DESIGN MANUAL FOR

SEGMENTAL RETAINING WALLS

SECOND EDITION

NATIONAL CONCRETE MASONRY ASSOCIATION

2302 Horse Pen Road
Herndon, Virginia 20171-3499
(703) 713-1900

SECTION 1
INTRODUCTION

This design manual was produced by the National Concrete Masonry Association (NCMA) to provide a standardized engineering approach for the analysis and design of segmental retaining walls (SRWs). A segmental retaining wall is constructed from dry-stacked units (without mortar) that are usually connected through concrete shear keys or mechanical connectors. SRW units may be dry-cast machine produced or wet-cast concrete products. A variety of proprietary SRW units are available from NCMA members.

The purpose of this manual is to provide generic design guidelines for segmental retaining walls constructed as gravity structures and as geosynthetically reinforced soil segmental retaining walls. Reinforced SRW systems are composite facing-mechanically stabilized earth (MSE) retaining wall structures that have unique features and design requirements that are not addressed in previous MSE design guidelines. This manual provides design guidelines and engineering procedures necessary to produce safe, cost effective retaining wall structures. The design methodologies contained in this manual allow the engineer to analyze the influence of all components of the SRW system on wall performance. The theories presented in this manual offer the designer the possibility to quantify performance of retaining wall structures built with segmental concrete facing units, geosynthetic reinforcement and soils.

The design concepts presented in this manual are based on conventional engineering principles and experience with the design and construction of a large number of SRW structures in North America over the past 10 years [Refs. 33-38, 44, 49]. This manual assumes that the designer is familiar with basic soil mechanics and principles of engineering mechanics. The analysis and design of a gravity or reinforced SRW can be carried out in a systematic step-by-step process using the methodologies presented herein. The computations associated with each step in the design and analysis procedures in this manual lend themselves to implementation within user-friendly computer software.

This manual also provides design standards and engineering guidance to assist the designer in selecting appropriate performance criteria for a project. The manual also identifies key components of a SRW system and their functions, as well as important construction techniques for satisfactory performance in the field. A set of generic specifications are provided to assist the architect/engineer to develop accurate and complete project-specific contract documents.

The design methodologies and other recommendations contained in this manual are comprehensive and provide a common basis for the development of design charts and specifications by manufacturers and suppliers of products used in the construction of SRW systems. The benefit of the single "consensus" design approach presented in this manual is greater understanding, confidence and acceptance of SRW systems by design and construction professionals.

SECTION 2
SEGMENTAL RETAINING WALLS

Segmental retaining walls are divided into two groups of gravity retaining walls.

> **Conventional SRWs** are structures that resist external destabilizing forces, due to the retained soils, solely through the self-weight and batter of the SRW units (**Figure 2-1A**).
>
> **Reinforced Soil** SRWs are composite systems consisting of SRW units in combination with a mass of reinforced soil stabilized by horizontal layers of geosynthetic reinforcement materials. (**Figure 2-1B**).

Common to both types of structures are dry-stacked segmental units that are constructed in a running bond configuration as shown in **Figure 2-1**. The majority of SRW units available on the market are dry-cast, machine produced concrete without internal reinforcement.

Conventional SRWs are separated into single depth and multiple depth categories (**Figure 2-1A**). The maximum wall height that can be constructed using a single depth unit is directly proportional to its weight, width, and vertical batter of construction for any given soil and site geometry conditions. The maximum height can be increased by implementing a conventional crib wall approach using multiple depth units to increase weight and width of the wall.

Reinforced soil SRWs utilize geosynthetic or metallic reinforcement to enlarge the effective width and weight of the gravity mass. Geosynthetic reinforcement materials are high tensile strength polymeric sheet materials. Geosynthetic reinforcement products may be geogrids or geotextiles, though most SRW construction to date has used geogrids. The geosynthetic reinforcement extends through the interface between the SRW units and into the soil to create a composite gravity mass structure. This enlarged composite gravity wall system, comprised of the SRW units and a reinforced soil mass, offers the required resistance to external forces associated with taller walls, surcharged structures or more difficult soil conditions.

Reinforced soil SRWs may also be referred to as MSE walls. The term mechanically stabilized earth (MSE) is often used to describe all forms of fill type reinforced soil structures [Refs. 1, 2, 3, 4, 5, 7].

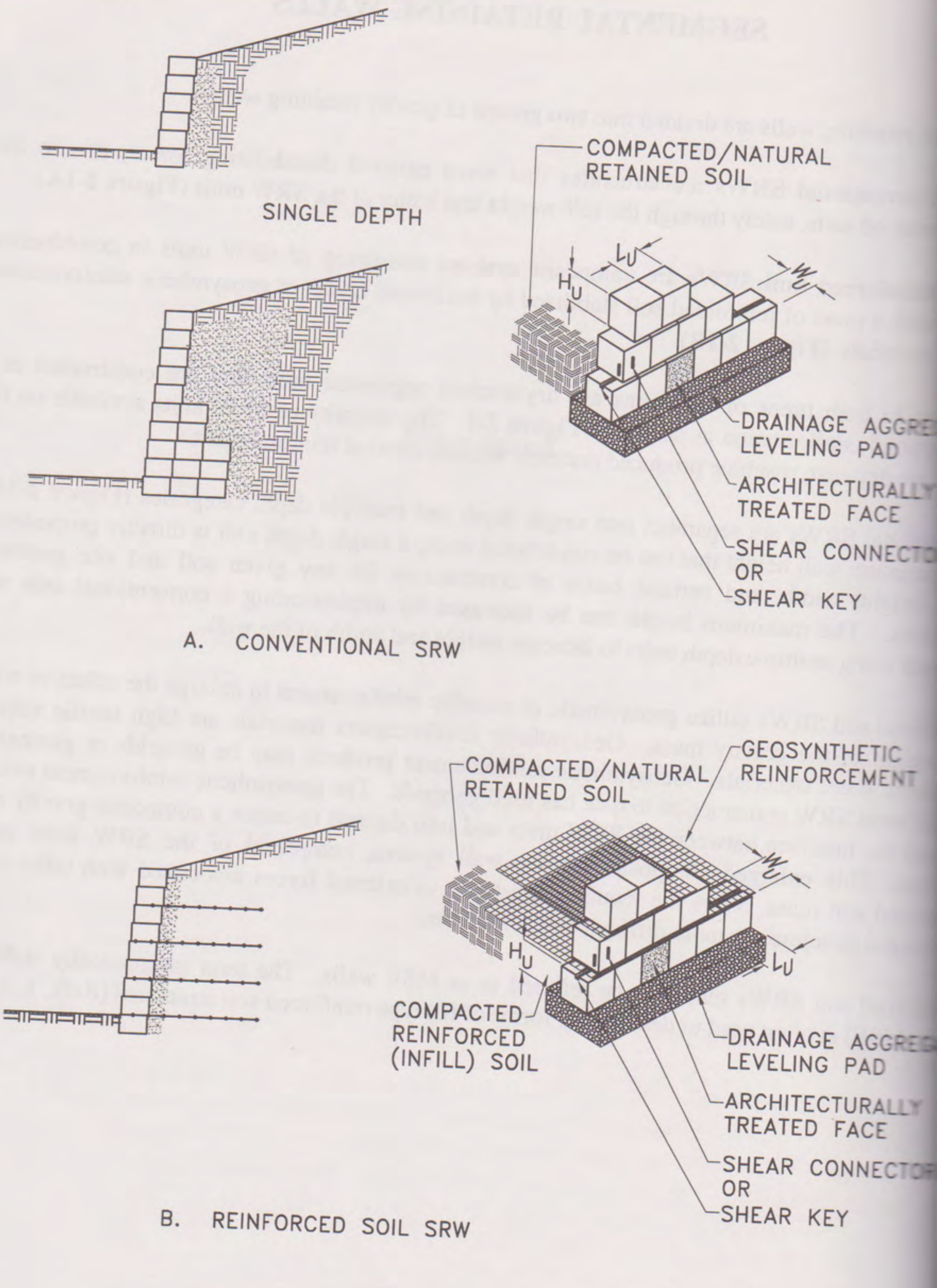


FIGURE 2–1: SEGMENTAL RETAINING WALL SYSTEMS

Section 2.1
ADVANTAGES OF SEGMENTAL RETAINING WALLS

SRWs offer several important advantages over other soil retaining wall systems.

- **Durability:** SRW units are manufactured from dry-cast machine formed or wet-cast concrete which creates durable and long-lasting retaining wall systems. SRW units are resistant to cracking, and do not splinter or decay like treated timbers or railroad ties. Segmental units utilize high compressive strength and low absorption concrete to resist spalling and freeze/thaw effects which has made conventional masonry blocks a durable building product for foundation and retaining walls for many years

  Additionally, the geosynthetic reinforcement used to stabilize the soil behind SRW units is manufactured from specially formulated polymers engineered to resist creep and environmental degradation throughout the design life of the structure. By implementing the design procedures outlined herein and provided in other references, a safe working strength for the geosynthetic reinforcement can be determined for a design life ranging 75 to 100 years.

- **Ecologically Friendly:** Concrete SRW units are an ecologically safe building material. SRW units are manufactured from naturally occurring materials that are in abundant supply and extracted with minimal environmental impact. Chemical preservatives (creosote, nickel/arsenic) used in treated timbers and railroad ties are not required for SRW units.

- **Aesthetics:** SRW units come in a variety of colors, shapes, styles and configurations and therefore provide architects, owners and engineers with a wide choice of aesthetically pleasing wall textures and appearances. The ability to match and complement other building materials such as brick, masonry, and stucco allows SRW units to visually enhance a project. The use of split-face SRW units provides an architectural appearance which is superior to the gray, drab, monotony of conventional cast-in-place concrete retaining walls.

- **Performance:** SRWs are relatively flexible structures which are typically founded on flexible aggregate leveling pad foundations. SRWs are dry-stacked systems that can tolerate movement and settlement without causing visual distress at the face since the SRW units may move and adjust relative to each other. This contrasts with more rigid retaining structures such as cast-in-place concrete and conventional mortared masonry walls.

  The dry-stack construction method used in SRW construction permits water to readily drain through the wall face. This "draining" action, along with the wall face underdrain, aids in preventing the development of hydrostatic pressure behind the SRW units.

- **Design Flexibility:** SRW units offer the site designer flexibility with respect to retaining wall location, wall layout, height, and constructability. The typical small size and weight of SRW units permits the construction of walls in difficult access locations. SRW units offer the designer the ability to incorporate tight curves or corners at a site and satisfy other complex architectural layouts.

- **Ease and Rate of Installation:** SRW units have been designed for rapid and easy installation. Many SRW units on the market can be placed by a single construction worker without the aid of construction equipment. The mortarless construction allows installation to proceed quickly. An experienced installation crew of three or four typically erect 200-400 square feet of wall per day. This minimizes the impact of retaining wall construction on project scheduling and shortens overall construction time.

- **Economics:** SRWs offer one of the best values in earth retaining wall systems. SRWs are a cost-effective, aesthetically pleasing earth retaining structures that typically offer a twenty-five percent to forty percent advantage over conventional cast-in-place concrete retaining walls. Furthermore, SRWs may offer a capital cost advantage over other proprietary retaining wall systems. When all project costs are measured and compared against performance benefits, SRWs are often the best value for construction dollars spent.

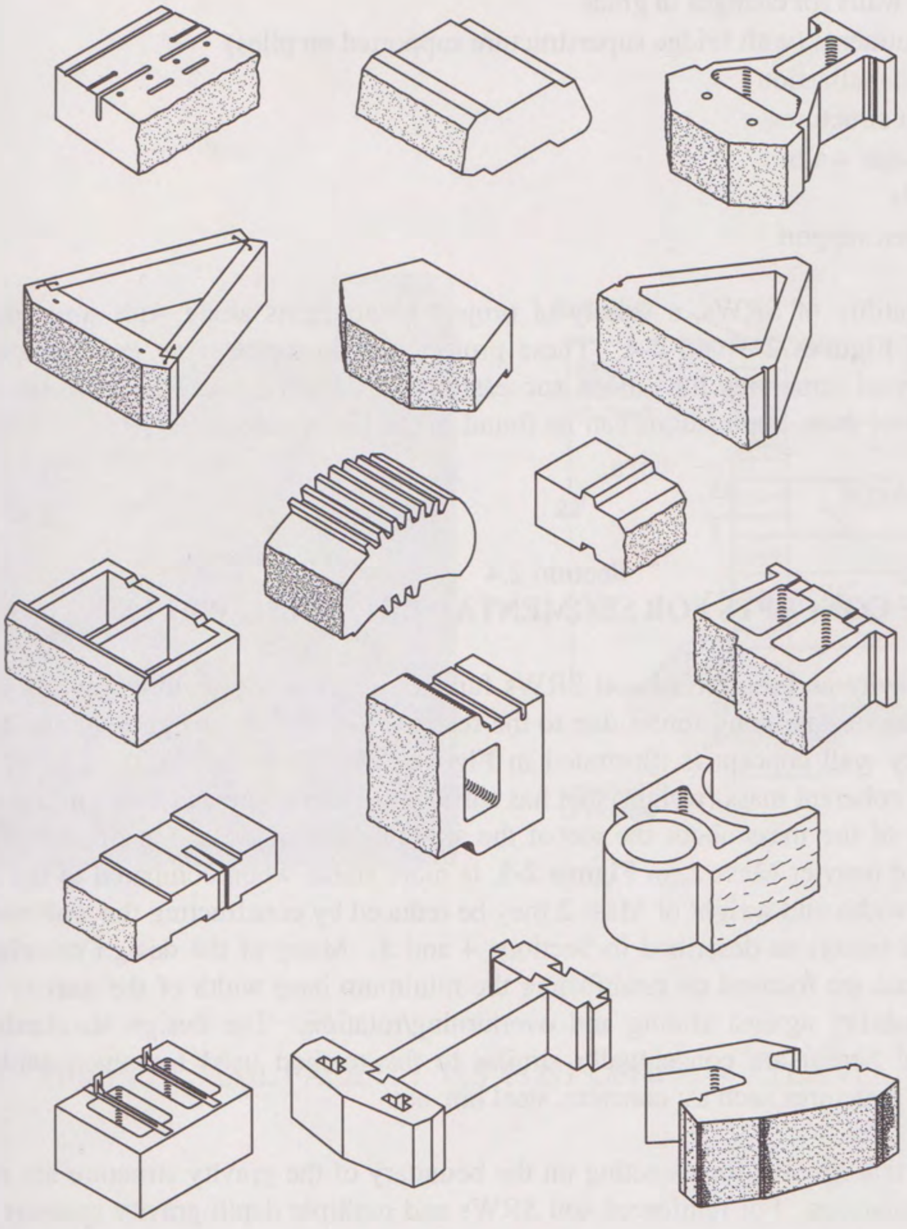
Section 2.2
PAST AND PRESENT SEGMENTAL RETAINING WALL SYSTEMS

The concept of segmental walls and reinforced soil is surprisingly old. The Ziggurats of Babylonia (i.e., Tower of Babel) were built some 2,500 to 3,000 years ago using soil reinforcing methods very similar to those described in this manual for reinforced soil SRWs [Refs. 1, 4]. The Great Wall of China was built using an early version of a MSE system, consisting of a mixture of clay and gravel reinforced with tamarisk branches [Refs. 1, 4]. Therefore, the currently available systems using concrete SRW units and geosynthetic reinforcement can be viewed as an advancement in the type of material utilized in this time-proven construction technique.

Interlocking concrete units were first introduced in the 1960's as concrete crib retaining wall systems. Modern SRW units are an advancement that provide an architecturally acceptable concrete facing system that can be machine made or cast without the need for internal steel reinforcement. Significant use of SRW units for conventional structures began approximately in 1984. Around 1986 the use of reinforced soil SRW systems began. Since that time, the number of conventional and reinforced soil SRWs installed has grown rapidly with over 100,000 completed wall installations in North America as of 1996, twenty-five percent of those being reinforced soil SRWs.

Section 2.3
CURRENT SEGMENTAL RETAINING WALL SYSTEMS AND APPLICATIONS

A variety of the proprietary segmental units are available (see **Figure 2-2**). The units shown on the figure are a sample of those available and serve to illustrate the variety in size, shape and interlocking mechanism. There are no restrictions on the size and shape of a SRW unit but most proprietary units are 3 to 24 inches in height ($\boldsymbol{H_u}$), 6 to 30 inches in width ($\boldsymbol{W_u}$) and 6 to 72 inches in length ($\boldsymbol{L_u}$). (See **Figure 2-1** for definition of $\boldsymbol{H_u}$, $\boldsymbol{W_u}$, $\boldsymbol{L_u}$).



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

FIGURE 2-2: EXAMPLES OF COMMERCIALLY AVAILABLE SEGMENTAL UNITS

Segmental retaining walls can be used in a number of applications including:

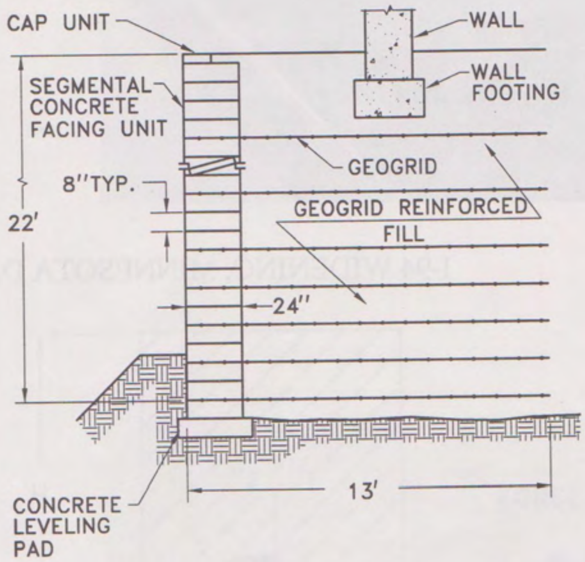
- Landscaping walls
- Structural walls for changes in grade
- Bridge abutments (with bridge superstructure supported on piles)
- Stream channelization
- Waterfront structures
- Tunnel access walls
- Wing walls
- Parking area support

To illustrate the versatility of SRWs, a variety of project photographs along with some project details are shown on **Figures 2-3** and **2-4**. These project case histories serve to illustrate that significant retaining wall structures have been successfully constructed using SRW systems. A description of several of these applications can be found in the list of references [Refs. 33-38, 44, 49].

Section 2.4
BASIC CONCEPTS FOR SEGMENTAL RETAINING WALLS

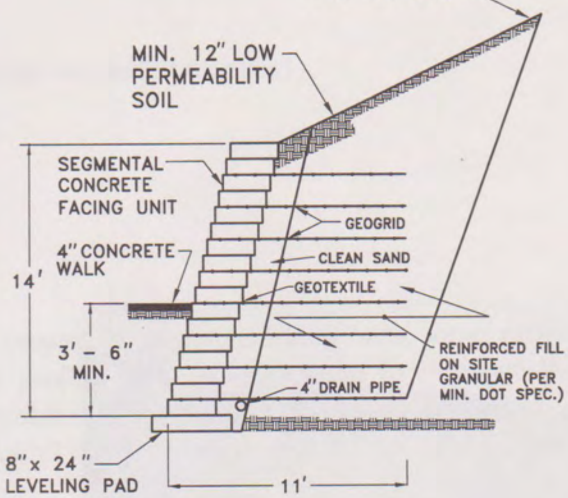
Both conventional gravity and reinforced soil SRWs function as gravity structures by relying on self-weight to resist the destabilizing forces due to the retained soil and surcharge loadings on the structure. The gravity wall concept is illustrated in **Figure 2-5**. To be stable, the gravity wall structure must form a coherent mass (weight) that has sufficient width to prevent both sliding at the base and overturning of the mass about the toe of the structure under the action of lateral earth forces. The wider and heavier Mass 2, in **Figure 2-5**, is more stable when compared to the more slender Mass 1. The width and weight of Mass 2 may be reduced by constructing the wall with an inclined face (vertical batter) as described in Sections 4 and 5. Many of the design calculations described in the manual are focused on establishing the minimum base width of the gravity mass required to ensure stability against sliding and overturning/rotation. The design standards and calculations presented herein are conceptually similar to the method used for any established gravity retaining wall structures such as: concrete, steel bin, etc.

Stability calculations that involve forces acting on the boundary of the gravity structure are called external stability calculations. For reinforced soil SRWs and multiple depth gravity systems a set of internal stability calculations are also required to ensure there is adequate strength and width to create a stable monolithic gravity mass. Finally, for both conventional gravity and reinforced soil SRWs the local stability of the dry-stacked column of SRW units should be analyzed. The site civil or geotechnical engineer should evaluate the SRW structure for possible excessive foundation settlement and potential global/overall slope instability.

