loads, back-to-back walls and tiered walls?**
  - The FHWA method deals with complex geometries, realistic loading, and more, which NCMA does not and cannot address. NCMA is based on simple closed-form equations that cannot deal straightforward with complex structures.

The author opines that wall designs should be based on FHWA criteria as many projects have complex geometries. MSE wall engineers can choose to use any design software of their choice; however careful decisions must be made as to the accuracy and ability of the wall program used in a final design.

The most conservative method would be "Modified Rankine" with the FHWA method being intermediate and NCMA being the least conservative. Listed below is a bit more detail regarding FHWA vs. NCMA design methods.



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### FHWA vs. NCMA

The differences between FHWA and NCMA are not that large when considering all reduction factors and safety factors used for design. Most experimental work shows FHWA to be conservative (but one needs to keep in mind that experimental results are for ideal conditions; e.g., firm foundation, very good construction; granular soils, no water, etc.). As a design engineer, the author recommends stability analyses for MSE walls using AASHTO/FHWA design criteria but this is a question/risk each designer must undertake. Simply, FHWA is an acceptable approach nationally and it considers **all** aspects of design. FHWA also allows for designs (external loading conditions and geometry) that are not available in NCMA.

MSE wall engineers must be able to interpret recommended earth pressure values provided within geotechnical exploration reports. Geotechnical recommendations are typically based on Rankine earth pressure theory.

- The FHWA design methodology uses Rankine earth pressure theory to calculate internal and external forces acting on the MSE wall system.
- The NCMA design methodology uses Coulomb earth pressure theory to calculate internal and external forces acting on the MSE wall system stability that takes into the account the wall batter. As wall batter increases, the Coulomb earth pressure values decreases. Wall designs using  $K_a$ -Coulomb is an acceptable approach as with the NCMA design methodology.

Designers should be aware that the NCMA method ignores the effects of the vertical force components due to Coulomb's inclined lateral earth pressure (inclined at delta,  $\delta$ ). Thus the NCMA method creates a 'half-pregnant' type of static analysis, i.e. they take the component of force they like and ignore others. This approach is not permissible when static (equilibrium) is used.

### **To illustrate different earth pressures between NCMA and FHWA consider the following example:**

Assume a MSE wall has a level toe, level backfill and a wall face batter of  $\Psi=7.1$ -degrees.

- The geotechnical engineer recommends an active earth pressure coefficient,  $K_a$ , for a silty-sand soil to be  $K_a=0.333$  (this corresponds to an effective internal friction angle of  $\phi=30$ -degrees).
- If the MSE wall engineer chose an effective internal friction angle of  $\phi=30$ -degrees and utilized the NCMA design method, the calculated design  $K_a$  value, based on a Coulomb earth pressure analysis would be equal to 0.249. This design  $K_a$  value indicates an earth pressure reduction approximately equal to 26%. Obviously this value is well below the recommended value provided for design by the geotechnical engineer.



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- The reduced earth pressure per NCMA criteria may result in some wall movement due to lower strength reinforcement and shorter reinforcement lengths, i.e. less reinforcement is required for lower  $K_a$  values.
- If global stability with respect to internal and compound internal stability using the reinforcement layout satisfying NCMA criteria is ignored “or” not properly analyzed it is likely to be found that factors of safety regarding internal and compound internal stability may not meet the minimum requirement of  $F_s > 1.30$ .
- If the design engineer chose an effective internal friction angle of  $\phi = 30$ -degrees and utilizes the FHWA design method, the calculated design  $K_a$  value, based on a Rankine earth pressure analysis would be equal to 0.333. This would meet the requirements and recommendations made by the geotechnical engineer.
  - If global stability with respect to internal and compound internal stability is properly analyzed using the reinforcement layout depicted by FHWA criteria it is likely to be found that factors of safety regarding internal and compound internal stability would meet or exceed the minimum requirement of  $F_s > 1.30$ .
- There are very few reported MSE wall failure or serviceability problems on State or Federal DOT projects. This is due to better quality granular backfill (crushed stone) and that the FHWA method is used for design.
  - The FHWA method is not the most conservative design method, but it is the most complete design method available for MSE wall design.
- A vast majority of wall failures have occurred when using a combination of:
  - the NCMA method
  - fine grained soils with more than 50% passing #200 sieve (silts and clays)
  - contractor design/build or material supplier produced designs with no design review on the owners behalf
  - inadequate quality control (QC) measure by the project owner
  - inadequate quality assurance (QA) measure by the wall contractor



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### III. Products and Soil Testing

#### Components of MSE Walls

- Geosynthetic Reinforcement
  - There are several brands of soil reinforcement products available for constructing MSE walls and slopes.
  - Geotextiles and geogrids commonly used in MSE walls are man-made products comprised of High Tenacity Polyester or High Density Polyethylene HDPE.
    - Author’s note: This course is focused on MSE walls with segmental blocks, however we do note that steel reinforcement is available but typically used on proprietary wall systems and not available to most designers.
- Masonry Block Facing Units
  - Blocks can be classified as having friction or mechanical connection capacity.
- Soil
  - Soil accounts for approximately 98% of the volume of MSE walls.
  - Soil is inexpensive and abundant construction material. The quality of the soil used in MSE system is critical.
  - The MSE wall engineer is responsible for providing soil specifications addressing soils for the structure that include the reinforced, retained and foundation zones.
    - The specification with respect to MSE wall soils must address required strength, unit weight, bearing capacity (foundation soil), classification, gradation and plasticity.
- Leveling Pad
- Drainage System

#### Geosynthetic Reinforcement Manufacturers and Type

Amoco	PP & PET Geotextiles
GeoStar	PET Geotextiles
Huesker	PET Geotextiles and Geogrids
LINQ	PP & PET Geotextiles
Lückenhaus	PET Geogrids & PET Geotextiles
Strata Systems	PET Geogrids
Synteen	PET Geogrids
Synthetic Industries	PET Geotextiles
TC Mirafi	PET Geogrids & PET Geotextiles
Tensar	HDPE and Polypropylene Geogrids

*\* Listed is a sample of reinforcement products typically used in MSE block walls and reinforced slopes. There are other products in addition to those listed.*



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### Example of Geosynthetic Reinforcement “Geotextiles”



### Example of Geosynthetic Reinforcement “Geogrid”



### Geosynthetic Reinforcement Data

In order to determine the geosynthetic-reinforcement allowable design strength the MSE wall engineer must.....

- Begin with the ultimate tensile strength of the reinforcement ( $T_{ult}$ ), which is the minimum average roll value (MARV) ultimate tensile strength per ASTM D4595
  - this value is adjusted by the Creep Reduction Factor,  $RF_{cr}$  (a minimum of one 10,000-hour creep tension test per ASTM D5262 is required to determine  $RF_{cr}$ )
  - along with the Durability Reduction Factor,  $RF_d$  (combined partial factor for potential chemical and biological degradation, default  $RF_d=2.0$  should be used if durability testing has not been conducted)
  - and Installation Damage Reduction Factor,  $RF_{id}$  (determined from construction damage tests for each reinforcement product based on ASTM D5818. Default  $RF_{id}=3.0$  shall be used if such testing has not been conducted with a minimum  $RF_{id}=1.10$ )
  - finally, apply a load reduction factor of  $FS_{UNC} = 1.5$

The end result is defined by the equations.....