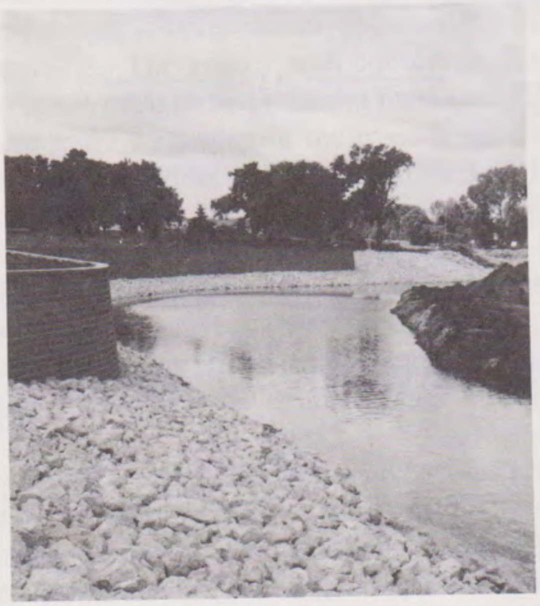


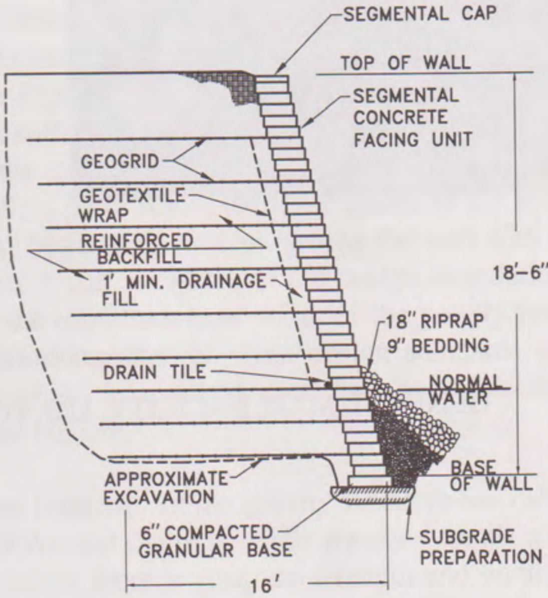
GENERAL MAIL FACILITY, U.S. POST OFFICE — ATLANTA, GA [Ref. 35]

FIGURE 2-3: COMPLETED PROJECT: U.S. POST OFFICE



I-94 WIDENING, MINNESOTA D.O.T. — ST. PAUL MN. [Ref. 64]





STREAM CHANNELIZATION, CORPS OF ENGINEERS —ROCHESTER,MN

FIGURE 2-4: COMPLETED PROJECTS: I-94 WIDENING AND STREAM CHANNELIZATION

UNSTABLE

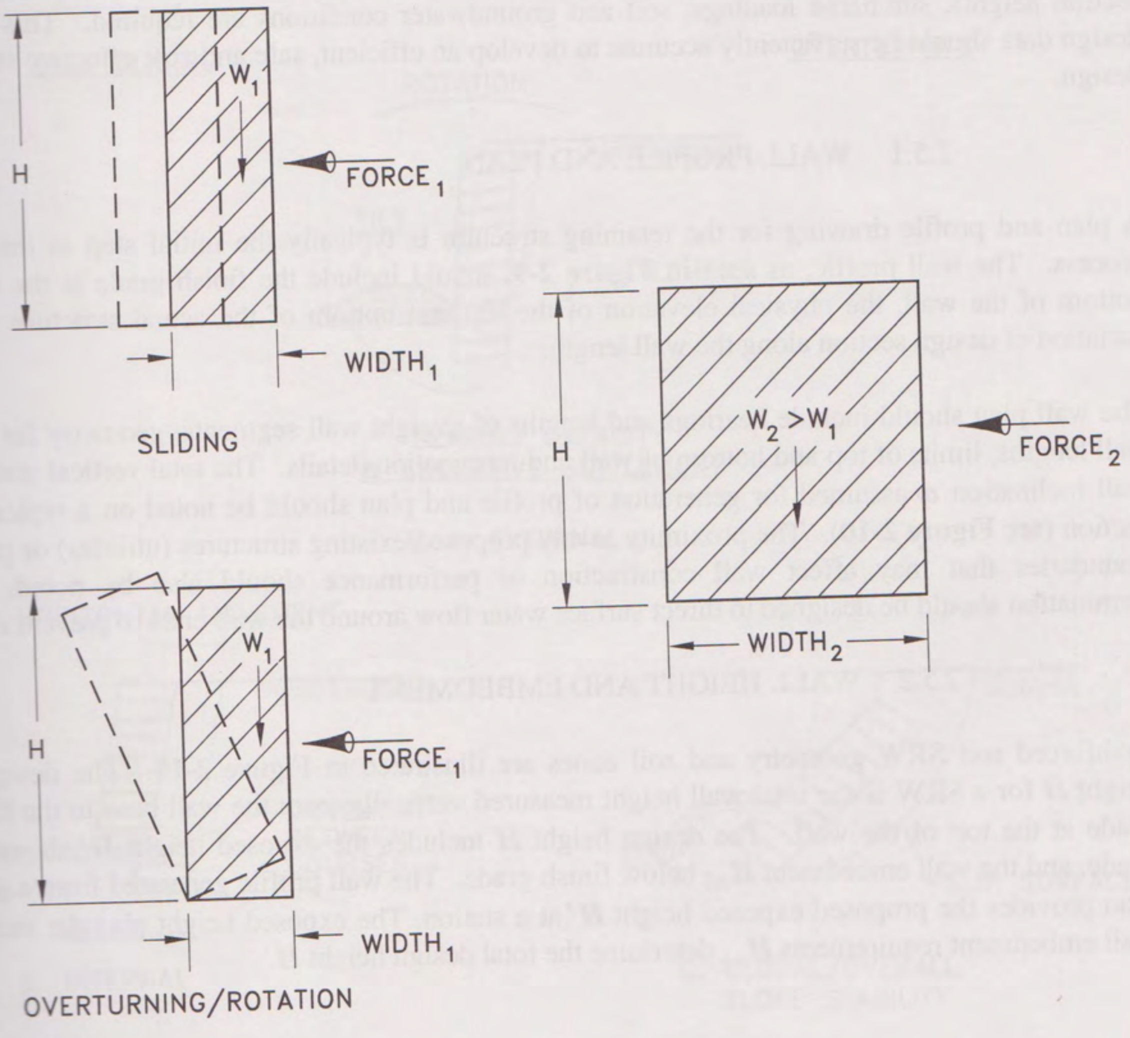
NARROWER WIDTH & LOWER WEIGHT PROVIDES:

- REDUCED FRICTIONAL RESISTANCE
- SMALLER RESISTING MOMENTS

STABLE

WIDER WIDTH & GREATER WEIGHT PROVIDES:

- LARGER FRICTIONAL RESISTANCE
- INCREASED RESISTING MOMENTS



A. MASS 1 B. MASS 2

FIGURE 2–5: GRAVITY WALL CONCEPT

A summary of potential failure modes for conventional gravity SRW structures and reinforced soil SRW structures are provided in **Figures 2-6** and **2-7**. Details of the stability calculations that are carried out with respect to each failure mode are described later in the text of this manual.

A flow chart for the design methodology and calculations associated with conventional and reinforced soil SRWs is presented in **Figure 2-8**.

Section 2.5
GENERAL DESIGN DATA FOR SRW STRUCTURES

Regardless of the type of SRW structure proposed at a site, the general information on wall profile, section heights, surcharge loadings, soil and groundwater conditions are required. This general design data should be sufficiently accurate to develop an efficient, safe and cost effective structural design.

2.5.1 WALL PROFILE AND PLAN

A plan and profile drawing for the retaining structure is typically the initial step in the design process. The wall profile, as seen in **Figure 2-9**, should include the finish grade at the top and bottom of the wall, the physical elevation of the top and bottom of the actual structure and the variation of design section along the wall length.

The wall plan should include bearings and lengths of straight wall segments, geometry for curved wall lengths, limits of top and bottom of wall and termination details. The total vertical wall batter wall inclination $\boldsymbol{\omega}$ assumed for generation of profile and plan should be noted on a typical cross section (see **Figure 2-10**). The proximity to any proposed/existing structures (utilities) or property boundaries that may affect wall construction or performance should also be noted. Wall termination should be designed to direct surface water flow around the wall ends to prevent erosion.

2.5.2 WALL HEIGHT AND EMBEDMENT

Reinforced soil SRW geometry and soil zones are illustrated in **Figure 2-10**. The design wall height $\boldsymbol{H}$ for a SRW is the total wall height measured vertically from the wall base to the finished grade at the top of the wall. The design height $\boldsymbol{H}$ includes the exposed height $\boldsymbol{H'}$ above finish grade, and the wall embedment $\boldsymbol{H_{emb}}$ below finish grade. The wall profile generated from a grading plan provides the proposed exposed height $\boldsymbol{H'}$ at a station. The exposed height plus the minimum wall embedment requirements $\boldsymbol{H_{emb}}$ determine the total design height $\boldsymbol{H}$.

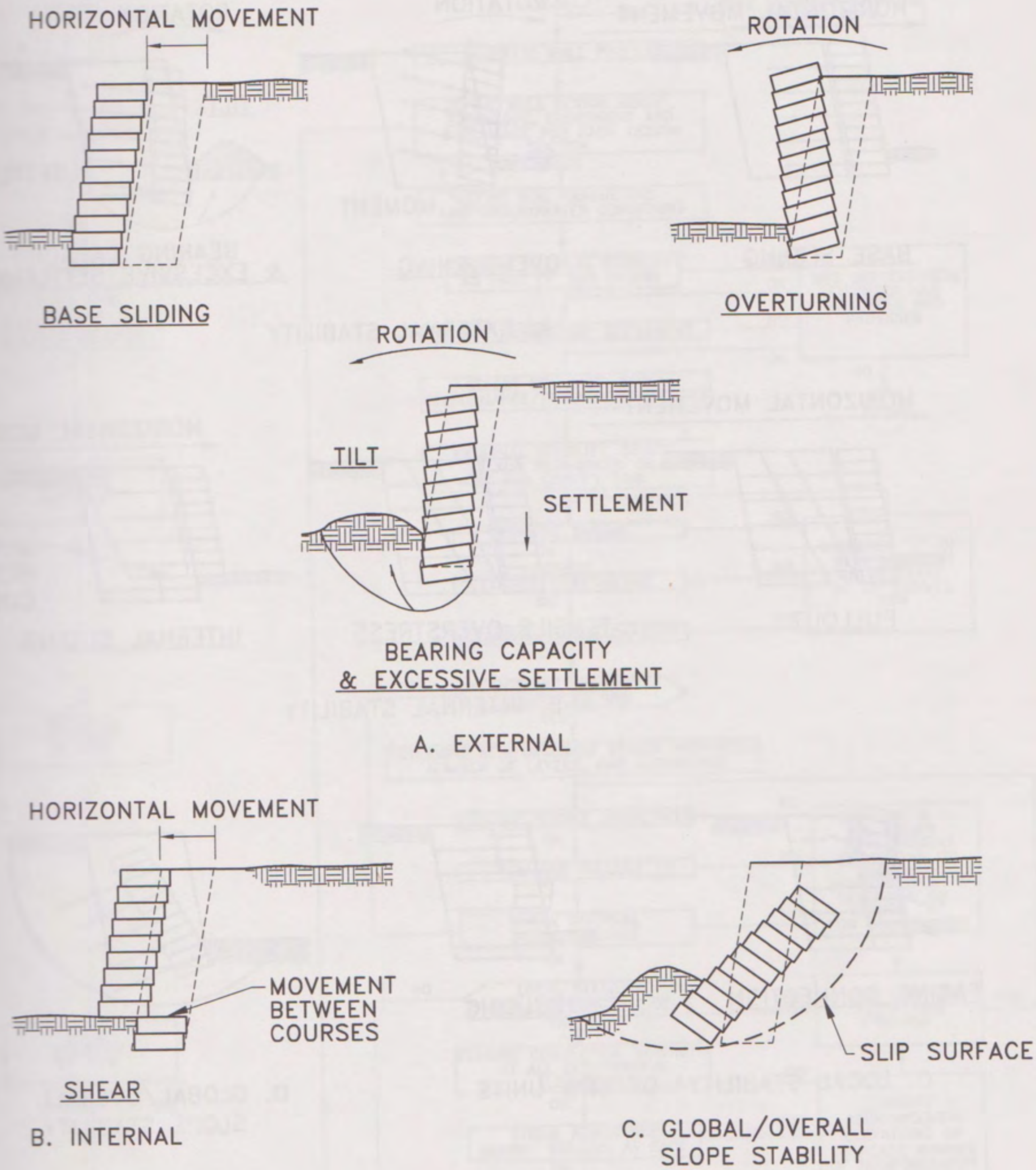
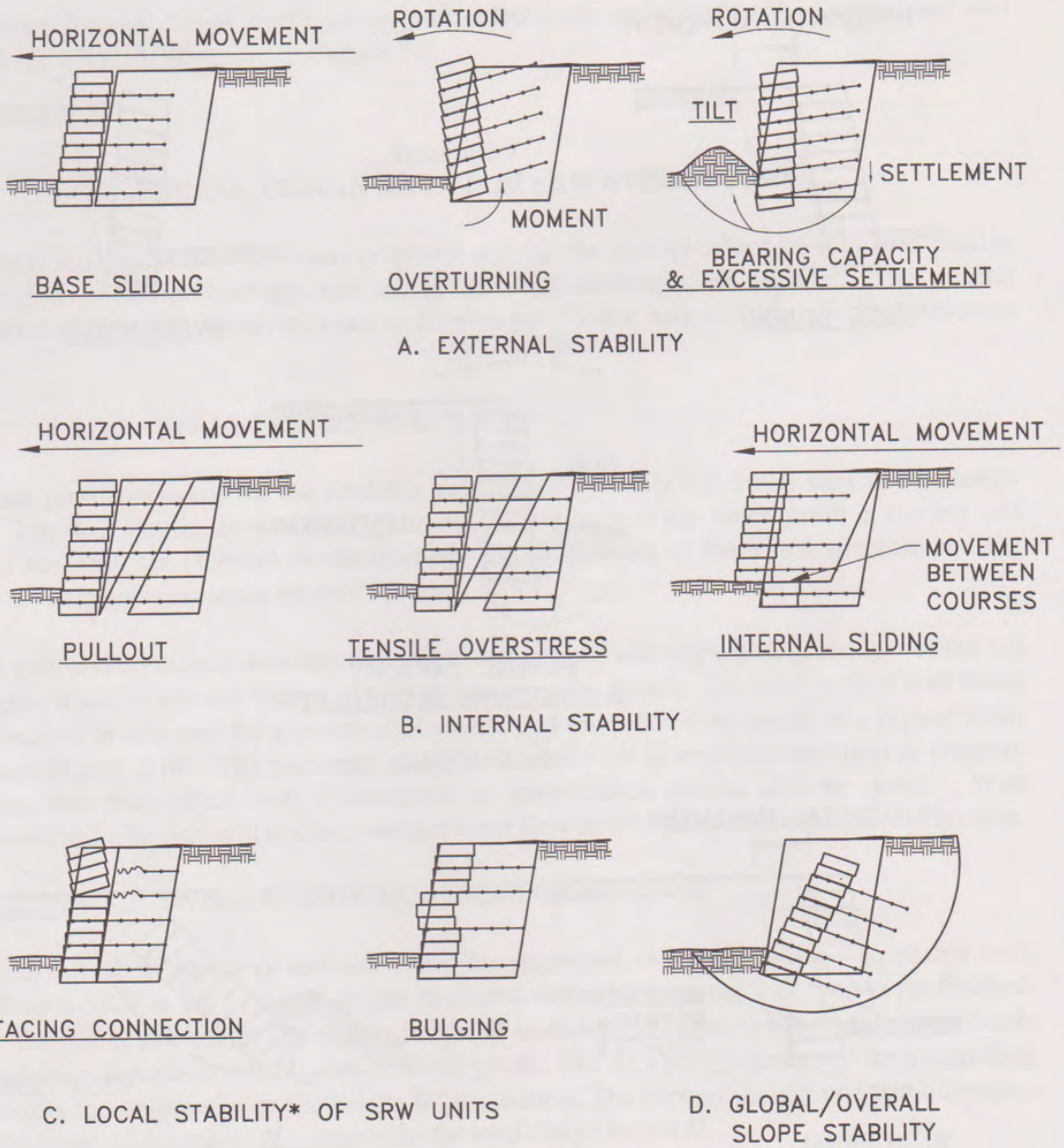


FIGURE 2-6: MAIN MODES OF FAILURE FOR CONVENTIONAL SRWs



* NOTE: THE MAXIMUM UNREINFORCED HEIGHT IS DETERMINED SIMILAR TO CONVENTIONAL SRWs, SEE FIGURE 2-6 A & B.

FIGURE 2-7: MAIN MODES OF FAILURE FOR REINFORCED SOIL SRWs

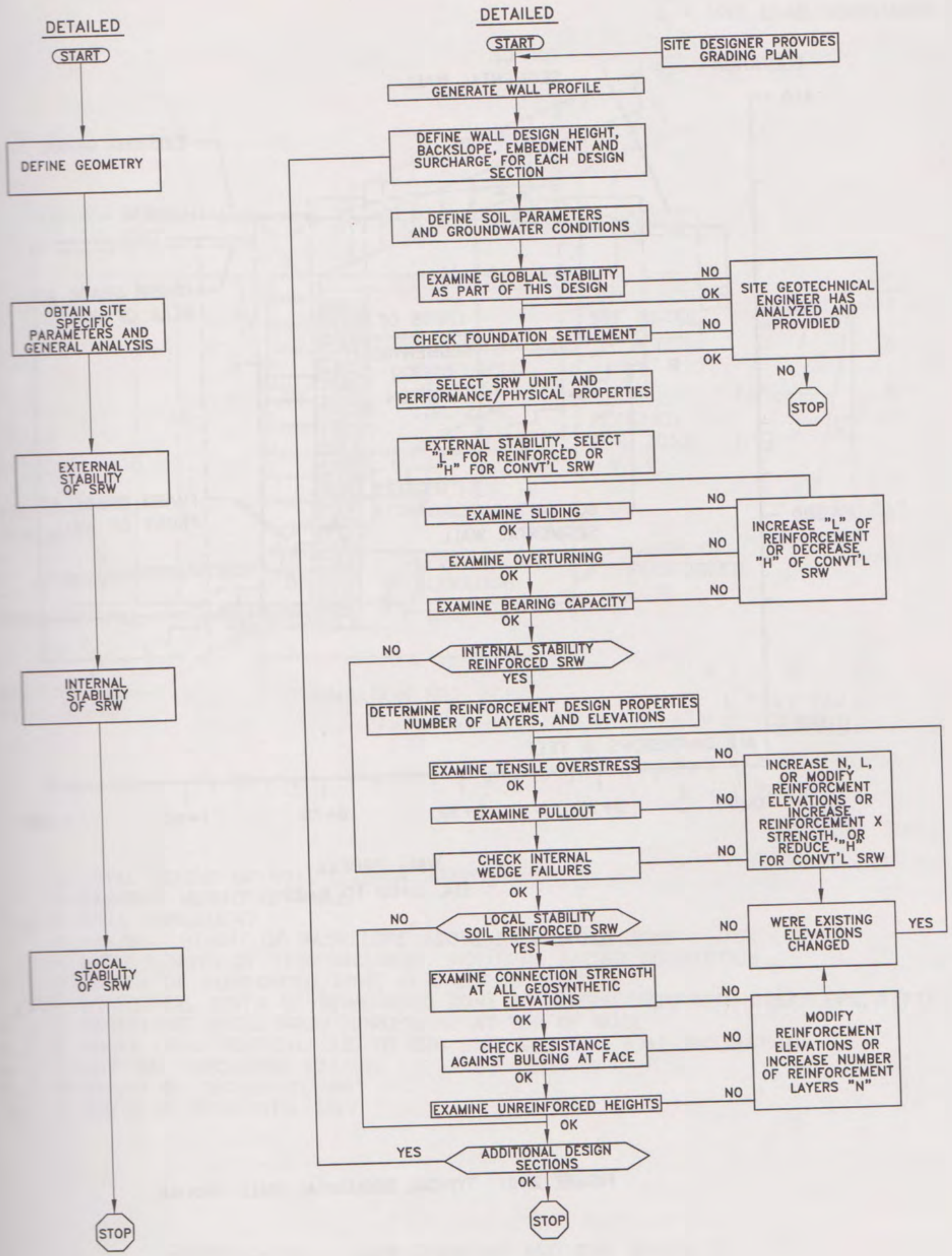


FIGURE 2–8: DESIGN METHODOLOGY FLOW CHART FOR SRWs

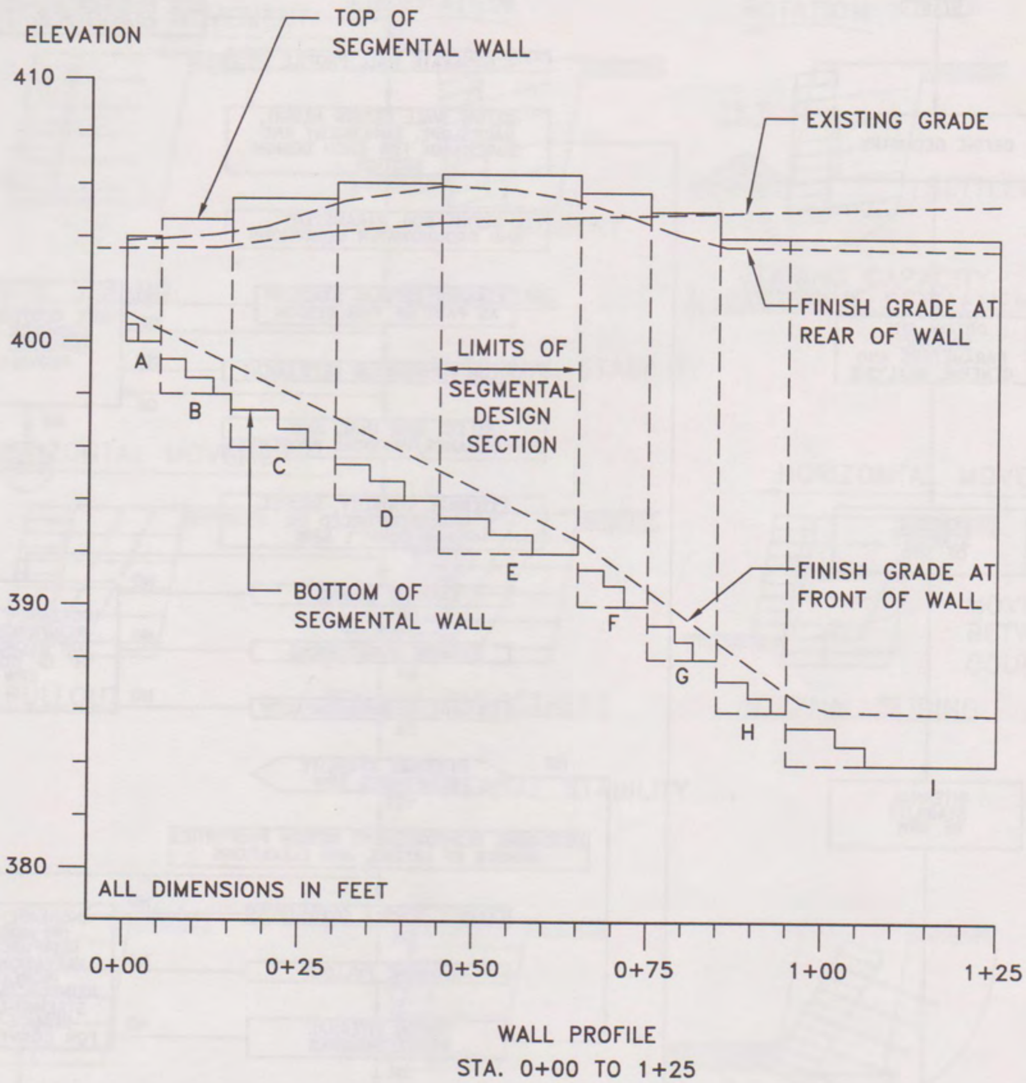
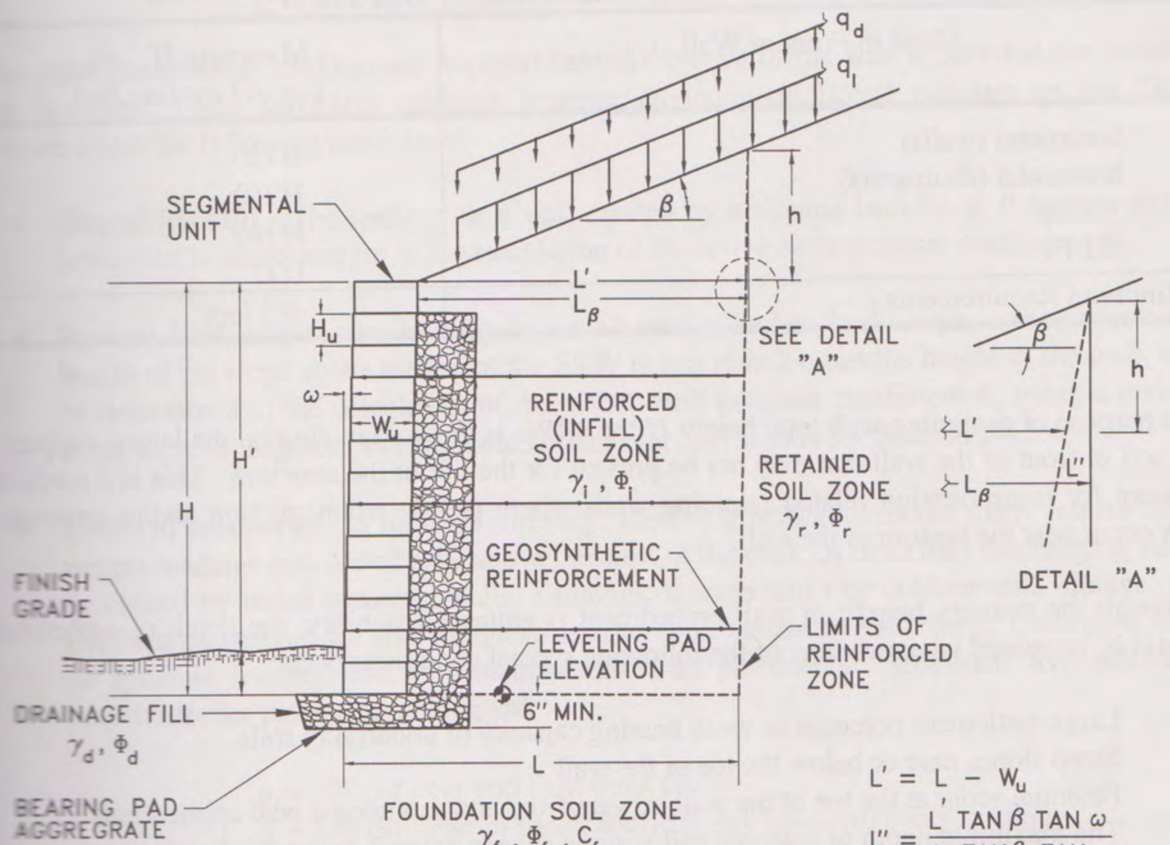