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$$LTDS = \frac{T_{Ultimate}}{RF_{CR} \times RF_{ID} \times RF_D}$$

$$T_{Allowable} = \frac{LTDS}{FS_{UNC}}$$

*Geosynthetic-reinforcement ultimate strength along with reduction factors are available through reinforcement suppliers or are published in the annual Geosynthetics Specifiers Guide.*

**Components of MSE Walls - “Masonry Block Units”**

- Masonry block facing units - there are many brands of masonry block units.
- Block units are categorized as having a **mechanical** or **frictional** connection.

**Segmental Block Manufacturers**

**Mechanical Connection**

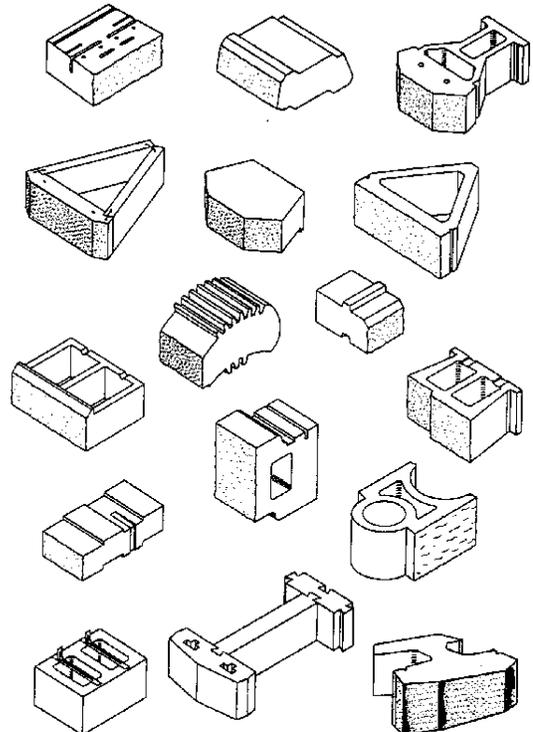
Cornerstone	MESA HP	Newcastle
-------------	---------	-----------

**Frictional Connection**

Allan Block	Amastone	Anchor	CMU	GeoStone
Lock & Load	Keystone	Risi Stone	Rockwood	Pisa
Select Stone	StoneGrid	Stonewall	Viking	Versa-Lok

**Examples of Commercially Available Segmental Units (NCMA, 1997)**

In design it does not matter whether the segmental block is mechanical or frictional, however the MSE wall design engineer must use correct connection capacity and block shear data for the specific segmental block and reinforcement combination in order to meet all factors of safety regarding facing stability, i.e. pullout and per FHWA or failure and serviceability per NCMA.



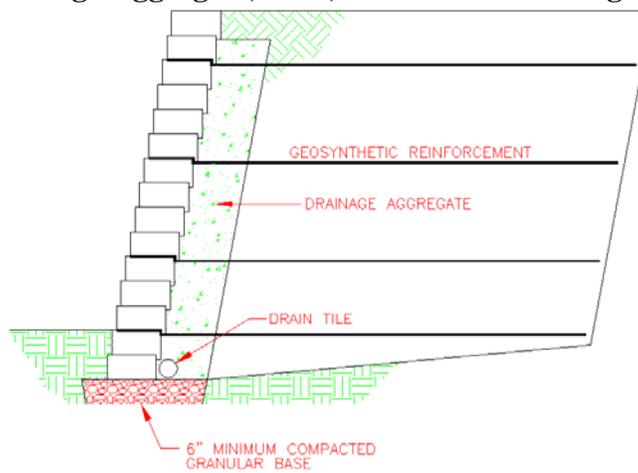


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### Components of MSE Walls

- Drainage system - drainage systems must be constructed to contain and/or control surface and subsurface water.
- Blanket drains are required when ground-water is close to the MSE wall foundation.
- Blanket and chimney drains are required when ground-water rises above the MSE wall foundation.

### Drainage Aggregate, Soils, and Gravel Leveling Pad



The owners' geotechnical engineer should provide information with respect to the location of the high groundwater table at the proposed MSE wall location to the MSE wall design engineer. If geotechnical data is not provided MSE wall engineers will typically assume (1) that the groundwater table elevation is deep enough such that seepage into the reinforced and retained backfill is minimal and foundation stability is not affected; and (2) the groundwater table is well below the leveling pad elevation, at a depth greater than or equal to  $0.66H$ , so as to not affect internal, external or global stability.

The author strongly recommends that a hydrological, geotechnical engineer or engineering geologist review the site prior to wall construction to determine the need for an internal drainage system at the location of each MSE wall, since analysis of hydrostatic loading is not within wall designer engineers scope of service.

If water is found to be present in the vicinity of the wall during excavation or construction, a proper functioning drainage system must be installed and sufficient drainage be provided such that hydrostatic loading (pore pressure) will not develop in the wall's reinforced zone. In the event owner or owner's geotechnical consultant determines during construction or by additional subsurface testing at the wall location prior to construction that the groundwater table is at a depth less than  $0.66H$ , then a blanket drain should be constructed at the base of the MSE wall as noted in the NCMA design manual (1997). Likewise, if the owner or owner's consultant



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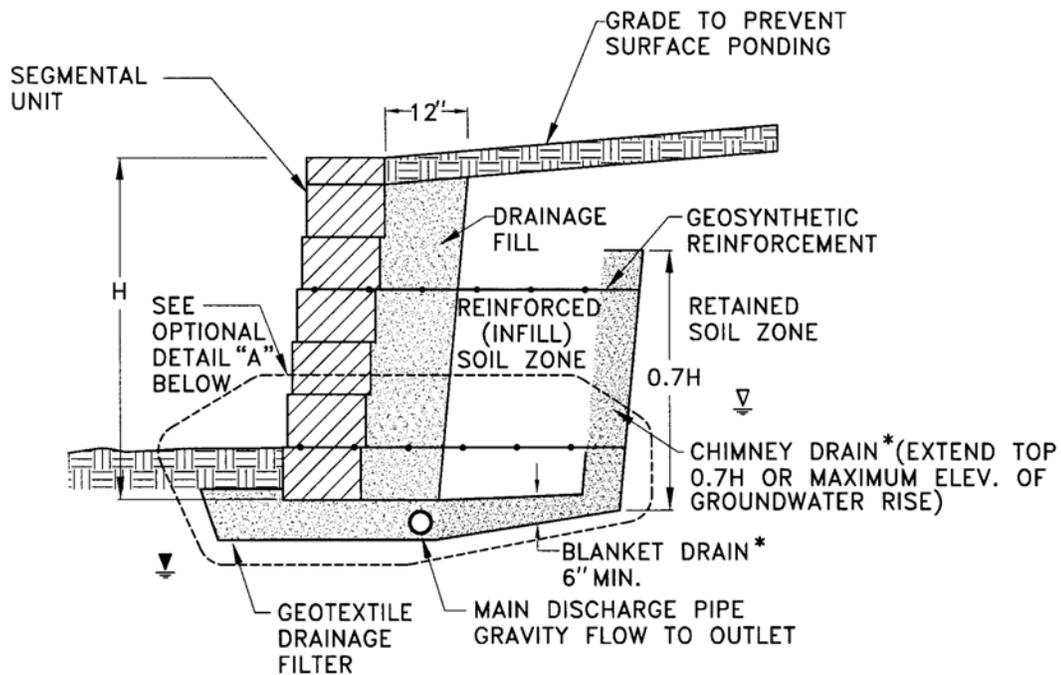
determines that the groundwater table is present within the retained soil zone, then a blanket drain and chimney drain should be constructed at the base and backside of the geosynthetic-reinforced zone of the MSE wall as noted in the NCMA design manual (1997). The drainage aggregate should be encapsulated within a geotextile filter fabric to minimize the migration of finer soil particles into the drainage gravel.