FIGURE 2-9: TYPICAL SEGMENTAL WALL PROFILE



$L' = L - W_u$

$L'' = \dfrac{L \tan\beta \tan\omega}{1-\tan\beta \tan\omega}$

$L_\beta = L' + L''$

$h = L_\beta \tan\beta$

H = TOTAL HEIGHT OF WALL (DESIGN HEIGHT)
H' = EXPOSED HEIGHT OF WALL
H_{emb} = WALL EMBEDMENT
h = MAXIMUM HEIGHT OF BACKSLOPE ABOVE REINFORCED ZONE
L = TOTAL LENGTH OF REINFORCEMENT, INCLUDING FACING CONNECTION
L' = LENGTH OF REINFORCED ZONE AT CREST OF WALL
L_β = HORIZONTAL WIDTH OF REINFORCED ZONE AT INTERSECTION WITH BACKSLOPE, β (FT)
β = BACKSLOPE ANGLE FROM HORIZONTAL AT TOP OF WALL
ω = ANGLE FROM VERTICAL DUE TO SRW UNIT SETBACK, WALL INCLINATION
q = UNIFORM SURCHARGE LOADING
H_u = HEIGHT OF SEGMENTAL UNIT
W_u = WIDTH OF SEGMENTAL UNIT

FIGURE 2–10: SRW GEOMETRY AND SOIL ZONES

The minimum wall embedment H_{emb} shall be measured vertically and determined according to **Table 2-1** below.

TABLE 2-1
MINIMUM WALL EMBEDMENT DEPTH, H_{emb}

Slope in Front of Wall	Minimum H_{emb} to Top of Leveling Pad
horizontal (walls)	H'/20
horizontal (abutments)	H'/10
3H:1V	H'/10
2H:1V	H'/7
Minimum Requirements -	0.5 feet

The purpose of designing with total height H for SRWs is to avoid relying on the lateral support of the soil in front of the wall that may not be present for the life of the structure. This is a particular concern for transportation related retaining walls where paving reconstruction and/or excavation may occur near the bottom of the wall.

Although the primary benefit of wall embedment is enhanced stability, the depth of embedment should be increased whenever any of the following special conditions occur:

- Large settlement potential or weak bearing capacity of underlying soils
- Steep slopes near or below the toe of the wall
- Potential scour at the toe of the wall in waterfront and submerged wall applications
- The maximum depth of seasonal soil volume change extends below the bearing pad
- Seismically active location

The influence of these conditions on wall performance should be addressed by a qualified geotechnical engineer familiar with the site soil and groundwater conditions. In particular, steep slopes beyond the toe of the wall may create global/overall slope instability problems **not** totally accounted for in **Table 2-1**.

Generally, the wall embedment depth (H_{emb}) does not need to extend below seasonal frost depths or other seasonal volume changes. SRWs supported on flexible granular leveling pads can accommodate movement caused by freeze/thaw cycles. The mortarless segmental units are free to move slightly in relation to each other without distress to the wall base. If groundwater and frost susceptible soils are present at the wall base, particularly soils subject to formation of ice lenses, localized movement can be prevented by increasing the thickness of the leveling pad aggregate beyond the minimum 6 inches (**Figure 2-10**). Effectively, this can extend embedment of the wall foundation beneath the frost line without unnecessarily increasing the design wall height (H) used for stability analysis. This also isolates the foundation drain pipes from freezing. Similarly, a moisture barrier outside the toe of the wall can prevent moisture migration and thereby minimize soil volume changes. Foundation soils susceptible to shrinking/swelling during seasonal moisture fluctuations can be addressed in the same manner. SRWs constructed directly on rock foundations still require the minimum aggregate leveling pad thickness of 6 inches to ensure adequate erosion protection of the leveling pad and provide a level surface for construction.

2.5.3 SURCHARGE LOADING AND SLOPED BACKFILLS

Often vertical surcharge loadings are imposed behind the crest of the wall in addition the retained earth loading. These surcharge loadings generate an increased lateral pressure on the SRW structure under the following conditions:

- **Sloped backfill** - The loading on a wall created by a sloping backfill at β degrees to the horizontal is accounted for in the calculation of the active earth pressure coefficient K_a.

- **Broken back slope** - the loading on a wall with a broken back slope, a case where the length of the slope above the top of the SRW is less than 2 times the height of the wall, will be determined in the calculation of the active earth pressure coefficient K_a using a revised slope angle B'degrees. For a detailed discussion on this case see Section 3.4.5.4.

- **Uniform surcharge** - A uniform surcharge loading q is used to model many routine static weight loadings distributed at or above the crest of the wall. A dead load surcharge q_d could be caused by tiered retaining walls, buildings, storage tanks or outdoor bulk storage, etc. Uniform surcharges are also routinely used to approximate live surcharge loadings q_l such as dynamic loading due to vehicular traffic on pavements. Common live surcharge magnitudes for pavement loading are:

  q_l = 100 psf - car and light truck traffic
  q_l = 250 psf - tractor trailer traffic or fire lanes, highway loadings

- **Line or point loads** - The influence of line or point loads applied to the surface behind the crest of a SRW can also be incorporated into this analytical method. Line or point loads may result from heavy isolated footings or continuous footings constructed in close proximity to the crest of the wall. The reader should refer to References 1, 4, 6, 9, 12 and 19 for details on the calculation of the influence of these concentrated loadings.

When defining surcharge loadings, it is important to distinguish between live load (q_l) and dead load (q_d) surcharges, as shown in **Figure 2-10**. Live surcharge loadings q_l are considered to be transient loadings that may change in magnitude and may not be continuously present over the service life of the structure. In this manual, live surcharge loadings are considered to contribute to destabilizing forces only and not to forces that act to stabilize the structure against external or internal failure modes. Examples of live load surcharges are vehicular traffic and bulk material storage facilities. Dead load surcharges, on the other hand, are considered to contribute to both destabilizing and stabilizing forces since they are considered to be of constant magnitude and continuously present for the life of the structure. The weight of a building or another retaining wall above the top of the wall are examples of dead load surcharges.

2.5.4 TIERED WALLS

Retaining walls are sometimes placed in a tiered (multi-level) arrangement. The effect of the upp tier walls is to act as a uniformly distributed dead load on the underlying tiers. Generally, if a tier retaining wall is placed within a horizontal distance (wall face to wall face) less than twice t height of the underlying wall, a surcharge load will be applied to the lower wall. **Figure 2-11** m be used to estimate the equivalent uniformly distributed surcharge loading applied to a lower w by the upper wall for both internal and external stability analyses of a tiered reinforced soil SR system. The same approach is recommended for conventional gravity SRWs except that t dimension L_t in the figure is restricted to the base width of the SRW units (W_u). In both instanc the lower wall height H_1, must be greater than the exposed height H'_2 of the upper wall to use t approximation in **Figure 2-11** . The approximation is also applicable to triple and quadruple tier wall systems by starting the analysis at the lowermost wall.

It is particularly important to check the global stability of the combined tiered wall syste and surrounding soils for both conventional gravity and reinforced soil SRW structures usi slope stability analyses.

2.5.5 GLOBAL STABILITY

The general mass movement of a SRW structure and the adjacent soil mass is called a glob stability failure (**Figures 2-6C** and **2-7D**). Global stability failures may result from changes grade, soil strength, or groundwater regime, and the additional gravitational forces imposed on t site soils as a result of SRW construction. The volume and shape of the failed mass is al influenced by adjacent surface geometry, soil stratigraphy and properties, magnitude of surchar loadings, and groundwater regime. Global stability analyses are routinely carried out usi conventional or modified slope stability methods of analysis.

Most slope stability methods of analysis are based on a "method of slices" approach in whi vertical slices of soil above a trial failure surface are examined with respect to force and mome equilibrium. The shape of the assumed failure surface and the treatment of forces acting on intern slices varies between methods of analysis. The simplest methods of slope stability analysis assur a circular failure surface (**Figure 4-6**). Some modifications are introduced to conventional slo stability methods to account for the stabilizing influence of geosynthetic reinforcement layers if trial failure surface penetrates the reinforced soil zone.

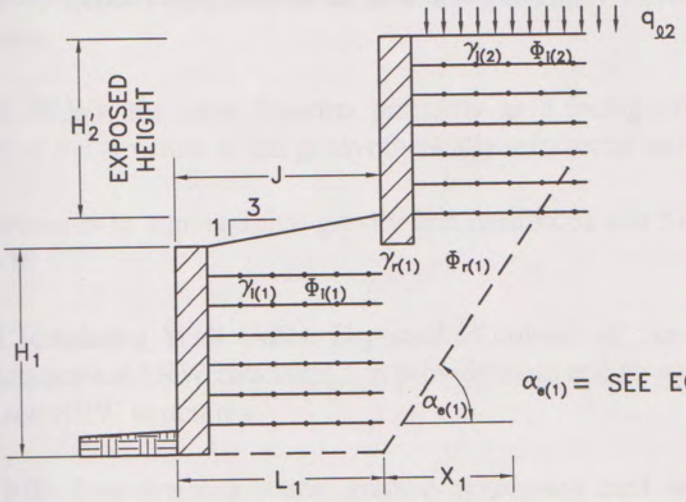
The slope stability calculations associated with a trial failure surface are often tedious and mai potential failure surfaces must be examined in order to determine the critical failure mechanism (i minimum factor of safety). For this reason slope stability analyses are usually carried out usi computer programs that implement one or more different methods of analysis. STABGM ar UTEXAS2 [Refs. 28, 29] are examples of programs that are available to carry out both reinforc

TO DETERMINE APPROXIMATE SURCHARGE
OF UPPER WALL 2 ON LOWER WALL 1
IMPLEMENT BY ITERATIVE PROCESS

1. ESTIMATE BASE LENGTH L_1, AND CALCULATE EXTERNAL FAILURE ANGLE,
2. CALCULATE $q_{\ell 1}$, AND q_{d1}, BASED UPON L_1
3. CALCULATE ACTUAL L_1 BASED UPON EXTERNAL STABILITY ANALYSIS USING $q_{\ell 1}$, AND q_{d1}, FROM STEP 2
4. IF CALCULATED L_1 (STEP 3) < ESTIMATED L_1 (STEP 1). OK. IF NOT REPEAT STEPS 1–4
5. CHECK GLOBAL/OVERALL STABILITY OF FINAL GEOMETRIC CONFIGURATION

J = HORIZONTAL DISTANCE FROM WALL FACE TO WALL FACE

L = BASE LENGTH OF GEOSYNTHETIC REINFORCEMENT FOR SOIL REINFORCED SRWs AND BASE WIDTH OF CONVENTIONAL SRWs



NOTE: H_1 MUST BE > H_2'

$\alpha_{e(1)}$ = SEE EQ. 3–14

$X_1 = (H + J/S)/\tan \alpha_{e(1)}$
S = 500 FOR FLAT (LEVEL) BACKFILL BETWEEN WALLS

FOR INTERNAL STABILITY WALL 1:
WHEN

$J > L_1$ NO INFLUENCE, $q_{d1} = 0$ $q_{\ell 1} = 0$

$.3L_1 < J < L_1$ USE PERCENTAGE OF SURCHARGE

$$q_{d1} = \frac{(L_1 - J)}{L_1} (\gamma_{l(2)} H_2') \qquad q_{\ell 1} = \frac{(L_1 - J)(q_{\ell 2})}{L_1}$$

$J < .3 L_1$ USE FULL SURCHARGE

$$q_n = \gamma_{l(2)} H_2' \qquad q_{\ell 1} = q_{\ell 2}$$

FOR EXTERNAL STABILITY WALL 1:
WHEN

$J > (L_1 + X_1)$ NO INFLUENCE, $q_{d1} = 0$ $q_{\ell 1} = 0$

$(L_1 + .5X_1) < J < (L_1 + X_1)$ USE PERCENTAGE OF SURCHARGE

$$q_{d1} = \frac{(L_1 + X_1 - J)}{X_1} (\gamma_{l(2)} H_2') \qquad q_{\ell 1} = \frac{(L_1 + X_1 - J)(q_{\ell 2})}{X_1}$$

$J < (L_1 + .5X_1)$ USE FULL SURCHARGE

$$q_{d1} = \gamma_{l(2)} H_2' \qquad q_{\ell 1} = q_{\ell 2}$$

NOTE: $.3L_1$ AND $.5X_1$ ARE ARBITRARY BUT EMPIRICALLY BASED GEOMETRIC LIMITS TO ENSURE A CONSERVATIVE SURCHARGE APPROXIMATION

FIGURE 2–11: SURCHARGE APPROXIMATION FOR TIERED WALLS

A detailed presentation of slope stability methods can be found in many geotechnical engineering text books [Refs. 4, 8, 17, 20, 25, 26, 27, 60]. A guideline for design specification and construction of reinforced slopes has recently been published by the FHWA [Ref. 24]. A sample calculation for potential global instability due to the presence of a gravity SRW is presented in Appendix A.

2.5.6 SEISMIC LOADING

The analysis and design methods for SRWs described in this manual are restricted to structures subject to defined static loads. Additional body forces imposed on a SRW due to seismic-induced ground accelerations can increase loadings on SRWs. However, reduced factors of safety for internal stability, external stability and global stability failure mechanisms are typically acceptable for seismic loading conditions. Detailed analysis due to seismic loading of SRWs is not presented in this manual, but a general discussion of available methods is presented in Section 8.3.

SECTION 3
SEGMENTAL RETAINING WALL COMPONENTS

The major components of conventional and reinforced soil SRWs are shown in **Figure 2-1**. This section is focused on the mechanical properties and engineering performance of these SRW components.

Conventional gravity SRWs are comprised solely of SRW units, leveling pad, drainage fills, and retained soil (**Figure 2-1A**). All destabilizing forces must be resisted by the weight of the stacked SRW units and the shearing resistance developed at the top and bottom surface of each unit. The effective width and weight can be increased using a multiple depth SRW system, though this is not fully addressed herein.

For reinforced soil SRWs, the units function primarily as a facing system to control the local stability at the front of the structure of the geosynthetically reinforced soil mass (**Figure 2-1B**).

The principal components in conventional gravity and reinforced soil SRW systems are shown in **Figures 2-1** and **2-10**.

- **Segmental Retaining Wall Units:** Dry-stacked column of concrete units that create the mass of conventional SRW structures and provide mass and structural stability at the face of reinforced soil SRW structures.

- **Drainage Fill:** Free draining coarse grained aggregates used as drainage fill to intercept groundwater and thereby relieve hydrostatic pressure or seepage forces. Drainage fill is also often used to fill cores of SRW units.

- **Reinforced (Infill) Soil**: Compacted structural fill placed behind the drainage fill or directly behind the SRW units if drainage fill is not required behind the SRW units. The reinforced soil will contain horizontal layers of geosynthetic or metallic reinforcement.

- **Retained Soil:** Soil immediately behind the reinforced soil in reinforced soil SRW systems or soil immediately behind the drainage fill or SRW in conventional gravity SRW systems.

- **Foundation Soil**: Soil mass supporting the leveling pad and the reinforced (infill) soil zone of a reinforced soil SRW system. Drainage fill may be placed above the foundation soil to form a blanket drain for the wall system or a bearing pad for the segmental units.

- **Leveling Pad:** Level surface (gravel or concrete) used to distribute the weight of the dry-stacked column of SRW units over a wider foundation area and to provide a working surface during construction. The pad is typically constructed with free draining granular soil to facilitate compaction and drainage.

TABLE 3-1

<table>
<tr><th colspan="8">UNIFIED SOIL CLASSIFICATION SYSTEM [Ref. 17, 31]</th></tr>
<tr><th colspan="6">Field Identification Procedures
(Excluding particles larger than 3 inches)</th><th>Group Symbols</th><th>Typical Names</th></tr>
<tr><td rowspan="8">Coarse-grained soils 50% larger than No. 200 sieve size</td><td rowspan="4">Gravels more than half of coarse fraction is larger than No. 4 sieve size</td><td rowspan="2">Clean gravels</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>GW</td><td>Well graded gravels, gravel-sand mixtures, little or no fines</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>GP</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines</td></tr>
<tr><td rowspan="2">Gravels with fines</td><td colspan="3">Nonplastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty gravels, poorly graded gravel-sand-silt mixtures</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey gravels, poorly graded gravel-sand-clay mixtures</td></tr>
<tr><td rowspan="4">Sand more than half of course fraction is smaller than No. 4 sieve size</td><td rowspan="2">Clean Sands</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>SW</td><td>Well graded sands, gravelly sands, little or no fines</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing</td><td>SP</td><td>Poorly graded sands, gravelly sands, little or not fines</td></tr>
<tr><td rowspan="2">Sands with fines</td><td colspan="3">Nonplastic fines (for identification procedures see ML below)</td><td>SM</td><td>silty sands, poorly graded sand-silt mixtures</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below)</td><td>SC</td><td>Clayey sands, poorly graded sand-clay mixture</td></tr>
<tr><td rowspan="8">Fine-grained soils 50% smaller than No. 200 sieve size</td><th colspan="5">Identification Procedures on Fraction Smaller than No. 40 Sieve Size</th><td></td><td></td></tr>
<tr><td colspan="2" rowspan="4">Silts and clays liquid limit less than 50</td><th>Dry Strength (crushing character-istics)</th><th>Dilatancy (reaction to shaking)</th><th>Toughness (consist-ency near plastic limit)</th><td></td><td></td></tr>
<tr><td>None to slight</td><td>Quick to slow</td><td>None</td><td>ML</td><td>Inorganic silts and ver fine sands, rock flour, silty or clayey fine sands with slight plasticity</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Medium</td><td>CL</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td></tr>
<tr><td>Slight to medium</td><td>Slow</td><td>Slight</td><td>OL</td><td>Organic silts and organic silt clays of low plasticity</td></tr>
<tr><td colspan="2" rowspan="3">Silts and clays liquid limit greater than 50</td><td>Slight to medium</td><td>Slow to none</td><td>Slight to medium</td><td>MH</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts</td></tr>
<tr><td>High to very high</td><td>None</td><td>High</td><td>CH</td><td>Inorganic clays of high plasticity, fat clays</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Slight to medium</td><td>OH</td><td>Organic clays of medium to high plasticity</td></tr>
<tr><td colspan="3">Highly Organic Soils</td><td colspan="3">Identified by color, ordor, spongy feel and fibrous texture</td><td>Pt</td><td>Peat and other highly organic soils</td></tr>
</table>

During stability calculations for reinforced soil SRW structures, SRW units and infill soils are treated as a single homogeneous zone contributing to the mass and width of the structure to simplify external stability calculations.

Section 3.1
SEGMENTAL RETAINING WALL UNITS

The structural integrity of a dry-stacked column of segmental retaining wall units is achieved by incorporating shear connections in the form of shear keys, leading/trailing lips, or pins/clips as shown in **Figures 2-2** and **3-1**. These shear connections increase the shear capacity of the interface between successive courses of SRW units which are generally stacked in a running bond configuration. The shear connections may also be used to control the horizontal setback for successive segmental unit courses and therefore assist in maintaining a constant wall facing batter ω (**Figure 2-10**). Typical facing batter angles are 1° to 15° from vertical. (All wall batter angles are positive when rotated towards infill soils.) The connections may also assist to align and hold geosynthetic reinforcement materials in place.

Segmental units also provide the following:

- **Formwork** - The segmental units provide a construction formwork to place and compact soil in the soil zone immediately behind the units.

- **Facing Stability** - Segmental units provide permanent local support to the vertical or near-vertical soil mass behind the units to prevent soil from ravelling out or eroding. Additionally, the segmental units protect the retained soil zone from erosion due to flowing water or scour from adjacent creeks or streams.

- **Aesthetics** - Segmental units offer architectural treatment and appearance that enhances the environment surrounding the SRW.

Section 3.2
PHYSICAL DIMENSIONS AND PROPERTIES OF SRW UNITS

Segmental retaining wall units come in a variety of sizes and shapes as illustrated in **Figure 2-2**. The dimensions, unit weight and connection performance must be carefully evaluated during the design of the SRW system.