In the event surface or subsurface water diversion or drainage system details are required to prevent the infiltration of surface water into the MSE wall's reinforced fill zone, however, such engineering, design, analysis, detailing and mitigation shall be solely the responsibility of the Owner or Owner's consultant.

**Drainage Aggregate, Soils, Chimney Drain and Blanket Drain (NCMA, 1997)**

GROUNDWATER CONDITIONS FOR CASE 3

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





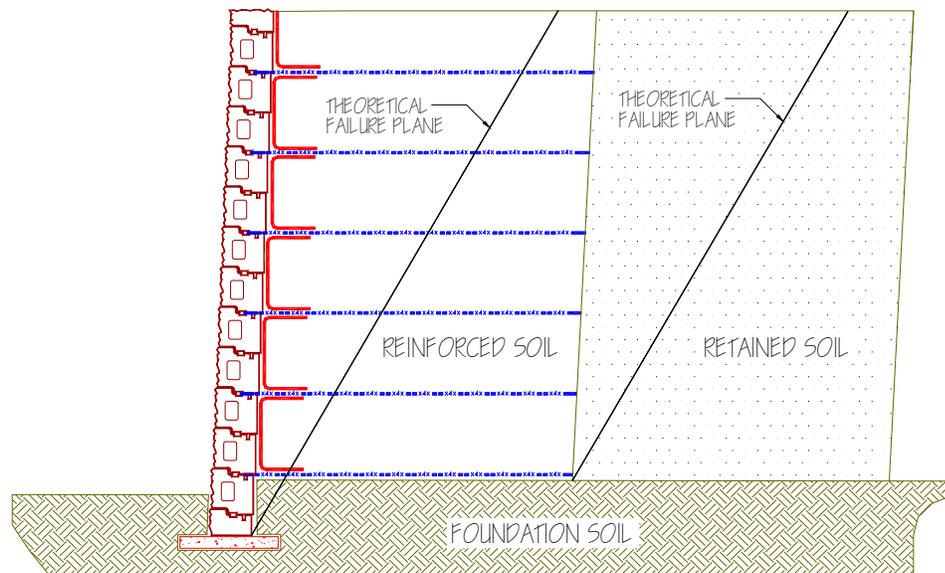
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#### IV. Design Information

##### Information Needed for a MSE Wall Design

- Soil Data Information
- Civil Drawings - Site Specific Information
- Geosynthetic Reinforcement Data
- Block Information
- Connection Strength Testing
- Unit to Unit Shear Testing
  
- **The Role of Soils**
  - Remember that 98% of a segmental retaining wall system consists of soil.
  - There is no other structural system where you assume 98% of the system and it will still work.
  - Therefore, accurate soil parameters specific to the construction site are essential to an accurate design.

##### Soil Zones



- **What do we need to know about these soil zones?**
  - Friction Angle ( $\phi$ ) - Triaxial or Direct Shear Test
  - Cohesion (c) - Triaxial or Direct Shear Test (ignored in reinforced and retained soil)
  - Unit Weight ( $\gamma$ ) - Proctor Test or Density Test



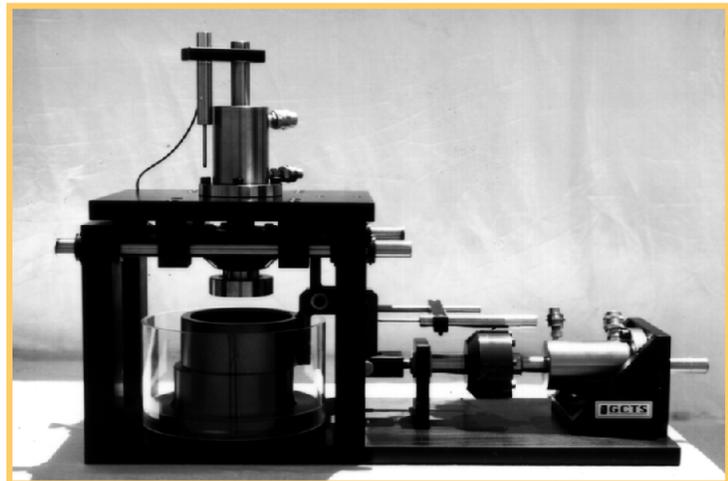
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### What do we need to know about these soil zones?

- In order to successfully design a segmental retaining wall, the MSE wall engineer needs to know or must be able to define the soils Effective Internal Friction Angle ( $\phi'$ ). This is a property of the soil type and defines the soil in terms of shear strength. The higher the angle, the “stronger” the soil in terms of resistance to sustained loads.
- The MSE wall engineer also needs to know or define a moist unit weight of the soil, known as gamma ( $\gamma$ ). This affects the driving and resisting forces.
- Another parameter of the soil shear strength is cohesion ( $c'$ ). This should also be determined. However, cohesion is ignored for the reinforced and retained soil zones, and is only used in the foundation soil zone.
- Geotechnical engineers describe soil shear strength using Mohr-Coulomb failure criteria.

### DIRECT SHEAR TEST PROCEDURES (REMOLDED SAMPLE)

A direct shear test can be performed on selected samples to determine shear strength parameters of cohesion, "c", and angle of internal friction, " $\phi$ ". Test specimens are typically prepared by tamping three equal layers of soil into a 2.5 inch diameter brass disc. The disc consists of an upper and lower portion, held intact with shear pins. Each soil layer consists of a measured specimen weight tamped to field density using a 2.49-inch diameter machined cylinder.



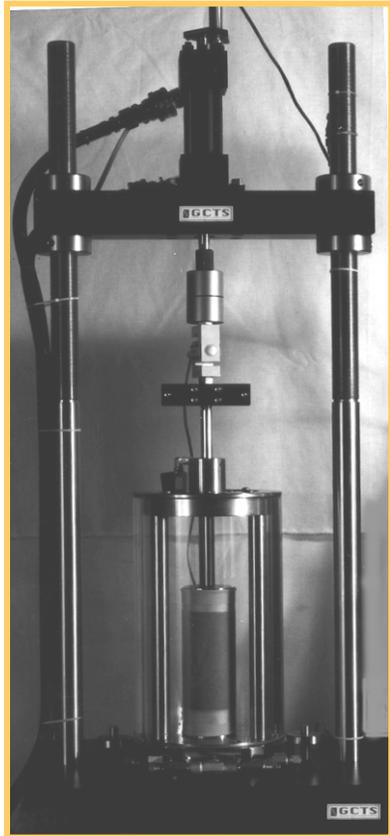
The brass discs are placed in the shear box with the lower disc fastened to the box. A normal load is applied to the specimen and the upper disc is raised (gapped) about  $\frac{1}{16}$  inch above the lower disc. Shear pins are removed, and a horizontal shear force is applied to the upper disc, shearing the soil across the gap in the discs. Proving ring readings of load and dial gauge readings of displacement are recorded.

Two other specimens are tested in the same manner with a different normal stress. Shear versus horizontal displacement curves are plotted for each test specimen and recorded on direct shear test sheets. A geotechnical engineer evaluates these curves, identifies an appropriate shear strength value, and plots the strength versus the applied normal stress. This plot provides the angle of internal friction, " $\phi$ " and cohesion, "c" that can then be used by the MSE wall engineer in the wall design calculations. The direct shear test is typically used for granular soils. Controlling drainage is very difficult if not impossible for fine grained soils, thus the test is not so suitable for drained conditions.



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### Triaxial Shear Test



- There are several Triaxial Tests
- The correct triaxial test needed for MSE wall design is the Consolidated Undrained (CU) Test with Pore Pressures.
- This test can take up to several weeks.
- This test is expensive.
- Therefore, plan your timetable wisely!

Consolidated, undrained triaxial shear tests with pore pressure measurements ("R" test) are performed on selected samples to determine the soils effective shear parameters of cohesion, "c", and angle of internal friction, " $\phi$ ". The samples can either be.....

- Remolded to the density and moisture content that would simulate field conditions.
- Undisturbed samples taken from Shelby tube that represent in-situ conditions.

Soil specimens are encased in a rubber membrane and placed in separate Plexiglas compression chamber. The specimen is sandwiched in the rubber membrane between porous stones, a Plexiglas pedestal and Plexiglas top that are connected via high-pressure tubing to an air over water pressure panel. Deaired tap water is added to the specimen's Plexiglas base pedestal and Plexiglas top cap. All the high pressure tubing lines and porous stones are then deaired by charging deaired water from burettes into the backpressure system. The specimens are allowed to seepage saturate under a controlled gradient and controlled effective confining pressure by applying a vacuum to the backpressure outflow line. Following seepage saturation, the specimen is backpressure saturated, then allowed to consolidate to the desired effective consolidation pressure. These consolidation pressures are selected in order to bracket the normal stress on shear planes anticipated by structural loadings.

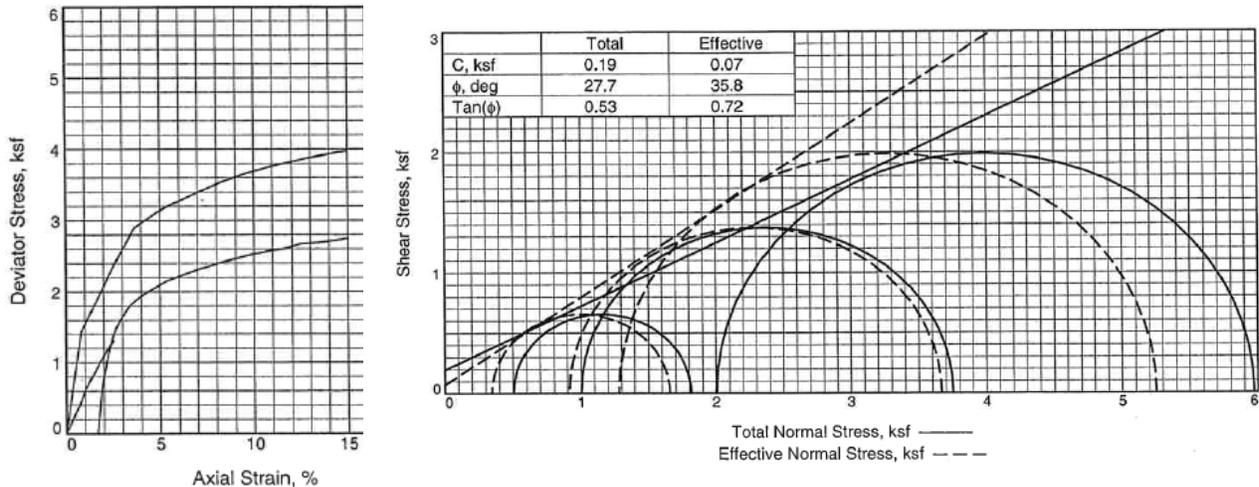
The specimen is sheared by a piston/platen mechanism at a constant rate of strain equal to at least twice the time of primary consolidation ( $2 \times t_{100}$ ). Load measurements are obtained with a proving ring and deformation readings obtained with a dial gauge accurate to 0.001-inch. The pore pressure inside the specimen is measured with an electric manometer connected to the base of the specimen. Volume change measurements are made throughout the testing process with burettes accurate to 0.1 cc. All measurements are taken to a strain level of 15 percent of the sample height.



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Mohr Diagrams are plotted by the geotechnical engineer that depict the soil shear parameters " $c$ " and " $\phi$ ". Also plotted are the stress-strain curves for each specimen as well as the effective and total stress paths (p-q diagrams).

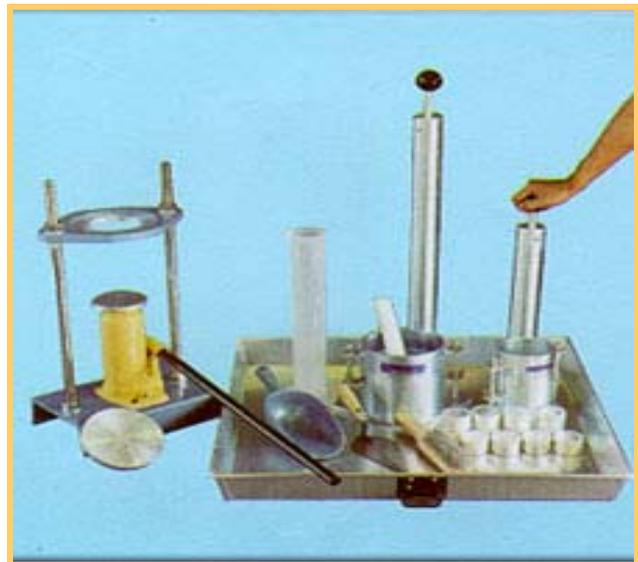
### “Stress vs. Strain” and Mohr Diagram from CU Triaxial Test



### STANDARD PROCTOR COMPACTION TEST PROCEDURES

A representative sample of proposed fill soil is obtained for a laboratory determination of density at various moisture contents. The test is used to predict the maximum density ( $\gamma_{dmax}$ ) to which the soil may be compacted in the field by conventional construction equipment.

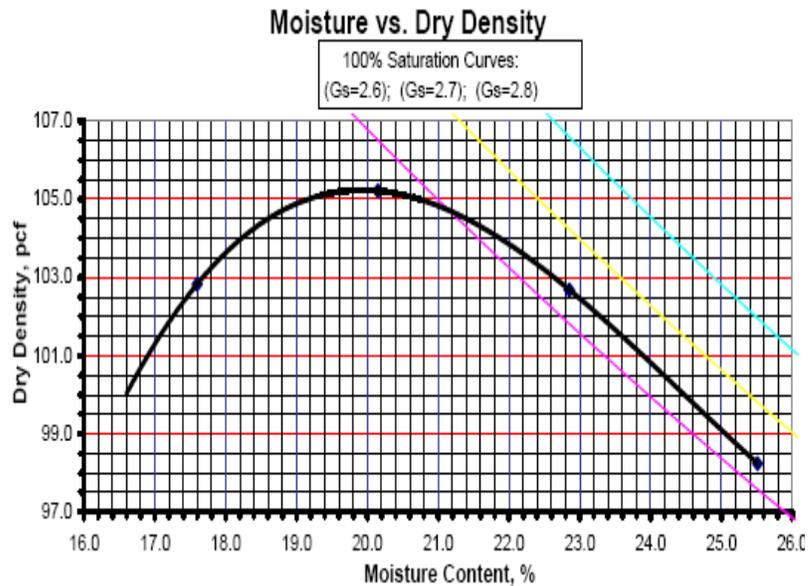
Each soil type is divided into four or more separate groups, air-dried, and each group is brought to a different moisture content. Each group is placed in a 4-inch diameter mold in 3 equal layers. Each layer is subjected to 25 blows of a 5.5-pound hammer falling 12 inches. The final compacted specimen has a known volume of  $1/30 \text{ ft}^3$ .