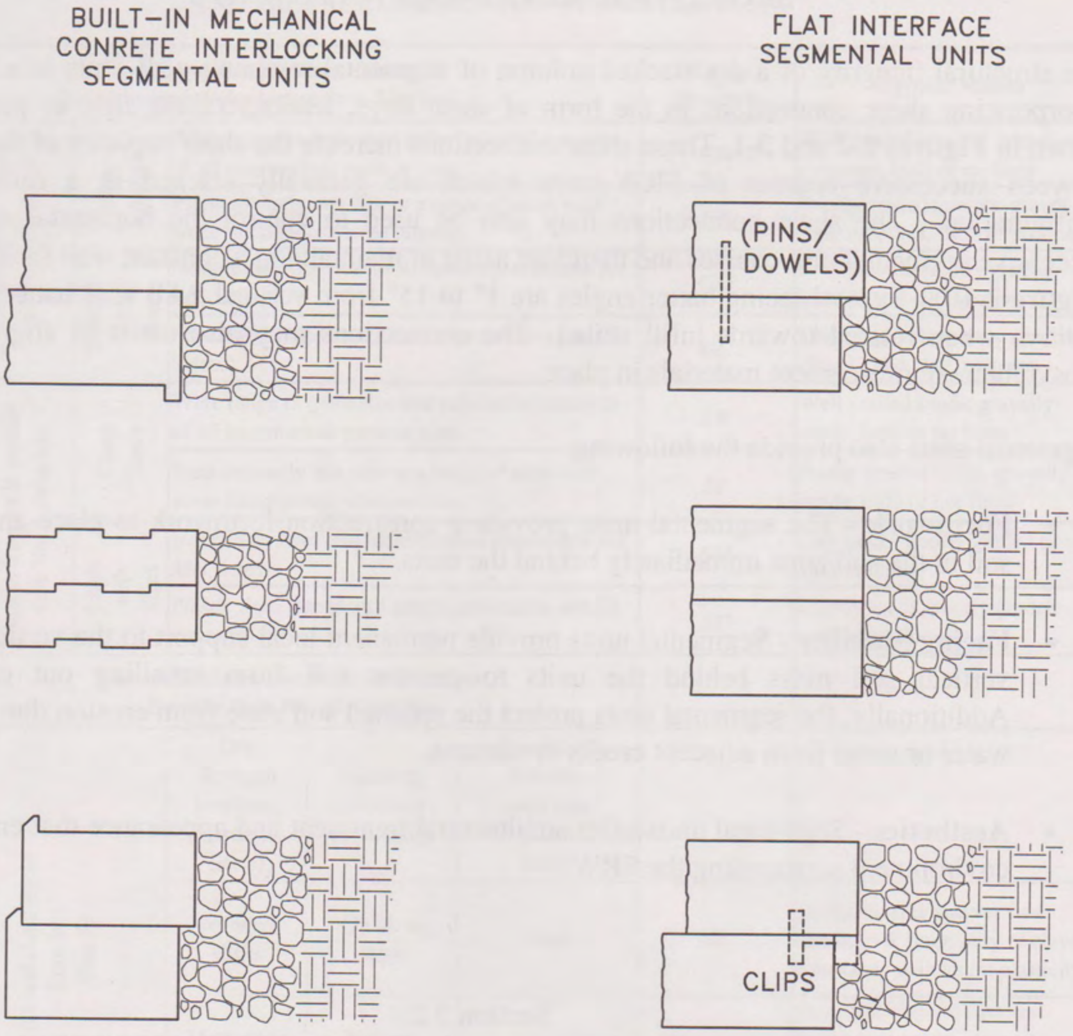


FIGURE 3-1: SHEAR CONNECTOR TYPES FOR SRW UNITS

3.2.1 DIMENSIONS

The physical dimensions of segmental units are defined below and can be referenced to **Figures 2-1 and 2-10:**

- **Height** - (H_u) The vertical dimension of a segmental unit in place (ft).
- **Length** - (L_u) The horizontal dimension of a segmental unit as viewed while observing a completed wall from the front (ft).
- **Width** - (W_u) The horizontal dimension of a segmental unit from front to back. This dimension will appear as a nominal width on a standard cross-section drawing (ft).
- **Weight** - (γ_u) The weight per unit volume of segmental units as placed including drainage fill in and between the segmental units, if applicable (pcf).
- **Setback** - (Δ_u) The horizontal distance (toe to toe) that each SRW unit course is displaced from the course below (in.). [Note: Positive distance into infill soils.]
- **Facing Batter** - (ω) The equivalent uniform facing angle that is created by a SRW setback. The facing batter is measured with respect to a vertical line drawn from the toe of the lowermost SRW unit (deg). [Note: Positive inclination into infill soils.]
- **Center of Gravity** - (G_u) The horizontal distance from the front face of a SRW unit to the center of gravity of the unit as constructed in the field (i.e. filled with aggregate, if applicable) (ft). [Note: G_u in vertical direction assumed to be $H_u/2$ for all equations.]

3.2.2 MIX DESIGN, COMPRESSIVE STRENGTH AND DIMENSIONAL TOLERANCES

Recommendations for minimum compressive strength, maximum water absorption and manufacturing tolerances of SRW units are contained in NCMA **TEK 2-4 Specification for Segmental Retaining Wall Units**. The job site acceptance or rejection of SRW units can be made on these three physical properties and the NCMA **TEK 2-4** requirements for finish and appearance, all of which should be incorporated into the job site specification.

The compressive strength requirements for SRW units in NCMA **TEK 2-4** are governed by the durability requirements of a particular application (installation) and/or the stress transfer requirements between successive courses. Historically, the freeze/thaw resistance and durability of concrete masonry is demonstrated in over 50 years of satisfactory performance in foundation and retaining walls.

Compressive strength and water absorption are two physical properties of the no-slump concrete used to make SRW units which have been related to freeze/thaw resistance [Ref. 53]. The required values for these properties are intended to provide a durable unit for in field service. The freeze/thaw resistance of segmental retaining wall units are affected by temperature cycles, exposure conditions, and moisture content of the units [Ref. 57]. The drainage details presented in this manual help to minimize the moisture content of the units by allowing water to drain freely to the discharge pipe at the base of the wall. In some weathering regions and for some exposure conditions (i.e., along a highway where units may be in contact with de-icing salts), higher SRW unit compressive strengths than the minimum value presented in **TEK 2-4** may be necessary to provide additional durability resistance in aggressive environments.

For routine concrete block masonry, the manufacturing tolerances on any molded dimension is ± ⅛ in. Such manufacturing tolerance is acceptable on both the width and length of segmental units. However, because of the dry-stacked construction method employed with SRW units, the manufacturing tolerance on unit height requires tighter control to maximize both the shear capacity and geosynthetic connection strength, and ensure uniform weight distribution. A manufacturing tolerance of ± 1/16 in. on unit height is recommended to ensure overall performance properties and quality.

3.2.3 SRW PERFORMANCE PROPERTIES

The unique shape of each SRW unit directly affects its ability to function in retaining wall applications. Most significant is the mechanism utilized to transfer stress to the units above and below it.

There are two common methods to create shear connections between successive vertical courses of segmental units as illustrated in **Figure 3-1:**

- **Built-in mechanical concrete interlock** - These SRW units have a positive, built-in mechanical interlock that is an integral part of the segmental unit shape. Examples of this positive mechanical interlock are shear keys and leading/trailing lips.

- **Flat interface** - This type of SRW unit develops shear capacity through interface friction.

These principal methods of connection can be augmented with:

- **Mechanical connectors** - The specially manufactured connectors that link successive vertical courses of units together may be designed as a mechanical interlock to provide additional shear capacity, and are also used to assist with unit alignment and control the wall facing batter during wall construction. Examples of mechanical connectors are pins, clips, or wedges.

- **Unit fill interlock** - SRW unit cavities filled with drainage fill can develop significant shear resistance through soil to soil and soil to unit shear strength.

The shear capacity V_u between segmental courses generated by these connections is an important parameter in the stability analysis of both conventional and reinforced soil SRWs. The magnitude of V_u will be a function of height of wall, unit type, infill, and in reinforced soil SRWs, the properties of any geosynthetic reinforcement layer that may be present between.courses of SRW units. Similarly, the connection strength $T_{c\ell}$ developed between the SRW unit and geosynthetic reinforcement may control the design of reinforced soil SRWs. Appropriate values for V_u and $T_{c\ell}$ used in design can only be established from the results of large-scale testing of the connection detail in SRW systems as described below and in Appendix C.

The magnitude of V_u and $T_{c\ell}$ calculated from full-scale test results will be compared to the actual applied force for each specific SRW unit elevation or geosynthetic reinforcement layer during design computations. The shear capacity and connection strength must be greater than the applied loads by the minimum required safety factors.

3.2.3.1 Connection Strength Between Geosynthetic Reinforcement and SRW Units

The procedure to quantify the connection strength between a geosynthetic and SRW units is given in **NCMA** Test Method **SRWU-1: Determination of Connection Strength between Geosynthetics and Segmental Concrete Units** provided in Appendix C.

The purpose of a test carried out in accordance with SRWU-1 is to establish the connection strength of segmental concrete units to a geosynthetic reinforcement layer for a given surcharge pressure applied normal to the connection interface. A series of tests are conducted to establish a relationship between facing connection strength and height of stacked segmental units defined by a Mohr-Coulomb failure criteria over a representative range of surcharge pressures to which the interface is exposed to. The surcharge pressures used in the test must cover the range of normal interface pressures anticipated in the proposed structure. This test is intended to be a performance test, since variations in segmental unit shape and geosynthetic strength/configuration will affect the results. Equivalent Mohr-Coulomb failure criteria parameters a_{cs} and λ_{cs} determined through full-scale testing can then be used by the designer to calculate the ultimate connection strength over the full face height of the wall. Typical relationships for connection strength performance are illustrated in **Figure 3-2**. Likewise, parameters a'_{cs} and λ'_{cs} can be utilized to determine the service state connection strength for design calculations. The relationship developed from testing may be linear or in some cases bilinear. For design purposes, systems that exhibit bilinear behavior will be represented by a linear equation for low normal loads and a maximum connection strength values $T_{cs(max)}$ or $T_{cs(max)'}$

3.2.3.2 Determination of Shear Strength Between SRW Units

The procedure to quantify the shear strength between SRW units is given in **NCMA** test method **SRWU-2: Determination of Shear Strength Between Segmental Concrete Units** provided in Appendix C. The purpose of a test carried out according with SRWU-2 is to measure the shear strength between segmental concrete units for a given pressure applied normal to the unit interface. A series of tests are carried out to establish a relationship between interface shear strength and height of stacked segmental units defined by a Mohr-Coulomb failure criteria over a representative range of normal loads. The normal pressures used in the test must cover the range of normal interface pressures anticipated in the proposed structure. This test is intended to be a performance test, since variations in segmental unit shape and geosynthetic strength/configuration will affect the results. Equivalent Mohr-Coulomb strength parameters a_u and λ_u determined through full-scale testing can then be used by the designer to calculate the shear strength over the full face height of the wall. Typical relationships for shear capacity performance are illustrated in **Figure 3-2**. Likewise, parameters a'_u and λ'_u can be utilized to determine the service state shear capacity for design computations. The relationship developed from testing may be linear or in some cases bilinear. For design purposes, systems that exhibit bilinear behavior will be represented by a linear equation for low normal loads and maximum shear strength $V_{u(max)}$ and $V'_{u(max)}$.

The full scale performance test described in **SRWU-2** is meant to provide shear capacity for segmental units with and without a geosynthetic reinforcement. The results of this testing are directly applicable to the analysis and design of both conventional and soil-reinforced SRWs.

Section 3.3
DRAINAGE MATERIALS

Engineered drainage materials are an important part of a properly designed SRW. Drainage materials are generally well-graded aggregates (such as coarse sands and gravel GW/SW). In many cases the drainage fill is separated from the fill soils by a geotextile and contains a drainage pipe to direct accumulated water away from the structure. A properly designed drainage system will do the following:

- Prevent the build up of hydrostatic pressures in the retained soils and foundation soils in the vicinity of the wall toe.
- Prevent retained soils from washing through the face of the wall.
- Provide a stiff leveling pad to support a column of stacked facing units and provide a working surface during construction.

3.3.1 INTERNAL DRAINAGE

Many retaining wall failures are caused by poor drainage. Poor drainage leads to development of hydrostatic pressures or seepage forces in the retained soils that in turn generate additional destabilizing forces on the wall system and can reduce shear strength of the soil. The design methodology presented in this manual assumes the groundwater table is well below the leveling pad elevation of the conventional or reinforced soil SRW structure (e.g. greater than 0.66H below the toe of the wall).

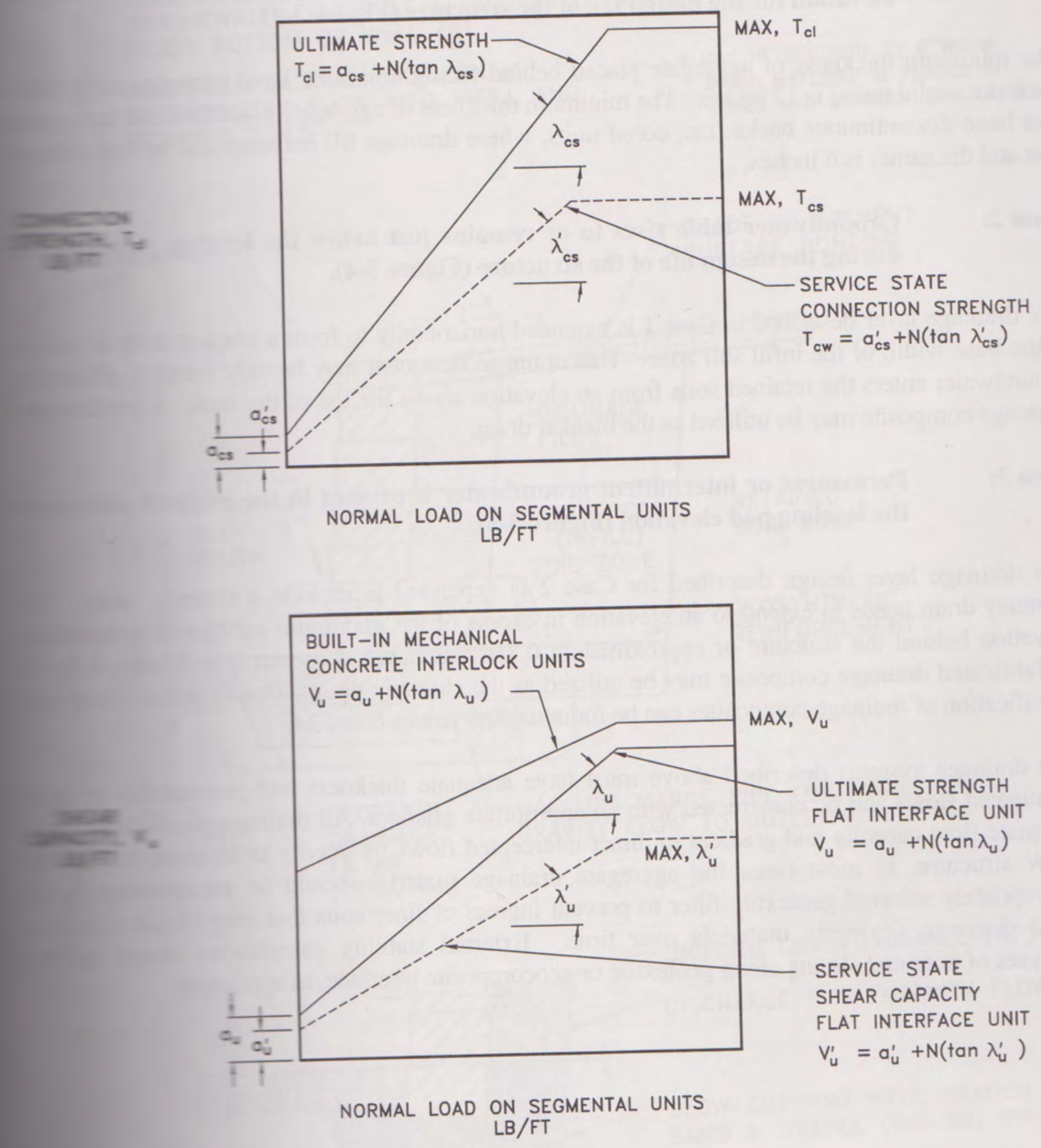


FIGURE 3-2: TYPICAL CONNECTION STRENGTH AND SHEAR CAPACITY PERFORMANCE PROPERTIES FOR SRW UNIT AND GEOSYNTHETIC REINFORCEMENT

Three strategies to prevent the infiltration of groundwater in a SRW structure are summarized below. The characteristics of the groundwater regime will dictate which option is required:

Case 1: Groundwater table remains a distance 0.66H below the base of the leveling pad elevation for the design life of the structure (Figure 3-3).

The minimum thickness of aggregate placed behind facing units that have a continuously closed back (i.e., solid units) is 12 inches. The minimum thickness of aggregate placed behind facing units that have discontinuous backs (i.e., cored units, where drainage fill between and behind units are one and the same) is 6 inches.

Case 2: Groundwater table rises to or remains just below the leveling pad elevation during the design life of the structure (Figure 3-4).

The drainage layer described in Case 1 is extended horizontally to form a blanket drain across the entire base width of the infill soil zone. This drainage treatment may be only partially effective if groundwater enters the retained soils from an elevation above the toe of the wall. A prefabricated drainage composite may be utilized as the blanket drain.

Case 3: Permanent or intermittent groundwater is present in the retained soils above the leveling pad elevation (Figure 3-5).

The drainage layer design described for Case 2 is expanded to include a chimney drain. The chimney drain needs to extend to an elevation in excess of the maximum anticipated groundwater elevation behind the structure or approximately $0.7H$ whichever is greater (see **Figure 3-5**). A prefabricated drainage composite may be utilized as the chimney drain. Details on the design and specification of drainage composites can be found in References 6 and 24.

The drainage systems described above must have adequate thickness and permeability to carry anticipated flows and be constructed with an appropriate gradient. All drainage pipes should have adequate flow capacity and gradient to direct intercepted flows by gravity to locations beyond the SRW structure. In most cases the aggregate drainage material should be encapsulated by an appropriately selected geotextile filter to prevent ingress of finer soils that may clog the coarser-sized drainage aggregate materials over time. External stability calculations should include analyses of potential sliding along geotextile or geocomposite interface, as applicable.

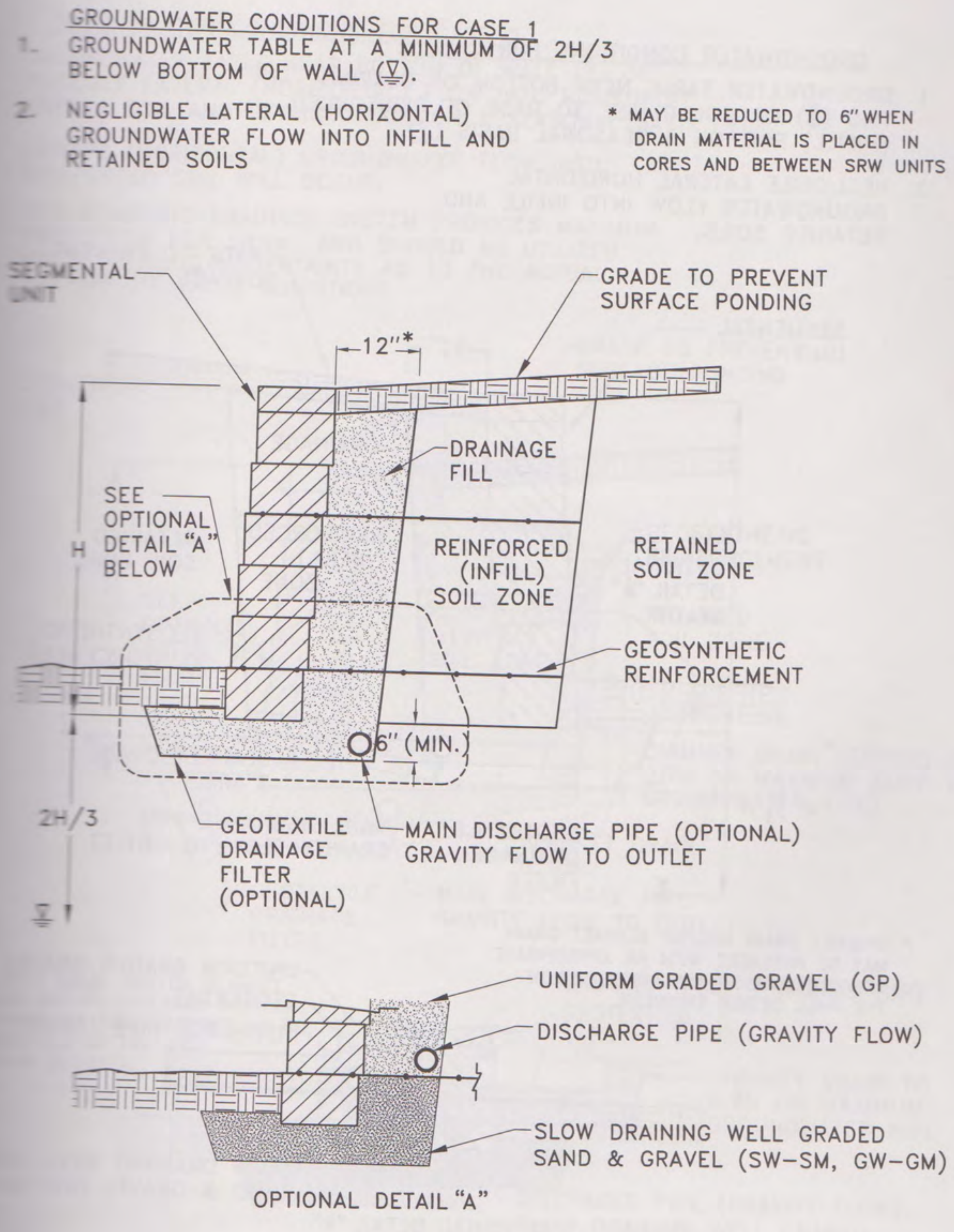


FIGURE 3-3: WALL FACE DRAIN – CASE 1

GROUNDWATER CONDITIONS FOR CASE 2

1. GROUNDWATER TABLE NEAR BOTTOM OF BEARING PAD (▼) OR COULD RISE TO BASE OF REINFORCED (INFILL) SOIL ON A SEASONAL BASIS (▽).
2. NEGLIGIBLE LATERAL HORIZONTAL GROUNDWATER FLOW INTO INFILL AND RETAINED SOILS.