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After compaction, the dry unit weight ( $\gamma_d$ ) and moisture content ( $w_c$ ) of the sample are obtained. These data are plotted on a graph of moisture content (abscissa) versus dry density (ordinate), and a smooth curve is drawn connecting the data points. Results of the test, including test method used, origin of the sample, maximum dry density ( $\gamma_{dmax}$ ), optimum moisture content ( $w_{opt}$ ), and curve defining the moisture-density relationship are provided by the geotechnical engineer (see actual Proctor curve on next page that has  $\gamma_{dmax}=105.5\text{-lb/ft}^3$  and  $w_{opt}=20\%$ ). The recommended unit weight can be used by the MSE wall engineer in the wall design calculations.



Moist unit weight ( $\gamma_{moist}$ ) based on 95-percent compaction that would be used in the MSE wall design calculations is determined by:

$$\gamma_{moist} = \gamma_{dmax} \left(1 + \frac{w_{opt}}{100}\right) (\% \text{ compaction})$$

$$\gamma_{moist} = (105.5\text{-lb/ft}^3) \left(1 + \frac{20}{100}\right) (0.95)$$

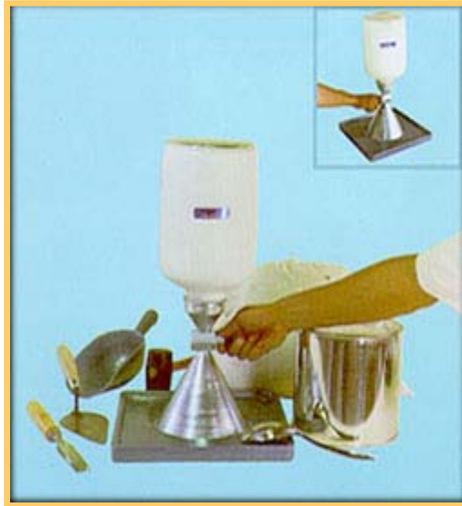
$$\gamma_{moist} = 120.3\text{-lb/ft}^3$$

In this case the recommended moist unit weight would likely be  $\gamma_{moist} = 120\text{-lb/ft}^3$ .

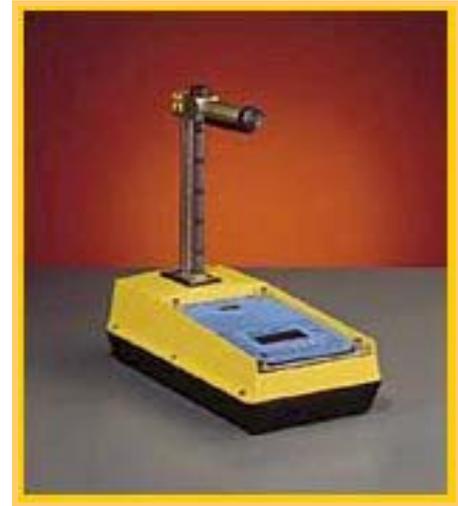
The author notes that the modified Proctor can also be specified. Local practice will likely dictate which Proctor method is used to determine the soils moisture-density relationship and unit weight.



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**Field Compaction  
Sand Cone, Drive  
Ring or Nuclear  
Density Gage  
examples of  
apparatus commonly  
used to determine in-  
situ density and  
moisture of on site  
soils.**



Compaction is the process of mechanically densifying a soil or aggregate. Densification is accomplished by pressing the particles into closer contact while expelling air from the soil mass. Compaction of a soil or aggregate will increase its density and shear strength and reduce its permeability. These changes are all desirable, and compaction is the simplest and most effective way to improve a soil's or an aggregate's engineering properties. Moisture content is defined as the ratio of the weight of water to the weight of solids in a given soil mass, usually expressed as a percentage. The amount of moisture in a soil mass affects the ability to compact the soil.

The moisture content of the soils used to construct the wall may vary considerably with weather conditions during construction. Drying or wetting of the soils may be necessary to achieve the recommended compaction criterion. If backfilling occurs during wet weather, these materials can likely not be dried sufficiently to obtain a satisfactory degree of compaction. As a practical consideration, these materials would generally be wasted and select materials trucked to the site. Any off-site materials should conform to the structural fill criteria discussed in the construction specifications.

In-place density testing should be performed as verification that the recommended compaction criterion has been achieved. In-place density testing for retaining walls is typically performed on a will-call basis, with a testing frequency of one test for every 100 to 150 linear feet of in-place fill. Tests are performed on at least 2-foot vertical increments. Areas failing to achieve the recommended compaction criteria are reworked and retested prior to proceeding with subsequent phases of construction.



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### **Discussion on Allowable Soil Backfill**

The success or failure of a MSE wall or slope is greatly dependent upon the soil used to construct the geosynthetic-reinforced zone and, to a lesser extent, the retained zone (soils located behind the reinforced zone). The selection of soil backfill with respect to the reinforced zone is extremely critical since about 98 percent of the structure is soil. Fine-grained soils such as SC, ML, CL, MH and CH can have a negative effect on the behavior of a wall or slope and therefore should not be used with any of the Mechanically Stabilized Earth (MSE) walls.

Fine-grained soils have a much greater potential for time-dependent movement (creep deformation) of the MSE wall system, leaving MSE walls and slopes more susceptible to failure if backfilled with fine-grained soils. The use of backfill with a large amount of fines is also a problem with all types of retaining walls. The lack of drainage, which may evolve over time, can eventually cause failure of any wall unless the wall is designed to retain water.

The NCMA method does allow for some fine grained soil types within the reinforced zone and there are MSE wall design engineers who will design with such soils (*although the author does not recommend the use of fine grained soils in the reinforced zone*). If a designer specifies the reinforced zone soils to classify as silt or clay a geotechnical engineer must be involved in the design to make sure the soil does not exhibit creep behavior.

FEA notes that the liquid limit, LL, and Plasticity Index, PI, can have a significant effect on the performance of a MSE wall. Soils used to construct the geosynthetic-reinforced zone must have a  $LL < 35$  and  $PI < 10$ . This is to assure that time dependent deformation will not be excessive and that backfill drainage will not be minimized.

Non-creeping soil types must be used for construction. Creep of MSE walls depends largely on the creep characteristics of the geosynthetic-reinforced soil. Field performance data have indicated that creep deformation of MSE walls is minimized when "well compacted granular fill" is used.