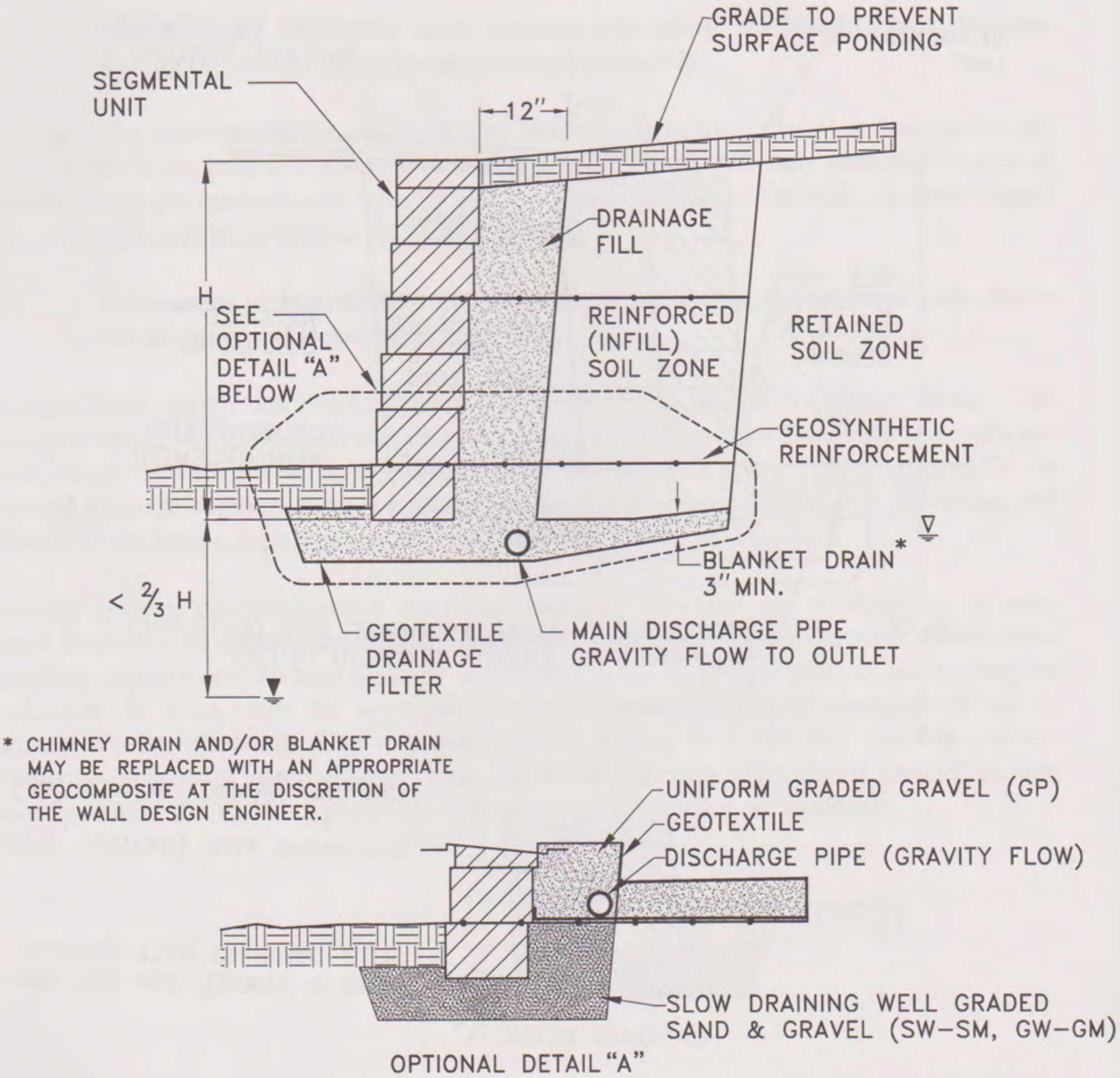


FIGURE 3–4: WALL FACE AND BLANKET DRAIN – CASE 2

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.

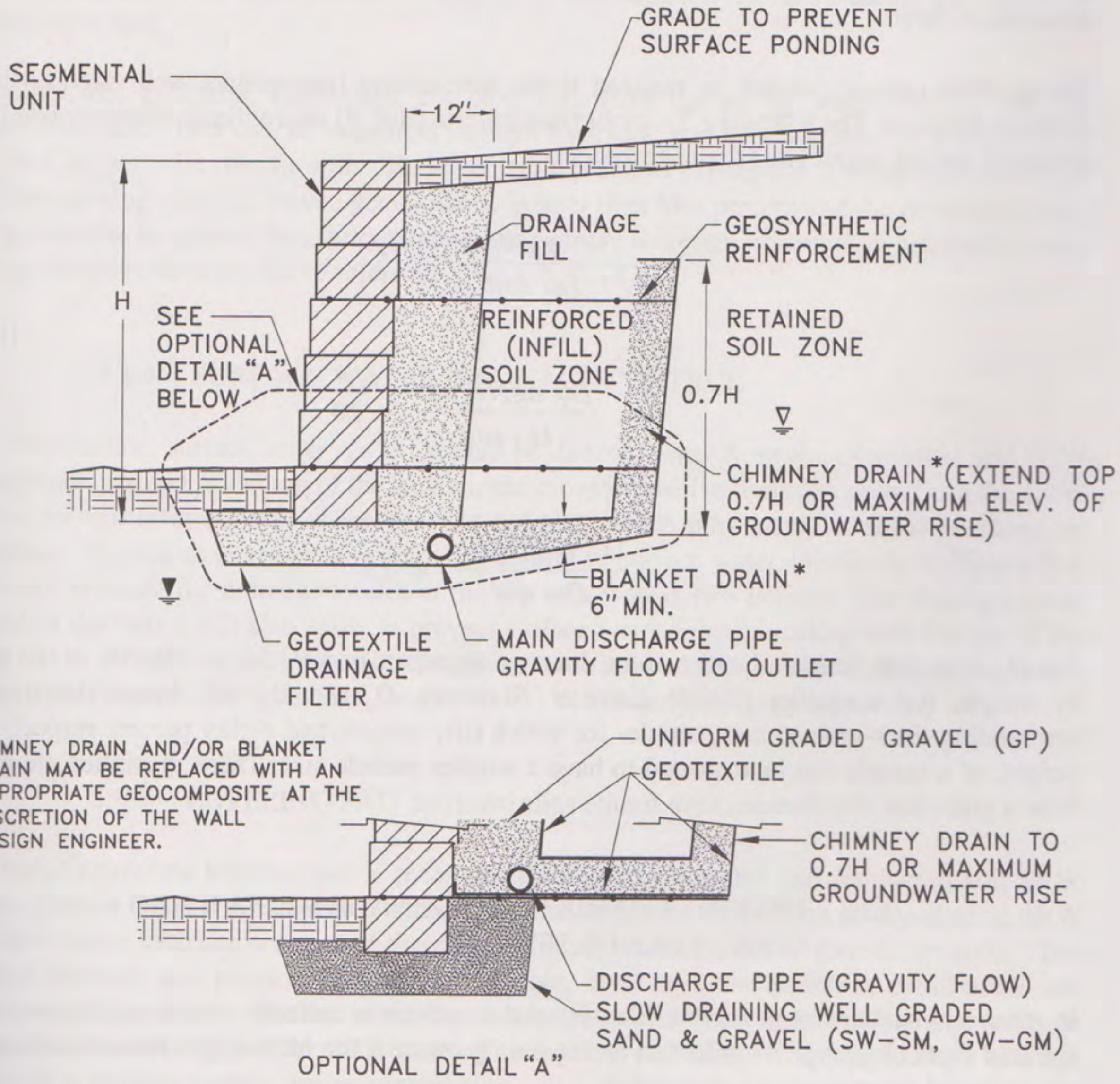


FIGURE 3–5: COMPLETE DRAINAGE SYSTEM – CASE 3

3.3.2 DESIGN CRITERIA FOR NATURAL DRAINAGE MEDIA

Drainage fill materials should be selected to provide the following:

- Sufficient permeability and cross-sectional area to carry anticipated flows.
- Filtration of fine-grained soil to prevent clogging of the aggregate drainage medium if a geotextile filter is not used.

The flow capacity of an aggregate drain can be estimated from Darcy's equation: $\boldsymbol{Q = kiA}$ where $\boldsymbol{Q}$ is the flow capacity (ft^3/sec), $\boldsymbol{k}$ is the permeability of the aggregate drain (ft/sec); $\boldsymbol{i}$ is the hydraulic gradient (ft/ft) and $\boldsymbol{A}$ is the minimum cross-sectional area (ft^2) of the drain perpendicular to the direction of flow.

Design flow capacity cannot be realized if the surrounding finer-grained soil migrates into the drainage medium. The following Terzaghi filter criteria [Ref. 8] are routinely recommended for the design of natural (aggregate) drainage systems:

[Eq. 3-1]

$$\frac{D_{15}\ aggregate}{D_{85}\ soil} < 5$$

[Eq. 3-2]

$$4 < \frac{D_{15}\ aggregate}{D_{15}\ soil} < 20$$

[Eq. 3-3]

$$\frac{D_{50}\ aggregate}{D_{50}\ soil} < 25$$

The D_{15} aggregate denotes the size of the drainage aggregate particle for which 15% of the material, by weight, has a smaller particle diameter. Similarly, D_{50} and D_{85} soil denote the size of the surrounding finer-grained soil particles for which fifty percent and eighty percent respectively, by weight, of a sample can be expected to have a smaller particle size. These quantities are extracted from a grain size distribution curve for the soils involved.

3.3.3 DESIGN CRITERIA FOR DRAINAGE MEDIA USING GEOTEXTILE FILTERS

In many applications, a geotextile filter is used to protect a uniform natural (aggregate) drainage material from clogging. To select an appropriate geotextile, the following criteria based on current conventional practice are recommended:

[Eq. 3-4]

$$\frac{AOS\ geotextile}{D_{85}\ soil} < 3$$

[Eq. 3-5]

$$\frac{AOS\ geotextile}{D_{15}\ soil} > 3$$

[Eq. 3-6]

$$k_{geotextile} \geq 10\,(k_{soil})$$

The AOS value is the Apparent Opening Size of the candidate geotextile determined from the results of the ASTM D 4751 method of test. The quantity $k_{geotextile}$ is the geotextile normal permeability and is calculated as the product of geotextile thickness (in.) times permittivity (1/sec) normal to the plane of the geotextile. Permittivity is determined from the results of the ASTM D 4491 method of test.

Additionally, the geotextile itself must not clog with fines carried by water passing into the aggregate material. This can be examined through hydraulic conductivity ratio or gradient ratio testing [Ref. 6] with site specific soils. However, from practical experience, it is recommended that the percent opening size of a woven geotextile be greater than four percent and the porosity of non-woven geotextiles be greater than thirty percent. For a more thorough treatment of geotextile drains the reader should refer to the list of references [Refs. 6, 9, 18].

3.3.4 SURFACE WATER DRAINAGE CONTROL

During construction, surface water run-off should be directed away from the excavation and SRW system. Finish grading at the top of the wall should provide positive drainage away from the SRW system to prevent infiltration of water into retained soils which may increase lateral pressures on the structure. Typical drainage swale details for control of surface water are shown in **Figure 3-6**. A minimum gradient for drainage swales at the top of a wall is two percent. The drainage swale illustrated in the figure will also assist to prevent surface water from cascading over the top of the wall. A 4 to 6 in. thick top soil cover is recommended to support the growth of surface vegetation that in turn provides erosion protection for the top soil and underlying low permeability soil.

3.3.5 LEVELING PAD

A compacted aggregate leveling pad is recommended for conventional and reinforced soil SRW structures (**Figure 2-1**). The pad serves to distribute the weight of the column of dry-stacked SRW units over a wider foundation area and thereby minimizes overstressing of foundation soils. The compacted leveling pad provides a stiff, but flexible, layer to assist in stress distribution and attenuation of differential settlement stress that minimizes stress concentrations for taller walls to reduce cracking and spalling of the SRW units. The leveling pad also performs a drainage function and provides a working surface during construction.

The compacted granular leveling pad should be constructed using GP, GW, SP or SW soil types for optimum stress distribution and drainage. The leveling pad should be densely compacted. Alternatively, thin/weak concrete leveling courses may be poured above the compacted leveling pad to speed construction. The leveling pad should not be less than 6 inch thick. The leveling pad should extend laterally at least a distance 6 inches from the toe and heel of the lowermost SRW unit.

In situations where gravity flow of the wall underdrain is unattainable, the leveling pad may be constructed of a densely graded-impermeable soil to preclude saturation and the drain pipe located above finish grade at toe of wall.

Section 3.4
SOILS

The soil materials within and adjacent to a SRW will typically exert the greatest influence on the final design of the structure. For a given wall height and geometry the properties of the retained soils will often control the choice of a conventional SRW or a reinforced soil SRW system. The soils selected for placement immediately behind the facing units in reinforced soil SRW construction are a principal structural component of these systems. The design challenge is to match the soil properties, segmental unit properties and geosynthetic reinforcement to obtain an optimized design.

This manual will overview some of the key elements of geotechnical engineering as background information. The reader is directed to the list of references for textbooks on soil mechanics principles [Refs. 8, 17, 20, 25, 26, 27] for a more complete explanation and details on soil properties and behavior.

A key economic advantage of SRW systems is that on-site soils can usually be used. This minimizes the costs associated with importing fill materials and/or removing excavated materials. Provided that groundwater conditions at a site are controlled by the recommendations given earlier in the manual, a wide variety of soil types become candidate materials for the infill soil in the reinforced soil zone. For example, while cohesionless free draining materials (less than 10% fines) are preferred, soils with fines with low plastic fines (i.e., CL, ML, SM, SC with PI $\leq$ 20) may be used for SRW construction provided the following four additional design criteria are implemented:

- **Proper internal drainage is installed (Section 3.3).**
- Only soils with low to moderate frost heave potential are utilized, see **Table 3-3**.
- The internal cohesive shear strength parameter **c** is conservatively ignored for stability analysis (Section 3.4.2).
- The final design is checked by a qualified geotechnical engineer to ensure that the use of cohesive soils does not result in unacceptable time-dependent movement of the SRW system.

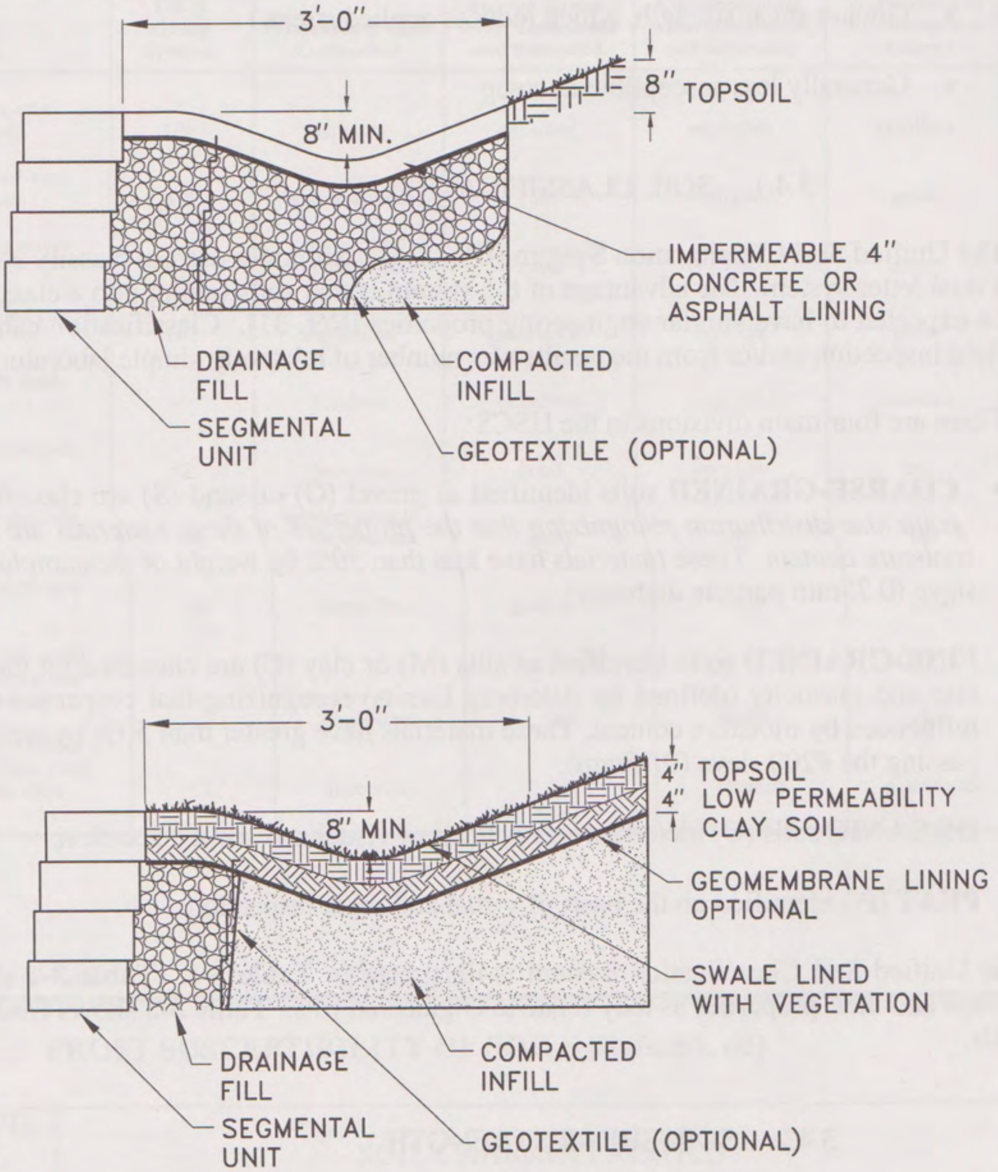


FIGURE 3-6: DRAINAGE SWALE DETAILS

The advantage of granular soil fills are:

- Easier to place and compact
- Higher permeability which assists drainage
- Greater shear strength, which reduces applied stress
- Generally less susceptible to creep

3.4.1 SOIL CLASSIFICATION

The Unified Soil Classification System (USCS) is commonly used to classify soils and is based on a dual letter system. The advantage of the system is that soils falling into a classification group can be expected to have similar engineering properties [Ref. 31]. Classification can often be based on field inspection and/or from the results of a number of relatively simple laboratory tests.

There are four main divisions in the USCS:

- **COARSE-GRAINED** soils identified as gravel (G) or sand (S) are classified on the basis of grain size distribution recognizing that the properties of these materials are not influenced by moisture content. These materials have less than 50% by weight of the sample passing the #200 sieve (0.75mm particle diameter).
- **FINE-GRAINED** soils identified as silts (M) or clay (C) are classified on the basis of particle size and plasticity (defined by Atterberg Limits) recognizing that properties of these soils are influenced by moisture content. These materials have greater than 50% by weight of the sample passing the #200 sieve (0.75mm).
- **ORGANIC** soils (O) identified on the basis of visual and scent inspection.
- **PEAT** (Pt) identified on the basis of visual and scent inspection.

The Unified Soil Classification System is illustrated in **Table 3-1**. **Table 3-2** shows typical soil groups and their properties as they relate to engineered fills. **Table 3-3** shows frost susceptibility of soils.

3.4.2 SOIL SHEAR STRENGTH

The conventional approach in geotechnical engineering is to describe the shear strength of a soil using a Mohr-Coulomb failure criteria. The Mohr-Coulomb failure criteria relates the normal stress acting on an internal soil failure plane to the peak shearing resistance that is available along that surface. The shear strength τ is expressed as

[Eq. 3-7]

$$\tau = c + \sigma_n \tan \phi$$

TABLE 3-2

ENGINEERING PROPERTIES OF SOILS [Ref.17,31]					
Typical Names of Soil Groups	**USCS Group Symbols**	**Permeability when Compacted**	**Shearing Strength when Compacted and Saturated**	**Compressibility when Compacted and Saturated**	**Workability as a Construction Material**
Well-graded gravels, gravel-sand mixtures, little or no fines	GW	pervious	excellent	negligible	excellent
Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	very pervious	good	negligible	good
Silty gravels, poorly graded gravel-sand-silt mixtures	GM	semipervious to impervious	good	negligible	good
Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	impervious	good to fair	very low	good
Well-graded sands, gravelly sands, little or no fines	SW	pervious	excellent	negligible	excellent
Poorly graded sands, gravelly sands, little or no fines	SP	pervious	good	very low	fair
Silty sands, poorly graded sand-silt mixtures	SM	semipervious to impervious	good	low	fair
Clayey sands, poorly graded sand-clay mixtures	SC	impervious	good to fair	low	good
Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	semipervious to impervious	fair	medium	fair
Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	impervious	fair	medium	good to fair

Table 3-3

FROST SUSCEPTIBILITY OF SOILS [Refs. 59, 60]	
FROST HEAVE POTENTIAL	SOIL CHARACTERISTICS
High	ML, SM, SP-SM, SP > 15% Fines CL with PI<12, OL
Moderate	GM, GC, SP, SC, SC-SM, SW>15% Fine CL with PI>12
Low	GW, GP, GC, SW<15% Fines CH, MH, OH

NOTE: MH, CH, OH, OL, and Peat Soil Types not recommended for SRW construction.

and plots as a straight line on a τ - σ_n diagram (**Figure 3-7**). The quantity **c** is the intrinsic cohesion of the soil sample that is independent of normal stress, σ_n, and **tan** ϕ is the coefficient of peak friction. The convention in geotechnical engineering is to describe the frictional part of the peak soil shear strength by the peak internal friction angle, ϕ, measured in degrees. This angle is the slope of the failure envelope plotted on **Figure 3-7**.

In order for a Mohr-Coulomb model to be accurate the normal stress must be the effective normal stress (σ'_n) acting on the failure plane. The effective normal stress is the total normal stress less the porewater pressure u acting at same location, hence:

[Eq. 3-8]

$$\sigma'_n = \sigma_n - u$$

For routine structures in which the groundwater table is not present within a depth of 0.66H of the height of the wall below the footing, nor where significant groundwater flow into the reinforced zone is expected, the distinction between effective and total stresses is not required (i.e., $u = 0$) and the terms may be used interchangeably. If porewater pressures are present in the analysis, the designer should engage a qualified geotechnical engineer who is familiar with the site conditions to perform analysis and design calculations and to select appropriate drainage systems.

The values for c and ϕ should be determined from the results of direct shear tests (ASTM D 3080, AASHTO T-236) for granular soils or standard triaxial compression tests (ASTM D 4767, AASHTO T-234) for granular and cohesive soils carried out using normal pressures that are representative of site conditions.

Note that in this manual the soil shear strength parameters used in design are peak strength values and are not reduced by application of a reduction factor nor related to the critical void ratio.

Testing of site specific soils is recommended for definition of strength parameters for final design. In the absence of project specific soils testing, the shear strength parameters may be estimated for preliminary design purposes. Typical peak shear strength values for a variety of compacted soils are given in **Table 3-4**.