The creep rate of geosynthetics and soil are different. Where "non-creeping" soil is used (*e.g.*, granular with less than 35% passing the #200 sieve), the reinforcement will creep faster than the soil and thus the soil will serve to restrain creep of the reinforcement, causing it to relax (reducing the load in the reinforcement by increasing the load transfer into the soil along its common interfaces with the reinforcement).

By contrast, research performed at the University of Colorado, Denver, has shown that clayey backfill enhances creep of geosynthetics by creeping itself more than the geosynthetics. If a fine-grained silty or clayey soil (*e.g.*, more than 50% passing the #200 sieve) were to be used to construct the reinforced zone, it may creep faster than the reinforcement and transfer load to the geosynthetic, resulting in increased load and rate of creep in the geosynthetic, leading to possible failure. Fine-grained silty or clayey soil, including SC, ML, CL, MH and CH, should therefore not be used for construction of the MSE wall or slopes.



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**What is the ideal soil for the reinforced zone (and retained zone if fill) of a MSE wall?**

***SAND "or" CLAY***

Some design methods allow fine grained soil (silts and clay) in the reinforced zone as noted in the NCMA manual. However reinforced zone soil that classify as course grained (sand and gravel) is recommended because.....

- They are easier to place and compact.
- Have higher permeability which assists drainage.
- Have greater friction angle which reduces stresses.
- Are generally less susceptible to creep.
- *If you're going to use silt or clay...(FYI - the author does not recommend silt or clay)*
  - Make absolutely sure that proper drainage is installed.
  - Be sure that the soil has a low to moderate frost heave potential.
  - The internal cohesive shear strength parameter "c" is ignored.
  - Pay special attention to the creep potential (consult with geotechnical engineer)
  - Plasticity Index (PI) should never be greater than 20.

**Segmental Retaining Wall Block Information Needed For Design**

- Block Dimensions
  - Typical dimensions of segmental retaining wall block used in MSE wall construction are 8" height, 18" width and 12" depth.
- Block Setback
  - Most segment block systems have a built in face batter between near vertical and 10-degrees.
- Weight of the Individual Blocks
  - This can range from 70-lbs to 120-lbs depending on the block shape and volume of the core.
  - Infilled weight of the blocks that includes #57-stone with in the block core and between adjacent units typically measures to be about 120-pcf.
  - Segmental block units should have a minimum 28-day compressive strength of 3,000-psi on the net area and have a maximum absorption rate of 8.0 percent.



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- Block/Grid Connection Strength
  - SRWU-1 Determination of Connection Strength between Geosynthetics and Segmental Concrete Units
- Unit to Unit Shear
  - SRWU-2 Determination of Shear Strength between Segmental Concrete Units

**The MSE wall engineer must choose a specific segmental block facing unit and reinforcement combination. Why does the MSE wall engineer need to know the specific block and grid combination used?**

**...because each unit has unique unit-to-unit shear-strength properties as well as unique connection properties with each individual reinforcement type.**

### **Block/Reinforcement Connection Strength Test**

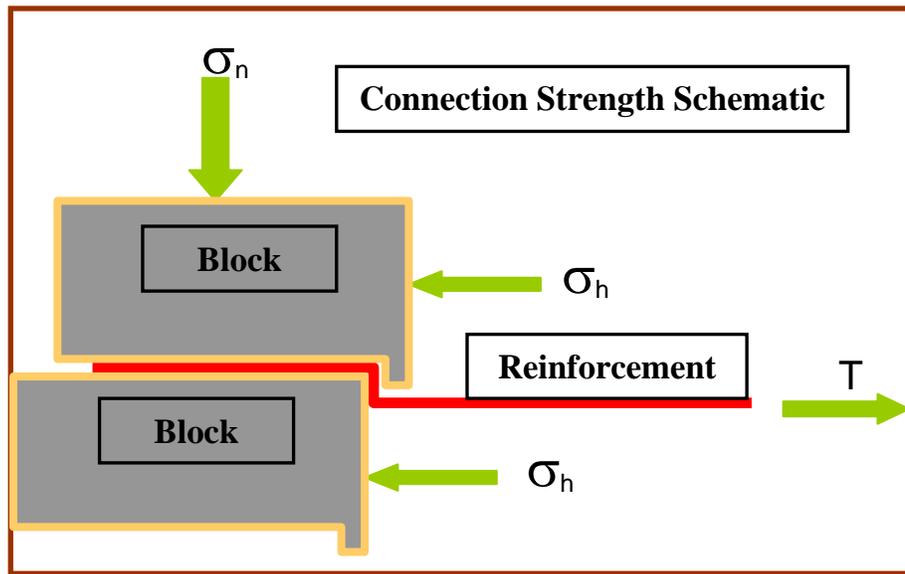
In a connection capacity strength test several normal loads ( $\sigma_n$ ) are applied to two segmental block units with a layer of geogrid or geotextile between the blocks. A horizontal load ( $\sigma_h$ ) is used to restrain the blocks while a tensile load (T) of up to 35,000 pounds is applied to the reinforcement.

Connection capacity tests are run until failure of the connection occurs, which could be pullout of the reinforcement between the blocks (typical of frictional connections) or rupture of reinforcement or locking bar (typical of a mechanical connection).





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The connection capacity curve is developed by applying several normal and tension loads to a specific block and grid combination. Various normal loads are used to simulate different wall heights that are reasonable for the reinforcement long term design strength (LTDS). Example - if  $K_a=0.333$  (using  $\phi=30$ -deg), soil unit weight is  $\gamma=120$ -pcf and assume maximum vertical spacing between reinforcement layers of  $S_v=2.0$ -ft then the maximum design test height  $H_{max}$  is:

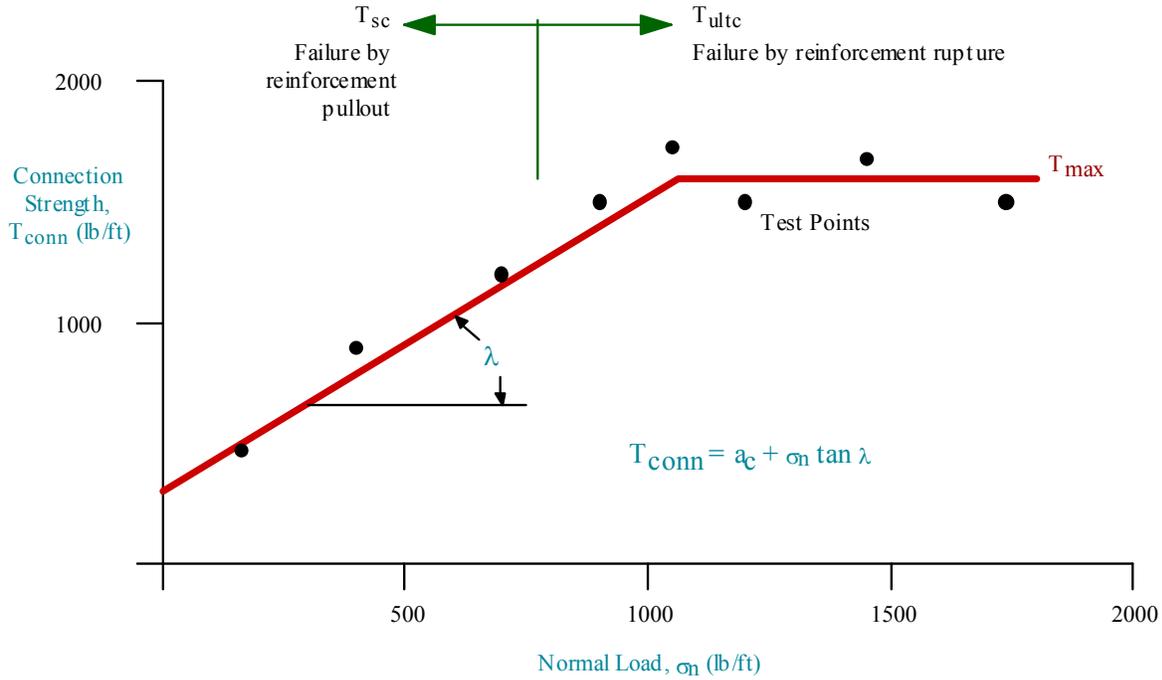
$$H_{max} \text{ (ft)} = \frac{LTDS}{K_a \gamma S_v 1.5} = \frac{LTDS}{0.333 (120 \text{ pcf}) (2.0\text{ft}) 1.5} = \frac{LTDS}{120}$$