Table 3-4

Summary of Friction Angle Data for Use in Preliminary Design (Ref. 17)	
Classification	**Friction Angle**
Silt (nonplastic)	26 - 30°
Uniform Fine to Medium Sand	26 - 30°
Well-Graded Sand	30 - 34°
Sand and Medium Gravel	32 - 36°

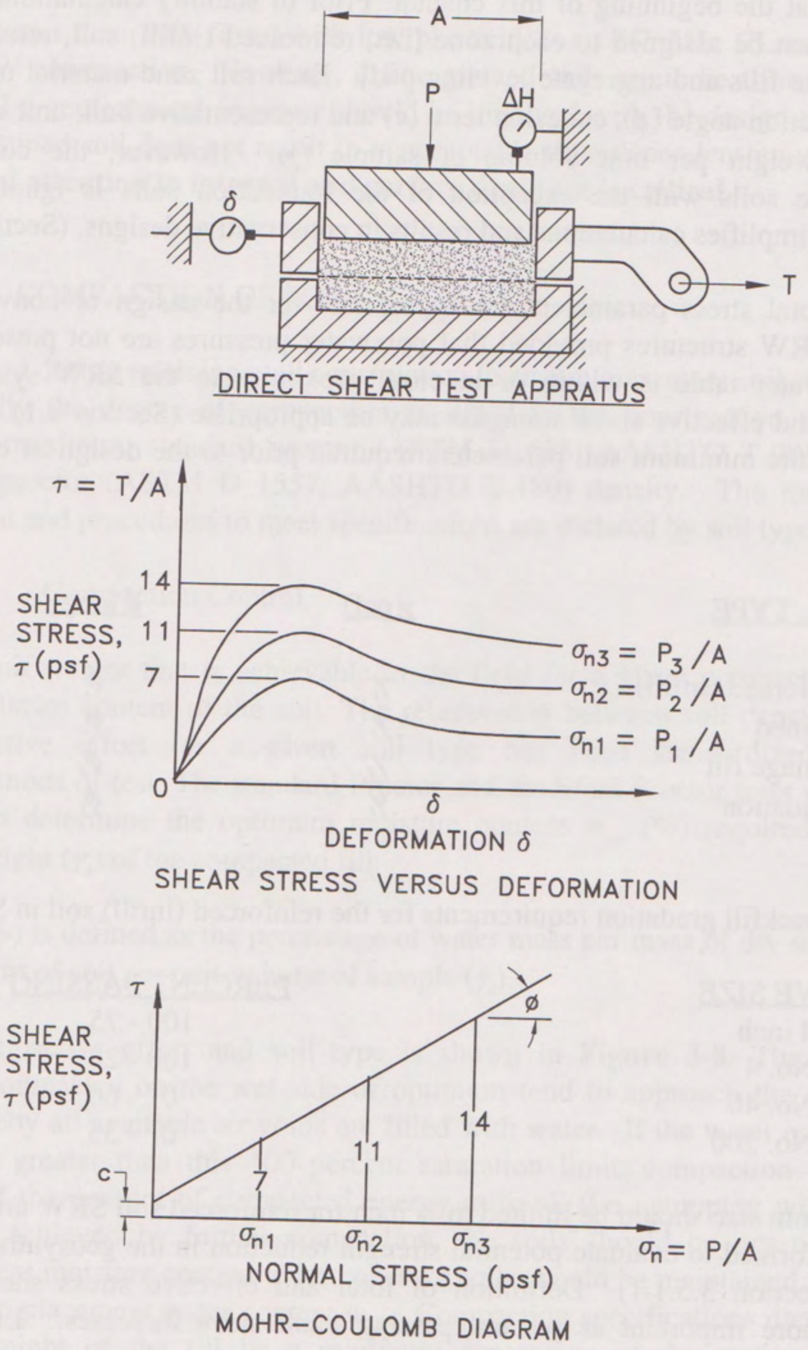


FIGURE 3–7: SHEAR STRENGTH OF SOILS

3.4.3 SOIL PROPERTIES REQUIRED FOR DESIGN OF SRWs

The principal soil components associated with the analysis and design of SRW systems are summarized at the beginning of this chapter. Prior to stability calculations, representative soil properties must be assigned to each zone [i.e., reinforced (infill) soil, retained soil, foundation soils, drainage fills and aggregate leveling pad]. Each soil zone material must be characterized by a peak friction angle (ϕ), cohesion term (c) and representative bulk unit weight defined as the moist unit weight per unit volume of sample (γ). However, the cohesion term for all representative soils with the exception of the foundation soils is ignored for design. This assumption simplifies calculations and results in conservative designs. (Section 3.4).

Generally, total stress parameters are appropriate for the design of conventional gravity and reinforced SRW structures provided that porewater pressures are not present. However, when the groundwater table is within or in close proximity to the SRW system, effective stress parameters and effective shear strengths may be appropriate (Section 8.1). The following table summarizes the minimum soil parameters required prior to the design of each wall section at a project site:

SOIL TYPE	γ(pcf)	ϕ (deg)	c (psf)
Reinforced (Infill)	γ_i	ϕ_i	NA
Retained	γ_r	ϕ_r	NA
Drainage fill	γ_d	ϕ_d	NA
Foundation	γ_f	ϕ_f	c_f

Suggested backfill gradation requirements for the reinforced (infill) soil in SRWs are:

SIEVE SIZE	PERCENT PASSING
4 inch	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 35

The maximum size should be limited to ¾ inch for reinforced soil SRW unless tests have been or will be performed to evaluate potential strength reduction in the geosynthetic due to installation damage (Section 3.5.1.1) Definition of total and effective stress shear strength properties becomes more important as percent passing #200 sieve increases. Likewise, drainage and filtration become more critical.

The plasticity of the fine fraction of the reinforced (infill) soil should be less than 20.

Granular soils are recommended as the reinforced (infill) soil for SRWs because; these soils are easier to place and compact than fine grained soils; have higher permeabilities than fine grained soils which assists in drainage; have greater shear strength than fine grained soils; and are generally less susceptible to creep.

Fine grained soils(greater than 50% fines) with low plasticity (i.e., SC, ML, CL with PI $\leq$ 20) may be used for SRW construction. However, if fine grained soils are to be considered for the reinforced (infill) soil a geotechnical engineer should be involved with the design to ensure that the use of the fine grained soil does not result in unacceptable time-dependent movement of the SRW system. **Special attention to internal and surface drainage is critical.**

3.4.4 COMPACTION OF SOILS

Soil fills are compacted during retaining wall construction to ensure maximum soil shear strength and stiffness. Usually the degree of compaction is stated in the construction specification, typically at 95% of maximum standard proctor (ASTM D 698, AASHTO T-99) or 90% of maximum modified proctor (ASTM D 1557, AASHTO T-180) density. The most effective compaction equipment and procedures to meet specifications are dictated by soil type.

3.4.4.1 Compaction Control

The maximum dry unit weight that is achievable in the field for a given compactive effort is controlled by the moisture content of the soil. The relationship between soil density, moisture content and compactive effort for a given soil type has been standardized based on ASTM/AASHTO methods of test. The standard Proctor and modified Proctor tests use different compactive energy to determine the optimum moisture content w_{opt} (%) required to achieve maximum dry unit weight (γ_d) of the compacted fill.

Moisture content w (%) is defined as the percentage of water mass per mass of dry soil. Dry unit weight is the dry weight of soil per unit volume of sample (γ_d).

The influence of compactive effort and soil type is shown in **Figure 3-8**. The figure also illustrates that soils compacted on the wet side of optimum tend to approach the 100 percent saturation limit, whereby all available air voids are filled with water. If the water content at the time of placement is greater than this 100 percent saturation limit, compaction will not be achieved regardless of the amount of compacted energy utilized, (i.e., pumping will occur). If soil density is to be achieved by further compaction, the soils should be dry of optimum. Generally, the placement moisture content w for cohesive soils should be maintained within +1% to -3% of the optimum placement water content w_{opt}. Compaction specifications dictate that the measured dry unit weight of the fill be a minimum percentage of the maximum density determined from one of the methods of test identified above. Compaction to the specified dry unit weight is critical to achieve the desired shear strength properties of the fill. Completed compaction lifts should not exceed the height (H_u) of SRW unit or 12 inches, whichever is smaller. For SRW unit heights less than 6 inches use multiples of unit height that create

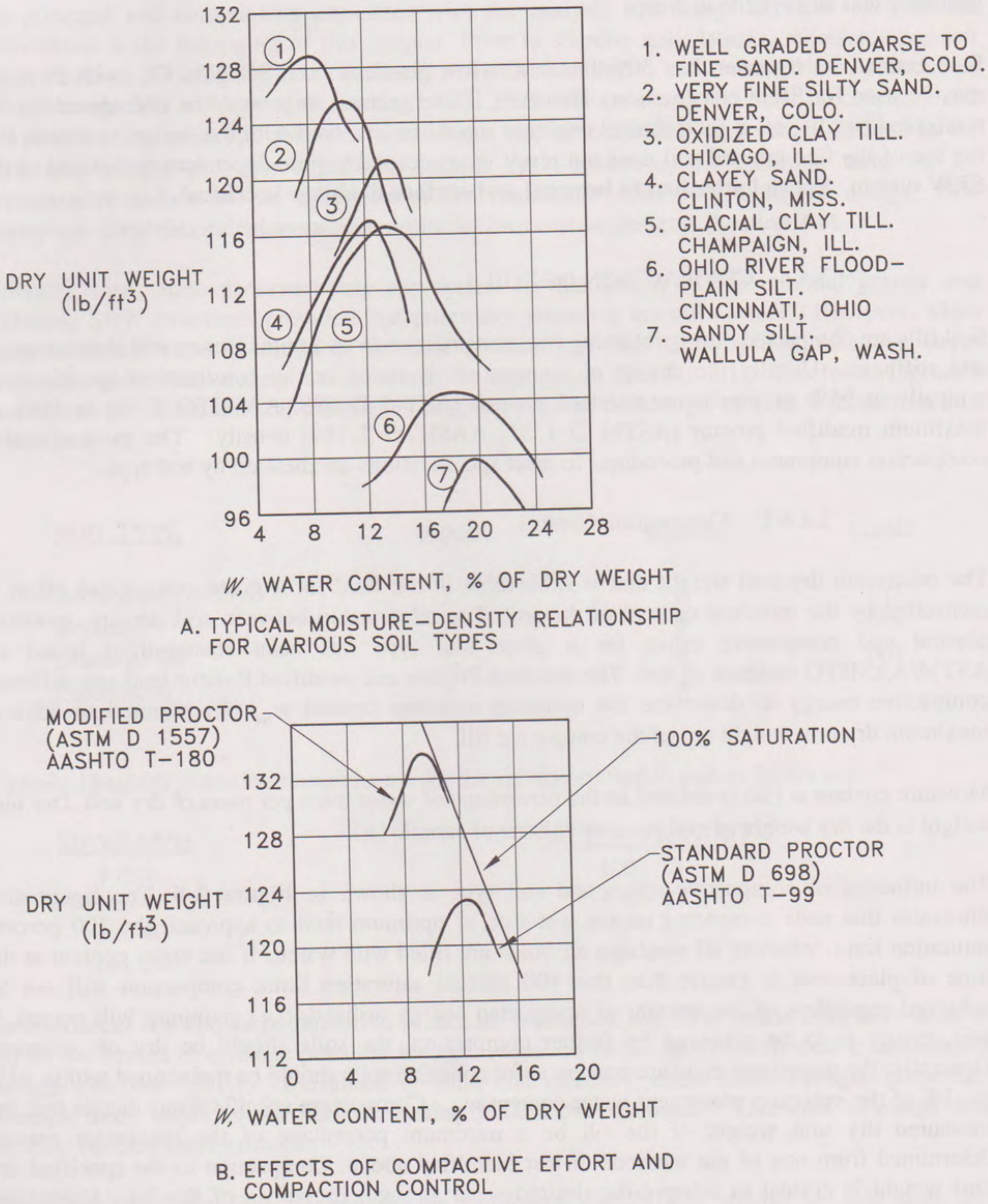


FIGURE 3-8: MOISTURE-DENSITY RELATIONSHIPS FOR SOILS
(after PECK, HANSON, & THORNBURN, 1974 [Ref. 20])

completed compaction lifts between 6 and 12 inches. SRW design and performance are based on the minimum peak internal friction angle, as measured during laboratory testing, being available in the fill materials. Insufficient compaction may lead to less shear strength and result in unsafe SRW performance. Additionally, poorly compacted soils may creep over time with water infiltration and, lead to unacceptable deformations.

3.4.4.2 Compaction Equipment

Generally, for granular soils, [i.e., composed mainly of gravels (G) and sands (S)], the most effective compaction method is vibration. Vibratory compactors consist of drum rollers and steel plates that oscillate at a high frequency as they pass over soil layers. The grain size distribution and particle shape will greatly affect the in-place density that can be achieved, as shown in **Figure 3-8**.

For cohesive soils [i.e., silts (M) and clays (C), sands/silt or sand/clay mixtures] kneading type compaction equipment using sheepsfoot or pneumatic-tired rollers is most effective. The key to achieving good compaction in cohesive soils is proper control of the water content w during placement. Care must be exercised during use of this equipment to minimize and/or eliminate potential damage of geosynthetic reinforcement during construction.

3.4.5 COULOMB EARTH PRESSURE THEORY

Earth forces acting on conventional and reinforced soil SRW structures in this manual are calculated using Coulomb earth pressure theory. Coulomb earth pressure theory has been adopted for the following reasons:

- The theory offers a consistent approach for both types of SRW systems and is used for both internal and external stability analyses in reinforced soil segmental retaining wall structures.
- Geometry that includes a wall facing batter and backslope is included explicitly in the theory.
- Coulomb theory allows the influence of interface shear to be included in the calculation of earth pressures. Fully-mobilized interface shear can reduce the magnitude of earth forces developed behind SRW structures.
- The theory is accepted practice for the design of conventional cast-in-place concrete gravity retaining wall structures with inclined faces [Ref. 39].
- A large body of empirical evidence is available in published literature that shows that Rankine active earth pressure theory over-estimates the internal forces acting on geosynthetic reinforced soil retaining walls [Refs. 34,35,36,40,43,49,51]. Furthermore,

conventional Rankine theory cannot directly account for the typical condition of an inclined wall face and concrete-soil interface friction behind the facing units that is characteristic of segmental retaining wall structures, and tends to reduce applied earth pressure.

Coulomb theory assumes the soil adjacent to the sloped or vertical wall face is at a state of limiting equilibrium along a planar failure surface propagating from the heel of the wall into the retained soil mass. In other words, the soil is at a condition of incipient failure and the shear resistance (either JK internally or MN externally in **Figure 3-9**) is described by a Mohr-Coulomb failure criterion (Equation 3-7) using shear strength parameters (c = 0, $\phi > 0$), at every location on the failure surface (**Figure 3-9**).

The state of incipient collapse for the wedge of soil immediately behind the wall (either IJK for non-reinforced walls or MNP for reinforced walls, **Figure 3-9**) is assumed to be developed by a small outward movement (yielding) of the drystacked column of SRW units or the reinforced soil mass, respectively. The soil behind the wall is assumed to be in an "active state" due to outward movement of the wall with respect to the infill and/or retained soils. The active state represents the condition of the soil that gives the minimum possible lateral earth pressure on a retaining wall structure.

Based on classical soil mechanics principles, the soil in front of the wall may be assumed to be in a "passive state" where incipient failure of the soil in front of the wall (if any) has been caused by the outward movement of the SRW structure into the soil mass (i.e., lateral compression). The passive state represents the condition of the soil that gives the maximum possible lateral earth pressure on a retaining wall structure. The influence of passive earth pressures on the stability of reinforced and unreinforced SRW structures is conservatively ignored. The reason for this is the depth of the soil in front of the wall structure might not be present over the design life of the structure. In addition, neglecting passive earth pressure simplifies stability calculations while resulting in a structure that is marginally more conservative (i.e., slightly lower factors of safety for external failure modes than would be the case if passive earth pressures were considered).

Coulomb theory is used to relate the lateral earth pressure to vertical pressure for the "active" case as illustrated by the following expression:

[Eq. 3-9]

$$\sigma_a = K_a \ \sigma_v$$

where:

σ_a = the lateral earth pressure for active state below the surface of the soil mass.
σ_v = the vertical pressure at depth z below the surface.
K_a = coefficient of active earth pressure.

The vertical pressure due to soil self-weight σ_v (commonly referred to as overburden pressure) at any location in SRW structures should be calculated as:

[Eq. 3-10]

$$\sigma_v = \gamma z$$

where z is the depth below the surface of the soil mass (i.e., top of wall) and γ is the unit weight of soil.

In general, the orientation of the lateral active earth pressure calculated using Equation 3-9 is not horizontal but is included at some angle δ to the surface against which earth pressures act (**Figure 3-9**). The orientation of earth pressures and forces is discussed in the following section.

3.4.5.1 Active Earth Pressure

The coefficient of active earth pressure K_a is calculated as:

[Eq. 3-11]

$$K_a = \frac{\cos^2(\phi + \omega)}{\cos^2\omega \cos(\omega - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \beta)}{\cos(\omega - \delta)\cos(\omega + \beta)}}\right]^2}$$

The quantity ϕ in this equation is the peak internal friction angle of the reinforced and/or retained soil. δ is the interface friction angle and ω is the wall facing batter. The backslope angle β is taken with respect to the horizontal and is positive in a counter-clockwise direction (**Figure 3-9**).

The calculation of coefficient K_a according to Equation 3-11 assumes that fully mobilized shear stresses may develop along the (interface) surface upon which active earth pressures act due to relative shear movement of the soil particles against the surface (PN and IK, **Figure 3-9**). The result of fully-mobilized interface shear is that active pressures will act at angle δ with respect to the horizontal.

Conventional geotechnical engineering practice is to assume the relative soil movement is downward for the active earth pressure condition and consequently the angle α has a positive (counter-clockwise) orientation as illustrated in Figure 3-9.

For the case of $\delta = \beta$ and a vertical wall surface ($\omega = 0$), the coefficient of active earth pressure becomes:

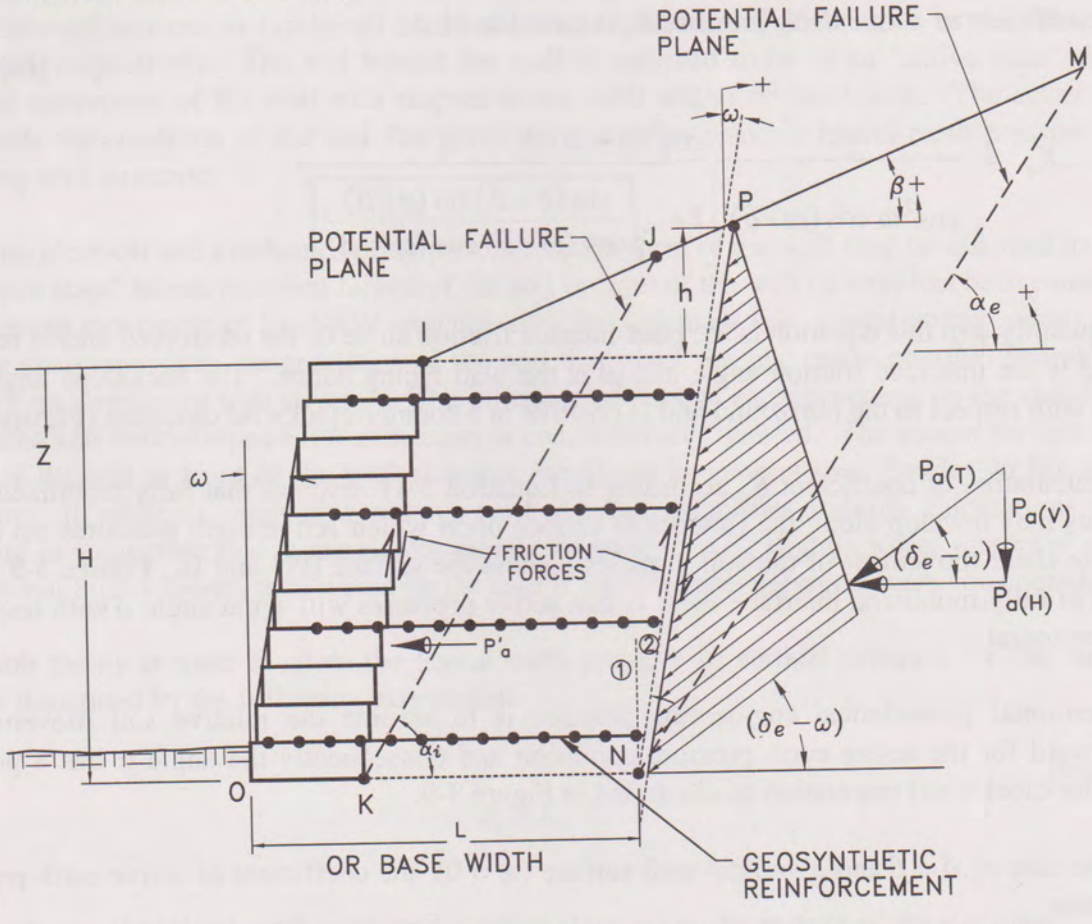
[Eq. 3-12]

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

which is the classic Rankine earth pressure equation.

NCMA ANALYTICAL ASSUMPTION

1. $P_{a(T)}$ BASED ON COULOMB EARTH PRESSURE THEORY.
2. WALL FRICTION δ_e = LOWER ϕ_i OR ϕ_e; $\delta_i = 2/3\,\phi_i$
3. $P_{a(H)} = P_{a(T)} \cos(\delta_e - \omega)$
   $P_{a(V)} = P_{a(T)} \sin(\delta_e - \omega)$
4. P_a BASED ON EXPANDED STRUCTURE HEIGHT (H+h).
5. ACTIVE EARTH PRESSURE FOR INTERNAL STABILITY P'_a CALCULATED IN SAME MANNER BUT BASED ON STRUCTURE HEIGHT (H).



NOTE: ① VERTICAL WALL FACE FOR EXTERNAL STABILITY OF MULTIPLE DEPTH SRWs.
② INCLINED WALL FACE FOR EXTERNAL STABILITY SOIL REINFORCED SRWs.

FIGURE 3–9: EARTH PRESSURE DISTRIBUTION AND FORCE RESOLUTION SRW SYSTEMS

For the case of a horizontal backslope ($\beta = 0$, $\delta = 0$) and a vertical wall surface ($\omega = 0$), the coefficient of active earth pressure becomes:

[Eq. 3-13]

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

Equations 3-11 and 3-12 are restricted to $\beta < \phi$ which in practical terms avoids the possibility of translational failure of the retained soils above the top of wall. The sign convention adopted in this manual is that the facing angle, (ω) is taken with respect to the vertical and is positive in a clockwise direction (i.e., rotation into wall fill soils, **Figure 3-9**).

3.4.5.2 Failure Surface Orientation, α

The orientation α of the critical Coulomb failure surface with respect to the horizontal may be determined using the following equation [Ref. 25].

[Eq. 3-14]

$$\tan(\alpha - \phi) = \frac{-\tan(\phi - \beta) + \sqrt{\tan(\phi - \beta)\,(\tan(\phi - \beta) + \cot(\phi + \omega))(1 + \tan(\delta - \omega)\cot(\phi + \omega))}}{1 + \tan(\delta - \omega)(\tan(\phi - \beta) + \cot(\phi + \omega))}$$

Equation 3-14 can be used to calculate both the internal α_i and external α_e failure plane orientation using the appropriate ϕ_i, δ_i and ϕ_e, and δ_e, respectively.

The evaluation of anchorage capacity is important to maintaining a monolithic gravity mass in the tied-back wedge (see IJK, **Figure 3-9**) method of analysis.

For the case of a horizontal backslope $\beta = 0$, $\delta = 0$ and a vertical wall surface $\omega = 0$ the orientation of the critical failure surface reduces to the Rankine solution:

[Eq. 3-15]

$$\alpha = 45 + \phi/2$$

3.4.5.3 Interface Friction Angle, δ

Movement between masses along a surface generates shear stresses due to friction. Incorporation of these friction forces in determining the Coulomb active earth pressure and orientation of earth forces has been illustrated above and represented as a friction angle, δ. Determination of this friction angle can be done by either field or laboratory testing. However, this is often cost prohibitive and the interface friction angles can be estimated from literature [Refs. 12, 17, 25], as follows:

[Eq. 3-16]

$$\delta_e = \text{the lesser of } \phi_i \text{ or } \phi_r$$

[Eq. 3-17]

$$\delta_i = 2\phi_i/3$$

The external interface friction angle δ_e shall be used for external stability and the internal interface friction angle δ_i for internal stability of both reinforced (infill) soil and multiple depth (crib) conventional SRWs. Whereas δ_i should be utilized in all analyses for single depth conventional gravity SRWs.

Tabulated in Appendix D are the Coulomb earth pressure coefficient K_a and failure surface orientation α for a wide range of ϕ angles and the two standard (i.e., ϕ and $2\phi_i/3$) interface friction angles, δ.

The basis for incorporation of δ was to accurately reflect the earth force magnitude and orientation. The influence of the vertical component $P_{a(V)}$ of the active earth pressure $P_{a(T)}$ will be conservatively ignored in this manual. To ensure this assumption is correct and the analytical approach presented in this NCMA manual is applicable, the following requirements must be met.

[Eq. 3-18]

$$\omega < \delta_i \text{ and } \omega < \delta_e$$

3.4.5.4 Broken Back Slope

The active earth pressure, for the case where the length of the slope above the top of the wall is less than two times the length of the wall, will be calculated based on a slope angle β'. **Figure 3-10** demonstrates how β' is determined.

The coefficient of active earth pressure for this case becomes:

[Eq. 3-19]

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

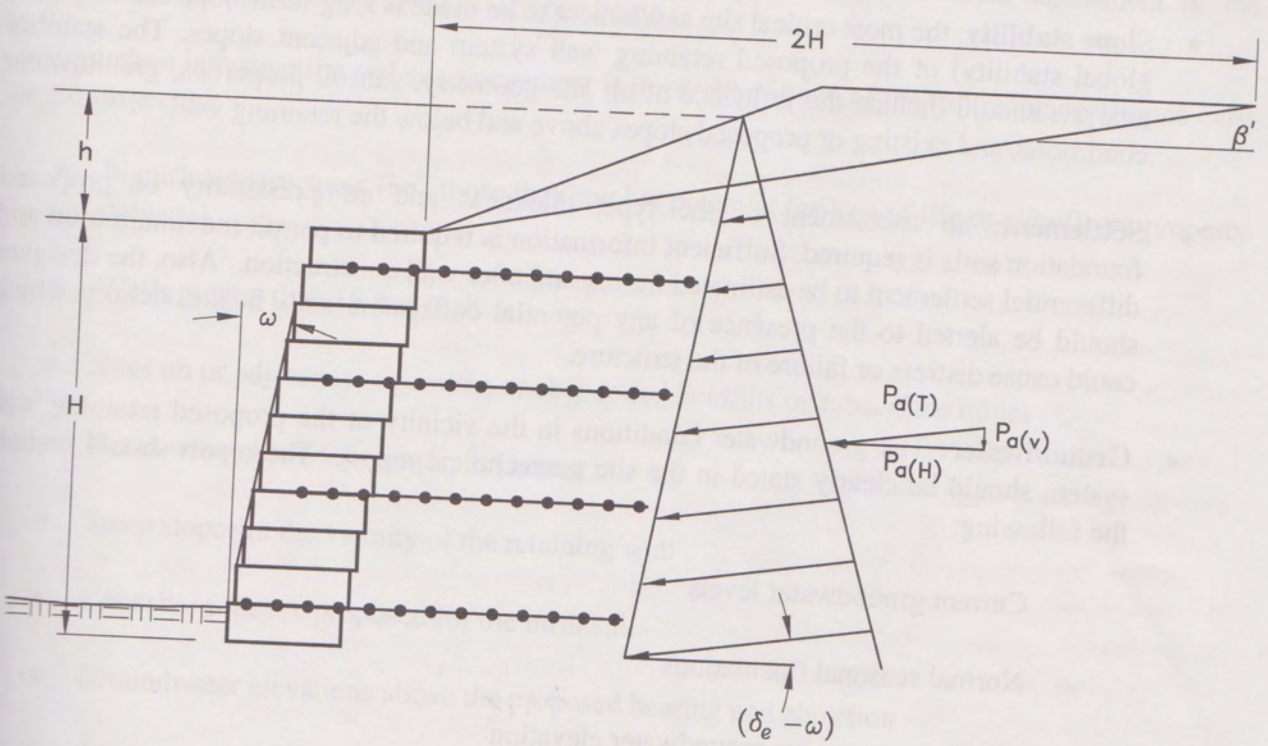


FIGURE 3-10: EARTH PRESSURE – BROKEN BACK SLOPE

3.4.6 GEOTECHNICAL ASSESSMENTS

A qualified geotechnical engineer familiar with the project site should evaluate all engineering data and provide recommendations and/or an assessment of the following items:

- **Slope stability**: the most critical site assessment to be made is long-term slope stability (i.e., global stability) of the proposed retaining wall system and adjacent slopes. The stability analysis should include the influence of all site geometry, subsoil properties, groundwater conditions, and existing or proposed slopes above and below the retaining wall.

- **Settlement:** an assessment of the type, thickness and compressibility of proposed foundation soils is required. Sufficient information is required to permit maximum total and differential settlement to be estimated during and after wall construction. Also, the designer should be alerted to the presence of any potential collapsible soils in the vicinity which could cause distress or failure of the structure.

- **Groundwater:** The groundwater conditions in the vicinity of the proposed retaining wall system should be clearly stated in the site geotechnical report. The report should include the following:

  - Current groundwater levels
  - Normal seasonal fluctuations
  - Maximum previous groundwater elevation
  - Regional groundwater flow
  - Probable maximum projected groundwater elevation
  - Probable influence of proposed site grading changes on groundwater flow and elevation
  - Identification of the potential for hydrostatic pressures or seepage forces on or within the soil retaining wall structure

- **Hazardous Materials:** Existing or potential hazardous materials, unnatural chemicals or microbiological activity at the site should be identified. The areal extent and concentration of these materials should be established and provided to the designer.

- **Seismic:** Potential seismic activity should be identified. A design horizontal and vertical acceleration should be recommended if applicable.

The assessments itemized above can be made independently of the final selection of a retaining wall structure for any site. The relative difference in applied stresses between different types of gravity earth retaining structures of equal height is small. Therefore, subsurface investigation and laboratory testing may proceed independently of the final structure selection.

Many SRW projects require a separate soil investigation and geotechnical assessment of the proposed site before retaining wall design can begin.

A subsurface investigation and site assessment is imperative if any of the following conditions exist or are suspected:

- Significant structures (i.e., those that involve potential for loss of life or significant property damage)
- Walls greater than 15 feet high
- Sites on or adjacent to operating or abandoned landfills or subsurface mines
- Areas where karst topography is present
- Steep slopes in the vicinity of the retaining wall
- Cohesive soils are proposed for the infill soil
- Groundwater elevations above the proposed bearing pad elevation
- Seismic activity in area

The geotechnical engineer of record should be required to provide all necessary information if any of the above information is omitted in the site geotechnical report.

Section 3.5
GEOSYNTHETIC REINFORCEMENT

For reinforced soil SRWs, geosynthetic reinforcement is placed in horizontal layers to unify the mass of the composite SRW structure and thereby increase the resistance of the system to destabilizing forces generated by retained soils and surcharge loads. The mass of soil that is reinforced in this manner is called the reinforced (infill) soil zone.

To create the composite structure the reinforcement layers must be of sufficient number, possess adequate tensile strength, and develop sufficient anchorage capacity to hold the composite mass (reinforced (infill) soil zone) together. A "tied-back wedge model" is used in this manual to analyze the reinforced (infill) soil mass for internal stability. In this model, the geosynthetic reinforcement layers are assumed to provide a tensile force to resist the outward movement of the reinforced soil wedge IJK illustrated in **Figure 3-9**. The orientation of the plane defining the limits of the failure wedge behind the wall facing is defined by angle α_i and calculated using Equation 3-14. The area within the reinforced soil (infill) zone beyond the failure wedge (IJK, **Figure 3-9**) is designated the anchorage zone for geosynthetic reinforcement.

Calculation of α_i using Equation 3-14 is consistent with the Coulomb wedge theory adopted in this manual to calculate active earth pressures. The results of instrumented geosynthetic reinforced soil retaining walls show that the location of the potential failure surface propagating up into the reinforced (infill) soil zone from the heel of the bottom SRW unit may be planar, log spiral or bilinear in shape and falls within the failure plane predicted using Equation 3-14 [Refs. 36, 40, 42, 51]. The most common failure surface used in tied-back wedge models is the theoretical Rankine surface (Equation 3-15) [Refs. 19,39], which yields greater α_i angles than Equation 3-14. Therefore, calculation of α_i using Equation 3-15 may underestimate the lateral extent of the failure zone within the reinforced soil zone. However, the influence of α_i on reinforcement lengths is typically limited to the topmost layers in the reinforced soil mass. Lower layers will have longer anchorage lengths and greater overburden pressures, thus will not need to be extended past the minimum uniform reinforcement length required to satisfy external stability requirements.

The designer must ensure that for the design life of the structure a candidate geosynthetic reinforcement has adequate tensile capacity within the soil, and at the facing connection plus sufficient anchorage length beyond the potential internal failure wedge to develop sufficient anchorage capacity to resist pullout of the reinforcement from the soil. A method to evaluate a candidate geosynthetic reinforcement material with respect to these criteria is described in the following sections.

3.5.1 LONG-TERM DESIGN STRENGTH

The Long-Term Design Strength ***LTDS***, of a geosynthetic reinforcement is strength at limit equilibrium conditions in the soil. The ***LTDS*** is defined as the strength in the geosynthetic reinforcement at the end of the service life of a reinforced soil SRW at which time all design criteria must be met for the structure to perform as intended. Therefore, in this manual reinforced soil SRWs will be designed for conditions (soils, environment and applied loads) anticipated at the end of the design service life.

The allowable strength (T_a) of the reinforcement is defined as the ***LTDS*** of the geosynthetic divided by a factor of safety to account for uncertainties in geometry of the structure, fill properties, reinforcement properties and external applied loads.

Selection of T_a for geosynthetic reinforcement is complex. The tensile properties of geosynthetics are affected by environmental factors such as creep, installation damage, aging, temperature and confining stress.

Polymeric reinforcements are generally durable materials that will perform for the life of the structure when properly designed. The considerations that are important in evaluating the long-term performance of the reinforcement are degradation due to physiochemical activity in the soil such as hydrolysis, oxidation and environmental stress cracking (depending on polymer type); installation damage and the effects of high temperatures at the facing and connections of SRWs.

Because of varying polymer types, quality, additives, product geometry and manufacturing processes each geosynthetic is different in its resistance to aging. Each product must, therefore, be investigated individually.

3.5.1.1 Determination of the Allowable Strength of the Geosynthetic

The allowable strength of the geosynthetic can be determined using the following procedure. This procedure is similar to that provided in FHWA Publication No. FHWA-DP. 82-1 "Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines." A full description of the procedure is provided in Appendix E.

The long-term design strength of the geosynthetic is determined as follows:

[Eq. 3-20]

$$LTDS = \frac{T_{ult}}{RF_D \cdot RF_{ID} \cdot RF_{CR}}$$

The allowable strength of the geosynthetic is determined as follows:

[Eq. 3-21]

$$T_a = \frac{LTDS}{FS_{UNC}} = \frac{T_{ult}}{RF_D \cdot RF_{ID} \cdot RF_{CR} \cdot FS_{UNC}}$$

where:

T_{ult} = Ultimate (or yield tensile strength) from wide width tensile strength tests (ASTM D 4595 or GRI "GG1: Single Rib Geogrid Tensile Strength"), based on minimum average roll value (MARV) for the product.

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. The typical range is from 1.1 to 2.0.

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

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, and can vary typically from 1.5 to 5.0.

FS_{UNC} = Overall factor of safety or load reduction factor to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads. The typical value is 1.5.

The determination of reduction factors for each geosynthetic product require extensive field and /or laboratory testing, briefly summarized as follows:

- *Durability Reduction Factor*—The protocol for testing to obtain this reduction factor is under current development. In general, it consists of oven aging polyolefins (PP and HDPE) samples to accelerate oxidation and measure their strength reduction, as a function of time and temperature. 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 relatively high temperature to replicate hydrolysis. For guidance on selecting $\boldsymbol{RF_D}$ see Appendix E.

- *Installation Damage Reduction Factor*—Installation damage reduction factor is determined by subjecting the geosynthetic material to a backfill and compaction cycle, consistent with field practice. The ratio of the initial strength, to the strength of retrieved samples defines this reduction factor. A current ASTM standard is under development. For a more detailed explanation of $\boldsymbol{RF_{ID}}$, see Appendix E.

- *Creep Reduction Factor*—This reduction factor is obtained from long term laboratory creep testing as detailed in Appendix E. Creep testing is essentially a constant load test on multiple product samples, loaded to various percentages of the ultimate product load, for periods of up to 10,000 hours. The creep reduction factor is the ratio of the ultimate load to the maximum sustainable load within the design life.

Appendix F provides an alternate procedure (Method "B") for determining the allowable strength of the reinforcement. This procedure generally follows European practice and relies heavily upon the work of Jewell and Greenwood [Ref. 10].

3.5.2 SOIL-REINFORCEMENT INTERACTION PERFORMANCE

Reinforced SRW design requires an estimate of two soil geosynthetic reinforcement interaction coefficients. The coefficient of interaction for pullout, $\boldsymbol{C_i}$, is required for calculations used to estimate the reinforcement pullout capacity within the anchorage zone of the reinforced soil mass. The coefficient of direct sliding $\boldsymbol{C_{ds}}$, is required to estimate the resistance to internal sliding that is generated along the surface of a layer of geosynthetic reinforcement when a portion of the reinforced soil mass slides along its interface.

3.5.2.1 Coefficient of Interaction for Pullout

The coefficient of interaction for pullout, $\boldsymbol{C_i}$, is used to relate the pullout resistance of the geosynthetic reinforcement to the available soil shear strength. The coefficient of interaction can be expressed mathematically as:

[Eq. 3-22]

$$C_i = \frac{R_{po}}{2\,L_e\ \sigma_n\ \tan\phi_i}$$

where:

L_e = horizontal length of geosynthetic in the anchorage zone (ft)
σ_n = normal stress acting over the geosynthetic anchorage length (psf)
ϕ_r = peak angle of internal friction for the reinforced (infill) soil (deg)
R_{po} = maximum pullout resistance (lb/ft)

The coefficient of interaction for pullout for geotextiles or geogrids is determined directly from large scale pullout testing using sample candidate products that are at least 24 inches in length and site specific soils (or similar soils based on USCS) compacted to densities anticipated in the field. Pullout testing procedures should follow GRI "GG-5: Geogrid Pullout" [Ref. 23] short-term loading test method (method "A"), regardless of whether the geosynthetic reinforcement is a geogrid or geotextile. Calculation of pullout capacity requires the peak internal friction angle, ϕ_r, of the anchorage soil be known. The reinforcement sample is placed over the reconstituted soil and then buried. A confining pressure σ_n is applied to the top of the soil sample to simulate the vertical pressure that is anticipated for the anchorage elevation examined.

In this manual the peak pullout capacity R_{po} for laboratory testing is the maximum force required to cause uniform pullout of the "entire" embedded geosynthetic through the soil.

Due to the variety of shapes and product structures in commercially available geosynthetics, it is important to note that C_i will vary between geosynthetic products and may change with magnitude of normal pressure applied to samples of geosynthetic. Therefore, it is important that tests be conducted over the range of confining (normal) pressures anticipated in the field and the appropriate C_i for each reinforcement elevation be utilized.

The typical service life (100 years) of SRWs requires the C_i used in design represent a long-term parameter. To ensure the C_i value determined from pullout testing is unaffected by time dependent properties, the pullout resistance in design should be limited to the creep limited strength of the geosynthetic (T_{ult}/RF_{cr}). In addition, the shear strength parameters for the soil should be adjusted for creep behavior if applicable. However, creep potential of reinforced soil is not a concern if well compacted granular soils are used in the reinforced (infill) soil zone. The potential for creep exists for cohesive soils such as MH, CH, OL, and OH types, and those soil types are not recommended for use with SRWs.

The pullout resistance of geosynthetic reinforcement is mobilized through one or two basic soil-reinforcement interaction mechanisms: 1) interface friction and 2) passive soil resistance developed against the transverse elements of open structure geogrids. The pullout resistance for geotextiles is limited to interface friction. For geogrids, the pullout resistance is a combination of both mechanisms depending upon product structure and extensibility.

For geogrid reinforcement products that rely on passive resistance of the transverse ribs, the long-term integrity of the load transfer mechanism between longitudinal and transverse ribs must be assured. To verify the coefficient of interaction C_i from short-term pullout tests is applicable under long-term load conditions, one of the following tests should be performed:

- Junction strength tests according to test method GRI "GG-2: Geogrid Junction Strength" [Ref. 48]. The sum of the geogrid junction strengths occurring in a one foot square area of geogrid sample must be greater than or equal to the ultimate strength of the geogrid T_{ult};
- Through-the-junction creep testing according to test method GRI "GG-3: Tension Creep Testing of (Stiff and Flexible) Geogrids" [Ref. 45];
- Short-term pullout tests of the geogrid with severed transverse ribs; or
- Long-term pullout tests of the geogrid in project-specific soils.

3.5.2.2 Coefficient of Direct Sliding

Geosynthetic reinforcement layers may create preferred planes of sliding within the reinforced (infill) zone of a SRW structure. The movement of a portion of the reinforced (infill) soil mass across a stationary layer of geosynthetic reinforcement is modeled as a direct shear failure mode in internal stability calculations. The coefficient of direct sliding, C_{ds}, shall be determined in accordance with ASTM D 5321 "Direct Shear Test Methods for Geosynthetic," or GRI "GS-6: Interface Friction Determination by Direct Shear Testing" [Ref. 50]. The test methods require project-specific soils to be placed at field densities above and below samples of the candidate geosynthetic and then sheared along the plane of the geosynthetic. Parameter C_{ds} is to be calculated as follows:

[Eq. 3-23]

$$C_{ds} = \frac{R_{ds}}{L\ \sigma_n\ \tan\phi}$$

where:

R_{ds} = maximum shear resistance from direct shear test (lb/ft)
L = stationary length of geosynthetic (ft)
σ_n = normal stress on geosynthetic sample (psf)
ϕ = peak angle of internal friction of the soil (deg)

The value of C_{ds} can be expected to vary with normal stress and, therefore, the tests should be carried out over a range of confining pressures expected for reinforcement layers in the proposed structure. In the absence of site-specific soil testing, the magnitude of C_{ds} may be estimated from manufacturer's test data using similar soil or soils with lower shear strength than project soils. In no case shall C_{ds} be assigned a value greater than one.

Section 3.6
QUALITY ASSURANCE FOR CONSTRUCTION

A proper quality assurance program at the construction site is essential to ensure correct installation of component materials and satisfactory post-construction performance of the SRW structure.

As part of project specifications, the manufacturer of the geosynthetic should provide to the engineer prior to delivery of any geosynthetic reinforcement a list of Minimum Average Roll

Values (MARV) for geosynthetic index property tests including weight; roll size; grab or single rib tensile strength; aperture opening; rib or yarn size; and/or other quality control tensile strength values. The MARVs can be used for comparison with index properties assumed in design for the geosynthetic reinforcement. To ensure delivery and installation of the appropriate geosynthetic product and type (strength), it is recommended the geosynthetic reinforcement manufacturer provide a manufacturer's certification of the actual strength and index test values listed above with each shipment. In a thorough quality assurance program for significant projects, the geosynthetic reinforcement products shipped to the job site are independently tested for the above properties to ensure the installed product meets or exceeds the design properties assumed in design.

It is recommended the manufacturer of the SRW units provide a manufacturer's certificate as to the compressive strength, water absorption, color and dimensional tolerance of the SRW units delivered to the site. This will ensure knowledge of and compliance with project specifications.

All local building codes and national standards should be followed to ensure all products associated with SRWs are adequately tested to establish conformance. Compaction control of fill materials by a qualified inspector is essential to ensure the satisfactory performance of any SRW structure. It is recommended the owner engage a knowledgeable professional not associated with the installation contractor to perform all quality assurance functions.

End of Section

SECTION 4
CONVENTIONAL SEGMENTAL RETAINING WALL DESIGN

Conventional segmental retaining walls (SRWs) are gravity structures that rely solely on their weight to resist destabilizing forces. There are two categories of conventional SRWs; single depth SRWs and multiple depth SRWs, as shown in **Figure 2-1**. Single depth SRWs are full gravity structures in which the height, width, weight, batter, and shear capacity of the SRW units determines the maximum single column SRW height for a given retained soil and surcharge loading condition. Multiple depth SRWs are formed by interlocking units to create cribs or bins that are filled with free draining gravel adding mass to the structure. Multiple depth SRWs are considered semi-gravity structures because typically the base is wider than the remainder of the structure, with the weight of the fill above the base contributing to the stability of the wall. Multiple depth structures are not specifically addressed herein, but can be designed following the same principals.

To safely determine the maximum height for gravity SRWs the analysis and design steps summarized below and in **Figure 2-8** are recommended:

- Determine SRW unit properties, wall geometry, and soil/groundwater parameters at the site.
- Calculate the driving forces on the structure.
- Calculate factors of safety for external (**Figure 4-1A**) and internal (**Figure 4-1B**) failure modes based on a trial design.
- Adjust wall height to meet minimum factors of safety criteria.
- Check global stability of the SRW system (**Figure 4-1C**).

Section 4.1
DESIGN ASSUMPTIONS

Conventional SRWs are analyzed using standard geotechnical engineering methods for concrete gravity retaining wall structures [Ref. 39] with some refinements that reflect the unique dry-stacked unit construction. Potential failure modes that are analyzed are summarized in **Figure 4-1**.

Coulomb earth pressure theory is used to calculate the earth forces/pressures acting at the base of the dry-stacked column of SRW units for single depth SRWs and internal stability of multiple depth SRWs (**Figure 3-9**). The external stability of multiple depth SRWs should use Coulomb earth pressure theory to calculate earth forces acting on a plane projected vertically from the heel (back) of the base width of the lowest course of SRW units. The Coulomb approach has been adopted in this manual to calculate earth pressures because it explicitly considers the influence of inclined wall facing; sloped backfills; and interface shear between the concrete SRW facing units and retained soils.