- If  $LTDS=2,156$  lb/ft, calculated max wall height = 17.97 ft (would likely test up to 20-ft)
- If  $LTDS=3,035$  lb/ft, calculated max wall height = 25.29 ft (would likely test up to 26-ft)

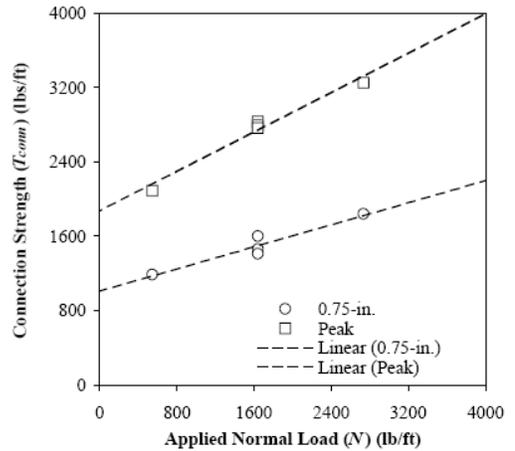
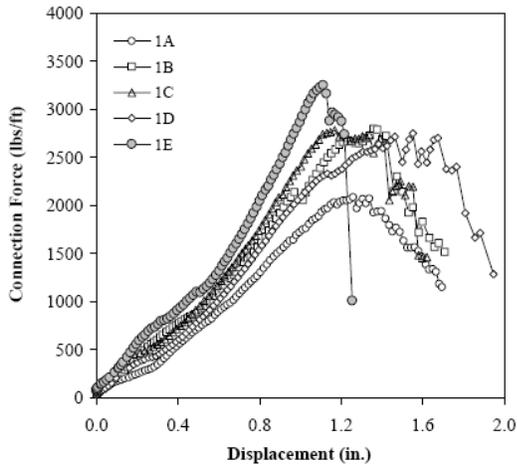


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**Typical Connection Capacity Curve**



Test showing load displacement and connection strength curves for mechanical block system.



Test No.	Test Specimen Width (in.)	Test Normal Stress (psi)	Equivalent Normal Load (lb/ft)	Approx. No. of Blocks	Approx. Height (ft)	0.75-in. Strength (lb/ft)	Peak Strength (lb/ft)	Connection Strength Equations ( $T_{conn}$ )
1A	32.0	3.8	548	5	5.0	1188	2087	$T_{0.75-in.} = 1010 + (N) \tan (17^\circ)$
1B	32.0	11.4	1643	15	15.0	1458	2828	
1C	32.0	11.4	1643	15	15.0	1600	2792	$T_{peak} = 1870 + (N) \tan (28^\circ)$
1D	32.0	11.4	1643	15	15.0	1407	2762	
1E	32.0	19.0	2738	25	25.0	1838	3247	

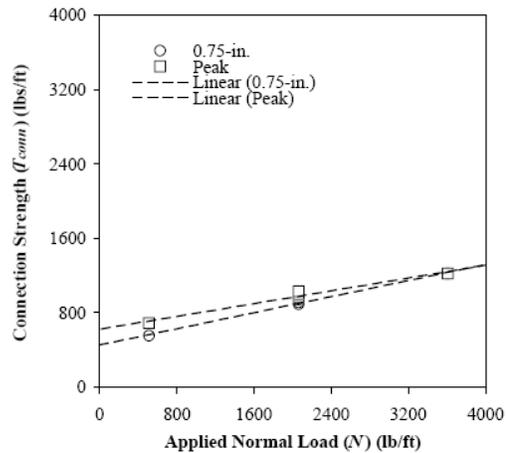
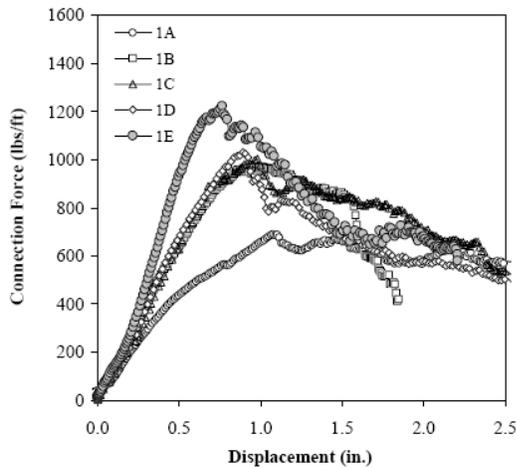


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Load displacement curves show an increase in connection force up to about 1.1 to 1.5-inch and then a sudden loss of connection, which is typical for a mechanical block system. In this particular test.....

- Geotextile failure - abrasion and rupture of the geotextile on top of the mechanical connector.
- Mechanical connector failure - buckling of connector bar.

Test showing load displacement and connection strength curves for frictional block system.



Test No.	Test Specimen Width (in.)	Test Normal Stress (psi)	Equivalent Normal Load (lb/ft)	Approx. No. of Blocks	Approx. Height (ft)	0.75-in. Strength (lb/ft)	Peak Strength (lb/ft)	Connection Strength Equations ( $T_{conn}$ )
1A	31.0	3.7	516	8	5.0	553	689	$T_{0.75-in.} = 460 + (N) \tan ( 12^\circ )$ $T_{peak} = 630 + (N) \tan ( 10^\circ )$
1B	31.0	14.9	2063	33	20.0	889	970	
1C	31.0	14.9	2063	33	20.0	911	1003	
1D	31.0	14.9	2063	33	20.0	942	1030	
1E	31.0	26.2	3610	58	35.0	1222	1222	

Load displacement curves show an increase in connection force up to about 0.75-inch and then a gradual loss of connection, which is typical for a frictional connection. In this particular test the.....

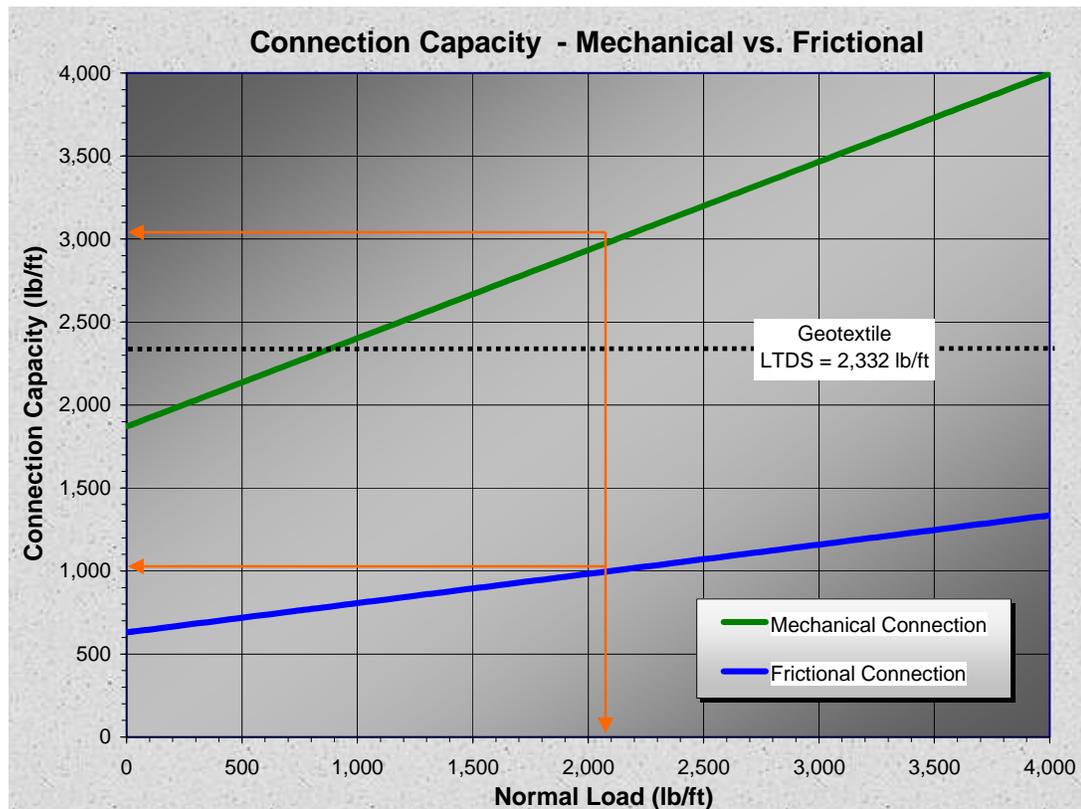
- Geotextile failure - abrasion and rupture of the geotextile between the block units.

Reinforcement in the two connection tests was a 4,800-lb/ft high strength geotextile product. A couple observations can be made between two tests (see overlay of both connection curves in the figure below.

- The frictional connection failed due to abrasion, rupture and pullout of the geotextile at a low connection load with  $T_{peak} = 630 + N \tan 10^\circ$ .
- The mechanical connection failed due to abrasion/rupture of the geotextile and buckling of connector bar at a high connection load,  $T_{peak} = 1870 + N \tan 28^\circ$ .



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If we directly compare the connection capacity between the mechanical and frictional block at a normal load (N) of 2,089-lb/ft we get the following result. Maximum connection load is limited by hinge height as discussed in more detail below. For the sample below 2,089-lb/ft represents a one foot deep by one foot high block battered at 3-degrees that weighs 109.5-lb/ft<sup>3</sup> block.

- $T_{\text{conn(frictional)}} = 630 + N \tan 10^\circ = 630 + 2,089\text{-lb/ft} (\tan 10^\circ) = 998\text{-lb/ft}$
- $T_{\text{conn(mechanical)}} = 1,870 + N \tan 28^\circ = 1870 + 2,089\text{-lb/ft} (\tan 28^\circ) = 2,981\text{-lb/ft}$

In this comparison the mechanical system has a connection capacity almost three times greater than the frictional block. Both mechanical and frictional block systems can be designed with success however it is critical that the proper connection data specific to the block/reinforcement combination is used in design calculations.

Many SRW walls are constructed with a front batter, the column weight above the base of the wall or above any other interface may not correspond to the weight of the facing units above the reference elevation. This is known as the hinge height ( $H_h$ ) concept.

The Hinge Height calculation is not a significant design consideration in vertical or near vertical wall structures. However it becomes a serious design limitation in heavily battered structures with small facing elements where sliding resistance and geosynthetic reinforcement connection capacity are reduced to levels below peak laboratory tested values.



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A graphical representation of the Hinge Height concept and its application is shown below.

$$H_{\text{hinge}} = \frac{2(W_u - G_u - 0.5 \cdot H_u \cdot \text{Tan}i_b) \cos i_b}{\tan(\omega + i_b)}$$

$H_{\text{hinge}}$  = Hinge Height

$W_u$  = Depth of segmental block unit (feet)

$G_u$  = Segmental block unit center of gravity  
=  $W_u/2$  (feet)

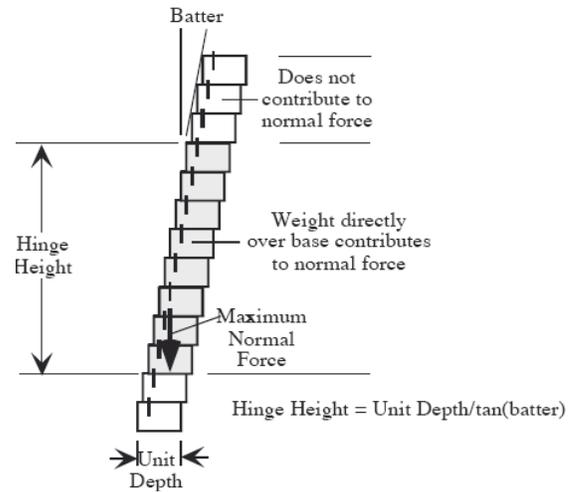
$H_u$  = Height of segmental block unit (feet)

$\omega$  = Wall face batter (degrees)

$i_b$  = Foundation block angle (typically  $0^\circ$ )

$\gamma_u$  = Segmental block unit (lb/ft<sup>3</sup>)

$H_{\text{load}}$  = Hinge height load (lb/ft) =  $\gamma_u H_{\text{hinge}}$



Calculate hinge height and maximum connection load for a block with the following properties:

$$W_u = 1.0\text{-ft} \quad H_u = 1.0\text{-ft} \quad \omega = 3\text{-deg} \quad i_b = 0\text{-deg} \quad \gamma_u = 109.5\text{-lb/ft}^3$$

$$H_{\text{hinge}} = \frac{2[1.0 \cdot \text{ft} - 0.5 \cdot \text{ft} - 0.5 \cdot 1.0 \cdot \text{ft} \cdot \text{Tan}(0)] \cos(0)}{\tan(3 + 0)} = 19.081\text{-ft}$$

$$H_{\text{load}} = (109.5\text{-lb/ft}^3) (19.081\text{-ft}) = 2,089\text{-lb/ft}$$

In design the maximum load at the connection would be governed by the hinge height. If the wall was 40-ft tall the maximum connection load would be 2,089-lb/ft.

Vertical spacing between reinforcement layers in a MSE wall should never exceed  $S_v=2.0\text{-ft}$ . A number of costly failures have occurred when large vertical spacing was used with big blocks or high strength reinforcement. Spacing controls performance not the block size or strength of the reinforcement. Segmental walls that use close vertical spacing of 8-inch or 16-inch are generally referred to as Geosynthetically Reinforced Soil (GRS) or Geosynthetically Confined Soil (GCS) but are beyond the scope of this course.



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