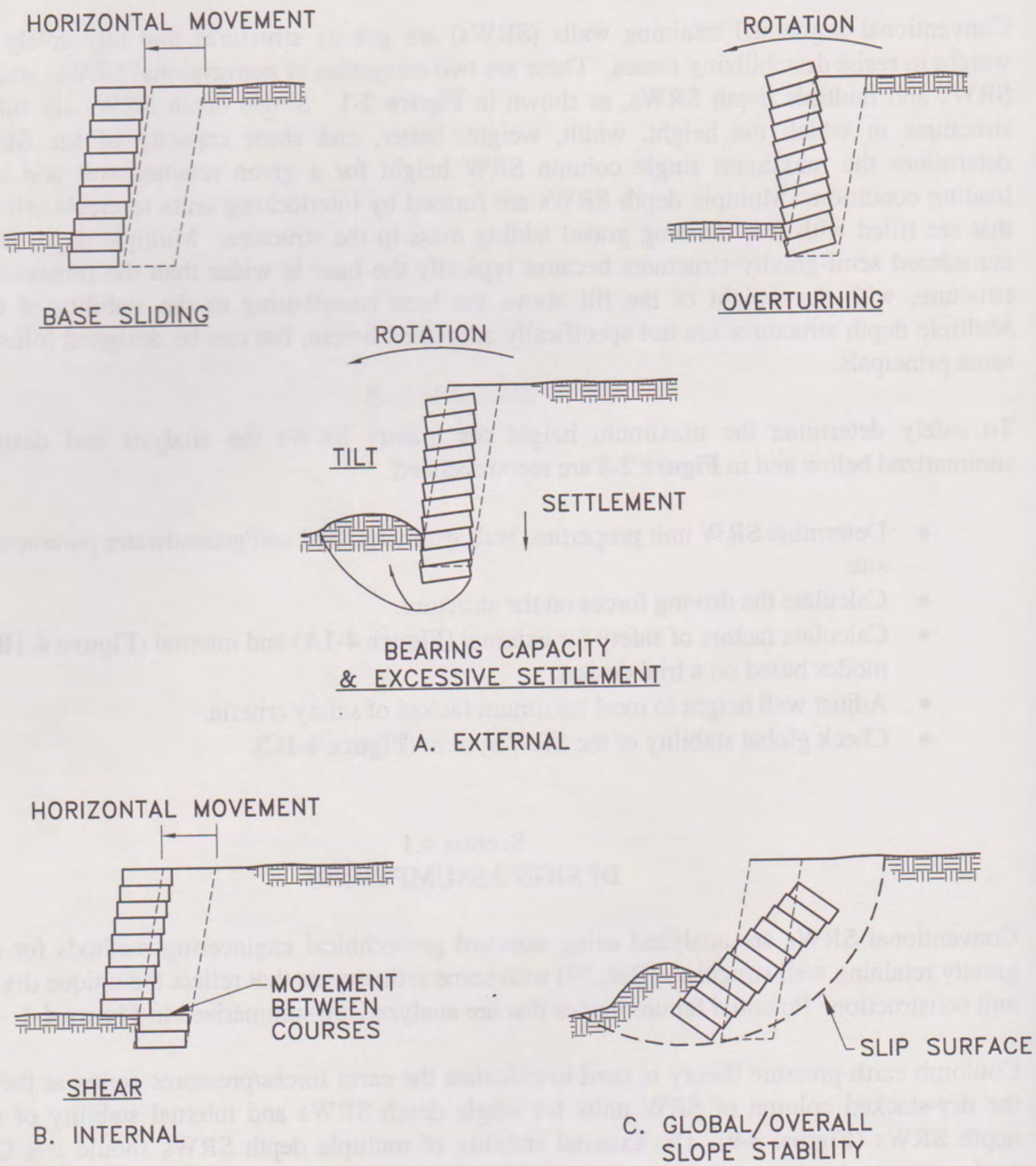


FIGURE 4–1: MAIN MODES OF FAILURE FOR CONVENTIONAL SRWs

Interface friction potential is assumed to be fully mobilized at the back of the concrete SRW units and the drainage fill/retained soils. The magnitude of interface friction capacity is described by angle δ in degrees and is calculated using Equation 3-16 for δ_e and Equation 3-17 for δ_i. The internal interface friction angle δ_i should be utilized in all calculations for single depth SRWs and internal stability of multiple depth SRWs. The external interface friction angle δ_e should be implemented only for external stability of multiple depth SRWs. The mobilized interface friction develops as a result of the relative downward movement of the retained soils with respect to the wall units that are supported on a bearing pad. The relative downward movement is the result of the vertical rigidity of the bearing pad and column of SRW units, the outward rotation of the column of SRW units; and compaction of the soil behind the facing units. However, in order to simplify calculations, only the horizontal component of earth pressures are considered in stability calculations. Neglecting the vertical force component is a conservative assumption. To ensure this is a valid assumption, the wall inclination ω must be less than δ_i and δ_e, (Equation 3-19).

A complete discussion of Coulomb earth pressure theory is given in Section 3.4.5. The design method presented herein to calculate forces on the wall is restricted to purely frictional granular soils (i.e., $c = 0$, $\phi > 0$).

The resisting forces used in stability calculations are assumed to develop through self-weight of the dry-stacked column of SRW units and the fill they contain. The self-weigh of the dry-stacked column generates the inherent shear capacity between SRW courses and sliding resistance at the base of the structure.

The design methodology and the equations presented below assume the groundwater table is well below the SRW bearing pad elevation (i.e., > 0.66H where H is the vertical height of wall). This assumption allows calculation of earth pressures and soil strengths based upon total stresses and eliminates the need to consider additional destabilizing forces due to hydrostatic pressure or groundwater seepage.

Section 4.2
HINGE HEIGHT

The flexible nature of the dry-stack SRW construction and the limited ability of SRW units to transmit moments is accounted for by implementing a maximum height of influence criteria. This influence criteria for the stability analysis of dry-stacked segmental retaining wall units is the "hinge height" concept illustrated in **Figure 4-2**. The hinge height (H_h) is related to the maximum number of SRW units that can be stacked in an isolated column at a facing inclination of ω without toppling.

Toppling of the isolated column will occur when the vertical weight of the column outside the heel of the lowermost unit exceeds the weight of the column inside the heel (i.e., $M_B > M_A$ in **Figure 4-2**). As illustrated in **Figure 4-2** the hinge height is measured vertically.

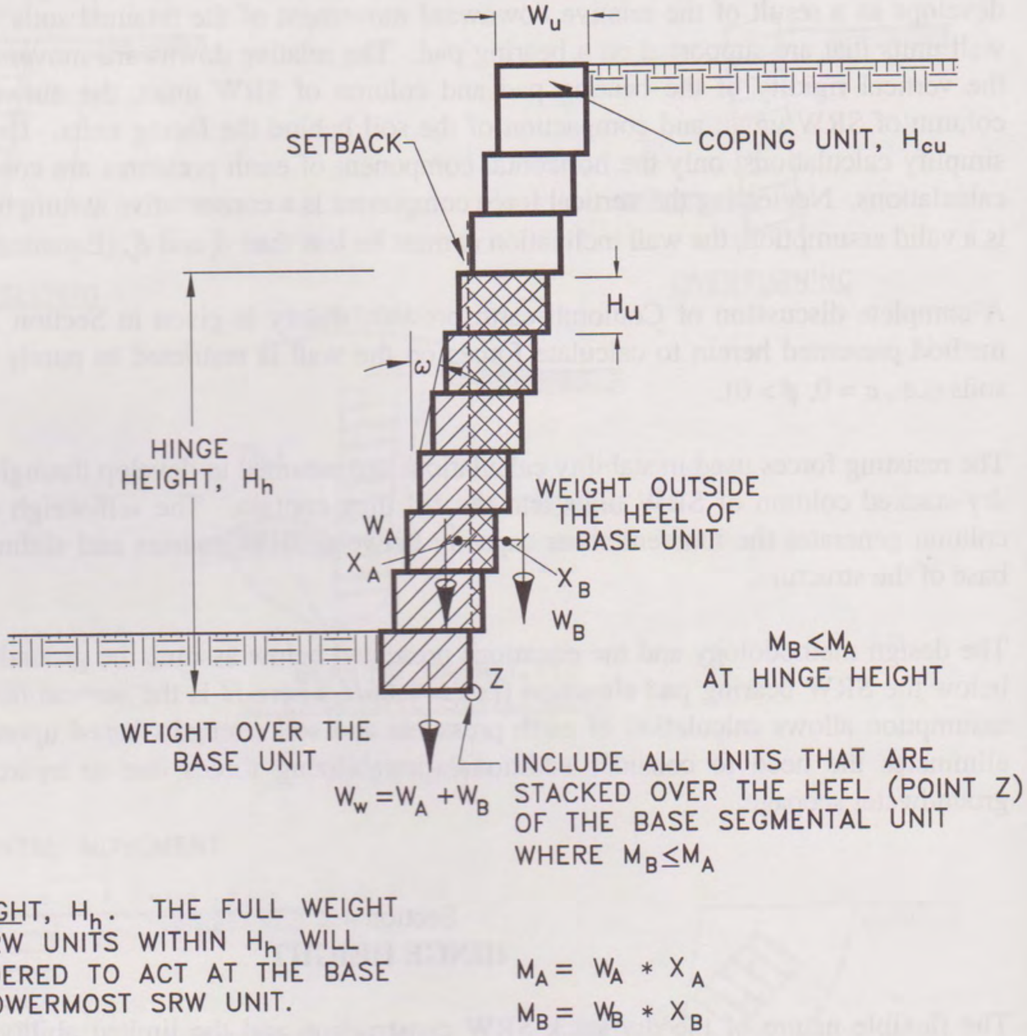


FIGURE 4–2: HINGE HEIGHT FOR SRW DESIGN

The purpose of the hinge height concept in conventional SRW stability analyses is to restrict the maximum design weight of the dry-stacked column of SRW units that can be transferred to the wall base or underlying courses. This concept will also be applied to reinforced soil SRWs in the calculation of facing connection strength and shear capacity.

Without the hinge height definition used in this manual, the magnitude of normal pressure at shear interfaces could be overestimated and the results of conventional SRW external stability analyses could provide wall heights that do not satisfy minimum factors of safety for global/overall slope stability.

For routine structures the hinge height definition will usually result in a wall design that is not controlled by global stability failure mechanisms. Therefore, the number of relatively tedious calculations associated with global stability analyses can be reduced.

The hinge height is determined by summing moments about the heel of the SRW unit at the wall base (point Z in **Figure 4-2**) and when $\omega > 0$ may be calculated as:

[Eq. 4-1]

$$H_h = 2(W_u - G_u)/\tan\omega$$

where:

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

The hinge height H_h will be used as a maximum height criteria in the calculation of weight acting on any SRW unit interface. Whenever, the hinge height is greater than the wall height or it is a true vertical wall, ($\omega = 0$) then use:

[Eq. 4-2]

$$H_h = H$$

where:

- H = total height of wall (ft)

The calculation of hinge height H_h using Equation 4-1 has been shown to be accurate to within one SRW unit height, H_u. This maximum overestimation of H_u is sufficiently accurate for the hinge height concept and simplifies calculations. Additionally, the minor influence of a typical coping (cap) layer is ignored in the calculation of hinge height in order to simplify calculations and partially compensate for the first simplification.

The hinge height concept is also adopted for internal sliding stability analysis of conventional and reinforced soil SRW structures.

Section 4.3
MINIMUM FACTORS OF SAFETY FOR STABILITY ANALYSES

Selection of appropriate factors of safety should be based on the certainty with which design parameters and the consequences of failure are known. **Table 4-1** lists the recommended minimum safety factors for gravity SRWs. Included in these recommended minimums are typical levels of uncertainty in wall geometry and imposed loadings.

TABLE 4-1

Recommended Minimum Factors of Safety for Design of Conventional SRWs

Failure Modes		*FS*
Base Sliding	FS_{sl}	1.5
Overturning	FS_{ot}	1.5
Bearing Capacity	FS_{bc}	2.0
Internal Shear Capacity	FS_{sc}	1.5
Global Stability	FS_{gl}	1.3 - 1.5

Section 4.4
SEGMENTAL UNIT PROPERTIES

Dimensions and mechanical properties of segmental units must be established prior to design. These parameters are:

H_u = SRW unit height (ft)
H_{cu} = SRW unit cap (coping) height (ft)
W_u = SRW unit width (ft)
γ_u = weight of segmental unit per unit volume as placed (includes stone fill if applicable) pcf)
G_u = distance to center of gravity of horizontal SRW unit, including drainage fill, measured from the front face of the unit (ft)
ω = wall batter due to segmental unit setback per course (deg)
μ_b = interface friction coefficient for base segmental unit sliding on bearing soils
a_{su} = apparent minimum shear capacity between segmental units (lb/ft)
λ_u = apparent angle of friction between segmental units (deg)
Δ_U = setback per course H_u (in.)

Section 4.5
EXTERNAL STABILITY

External stability modes of failure for overturning, base sliding and bearing capacity are illustrated in **Figure 4-1A**. All external stability calculations shall be performed on a wall section of unit length (1 ft). All forces in stability analyses are expressed as force per unit length of wall (lb/ft) and moments as force-length per unit length of wall (lb-ft/ft). The calculations described herein are specifically for single depth conventional SRWs. Similar calculations can be performed on multiple unit (crib) walls by making the modification described in Section 4.1 and using procedures in Section 5.5.

4.5.1 EARTH PRESSURES AND FORCES

The distribution of the horizontal component of earth pressures and forces due to the infill or retained soil self-weight and surcharge loadings which are assumed to act directly on the back of single depth conventional SRWs is shown in **Figure 4-3**. Since the interface friction angle between the back of the SRW units and the retained soils is assumed to not be zero (i.e., $\delta_i = 2\phi_i/3$ Equation 3-17), the earth pressures and forces do not act perpendicular to the back of the dry-stacked column of wall units. To ensure that wall inclination does not result in an upward component of lateral earth pressures which will reduce horizontal sliding resistance and resisting moments in external stability calculations, the following criterion must be satisfied: $\omega < \delta_i$, Equation 3-18. This criterion is satisfied for most gravity wall structures since wall inclination is typically restricted to $\omega < 15°$, which is less than the value of δ_i (Equation 3-17) calculated for good quality granular materials in contact with concrete surfaces. To simplify calculations only the horizontal component of earth pressures and forces are considered in stability calculations for conventional SRWs described in this manual. This assumption results in a more conservative design.

The magnitude of the horizontal component of lateral earth pressures at depth **z** below the crest of the wall is calculated as:

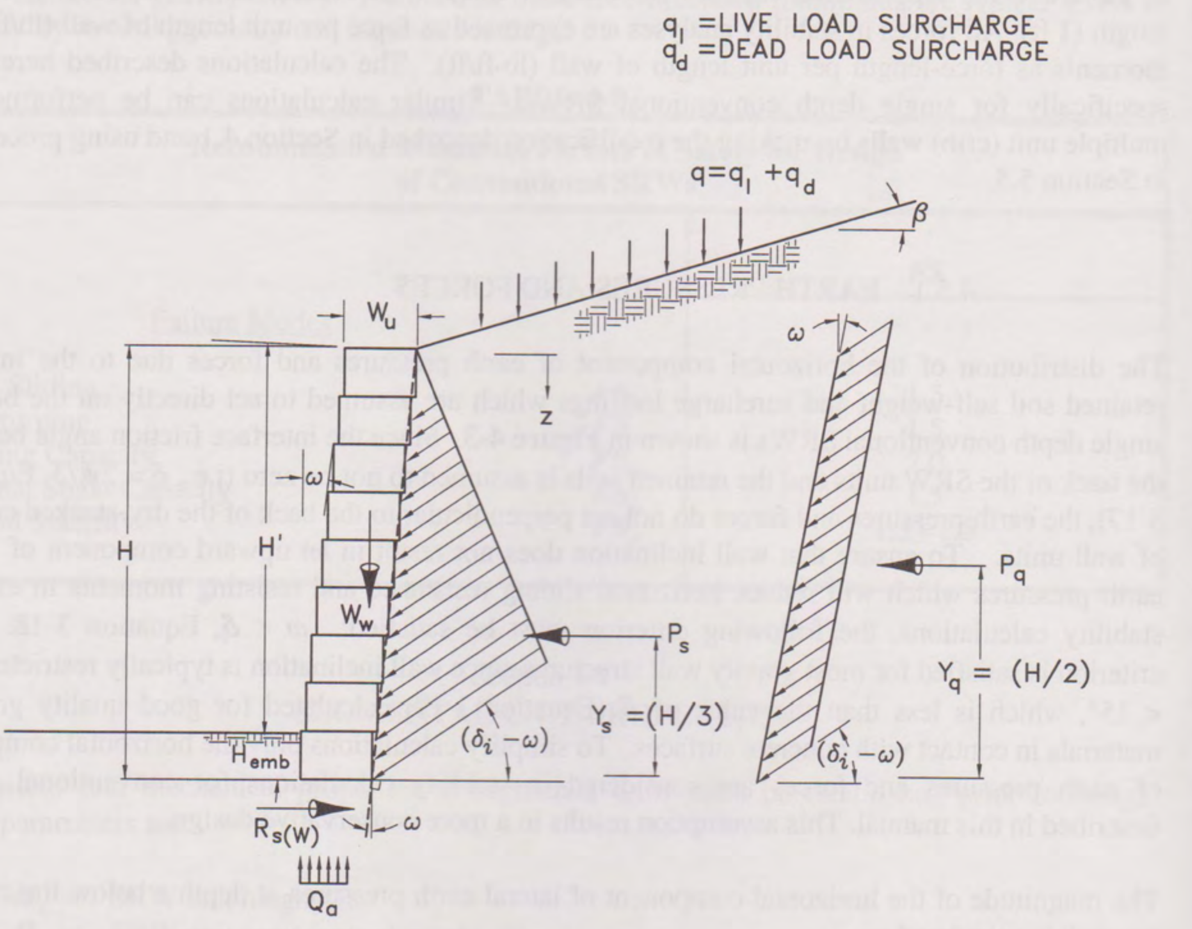
[Eq. 4-3]

$$\sigma a_z = (\gamma_i (K_a) \cos(\delta_i - \omega)\ z) + ((q_l + q_d)\ K_a \cos(\delta_i - \omega))$$

The Coulomb active earth pressure coefficient K_a is calculated using Equation 3-11 as shown below:

[Eq. 3-11]

$$K_a = \frac{\cos^2(\phi_i + \omega)}{\cos^2\omega \cos(\omega - \delta_i)\left[1 + \sqrt{\dfrac{\sin(\phi_i + \delta_i)\sin(\phi_i - \beta)}{\cos(\omega - \delta_i)\cos(\omega + \beta)}}\right]^2}$$



$P_a = P_s + P_q$

K_a = USING COULOMB EQUATION 3-11 AND REINFORCED (INFILL) SOIL PROPERTIES

$$P_s = 0.5 K_a \gamma_i H^2 \cos(\delta_i - \omega)$$

$$P_q = K_a q H \cos(\delta_i - \omega)$$

FIGURE 4-3: FORCES AND GEOMETRY FOR EXTERNAL STABILITY ANALYSIS OF CONVENTIONAL SINGLE DEPTH SRWs

Furthermore, selection of the peak friction angle ϕ to use in the above equation is based on the properties of the weakest dominant soil material (ϕ_i or ϕ_r) located in the distance of up to $H/2$ behind the segmental retaining wall structure. The term β is the backslope angle from the horizontal in degrees and δ_i is the mobilized interface friction angle in degrees.

In the scenario when a wall is vertical with no backslope (i.e., $\beta = 0$ and $\omega = 0$) and the designer wishes to conservatively ignore interface friction (i.e., δ_i=0) the coefficient of active earth pressure simplifies to Equation 3-3.

[Eq. 3-13]

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$$

The total horizontal active earth force $\mathbf{P_a}$ is calculated by integrating the earth pressure expression described by Equation 4-3 over the wall height H to give:

[Eq. 4-4]

$$P_a = P_s + P_q$$

where P_s is due to soil self-weight and is calculated as follows:

[Eq. 4-5]

$$P_s = 0.5 K_a \gamma_i H^2 \cos(\delta_i - \omega)$$

and P_q is due to the uniformly distributed surcharges extending over the backfill surface and is calculated as:

[Eq. 4-6]

$$P_q = (q_l + q_d)\, K_a\, H \cos(\delta_i - \omega)$$

The resultant forces P_s and P_q are assumed to act at distances of Y_s and Y_q above the front toe of the lowermost SRW unit:

[Eq. 4-7]

$$Y_s = (H/3)$$

[Eq. 4-8]

$$Y_q = (H/2)$$

Passive resistance developed in front of the wall is conservatively ignored for the reasons given in Sections 2.5.2 and 3.4.5.

4.5.2 WEIGHT OF SEGMENTAL RETAINING WALL

The weight W_w of the column of SRW units on the leveling pad is based on the hinge height H_h of the wall facing (Equation 4-1 or 4-2) and the weight per unit volume of SRW units γ_u. The SRW wall weight W_w per lineal foot acting at the base will be limited by the hinge height H_h and can be calculated as: [Eq. 4-9]

$$W_w = H_h \gamma_u W_u$$

If the hinge height calculated according to Equation 4-1 is larger than the actual wall height H, then the wall height H should be used as the hinge height (i.e., $H_h = H$) as in Equation 4-2.

4.5.3 BASE SLIDING

The external forces acting to destabilize the conventional single depth SRW are shown in **Figure 4-4**.

The base sliding resistance is calculated as follows:

[Eq. 4-10]

$$R_{S(W)} = \mu_b \, [W_w \tan \phi + c \;\; W_u]$$

The SRW weight W_w is calculated using Equation 4-9.

The soil strength parameters c and ϕ should be selected according to the soil type upon which the SRW unit is founded. Normally, this will be an aggregate leveling pad. For some projects, the base unit may rest on the foundation soils. In either case, the available sliding resistance must be reduced by a masonry friction reduction factor μ_b applied to the underlying soil friction coefficient **tan** ϕ and c. This factor accounts for reduced shear resistance due to the relatively smooth masonry unit sliding across soil determined from large scale testing. Actual test data specific to soil type and SRW unit should be used in design. In the absence of specific test data a reasonable value for μ_b may be selected using **Table 4-2**.

TABLE 4-2

Masonry Friction Reduction Factor, μ_b [Ref. 27]

Soil Type (USCS)	Soil ϕ (Deg)	Masonry Friction Reduction Factor μ_b
GW, GP	37 - 42	0.7
GM, SW, SP	33 - 40	0.65
GC, SM, SC	28 - 35	0.6
ML, CL	25 - 32	0.55

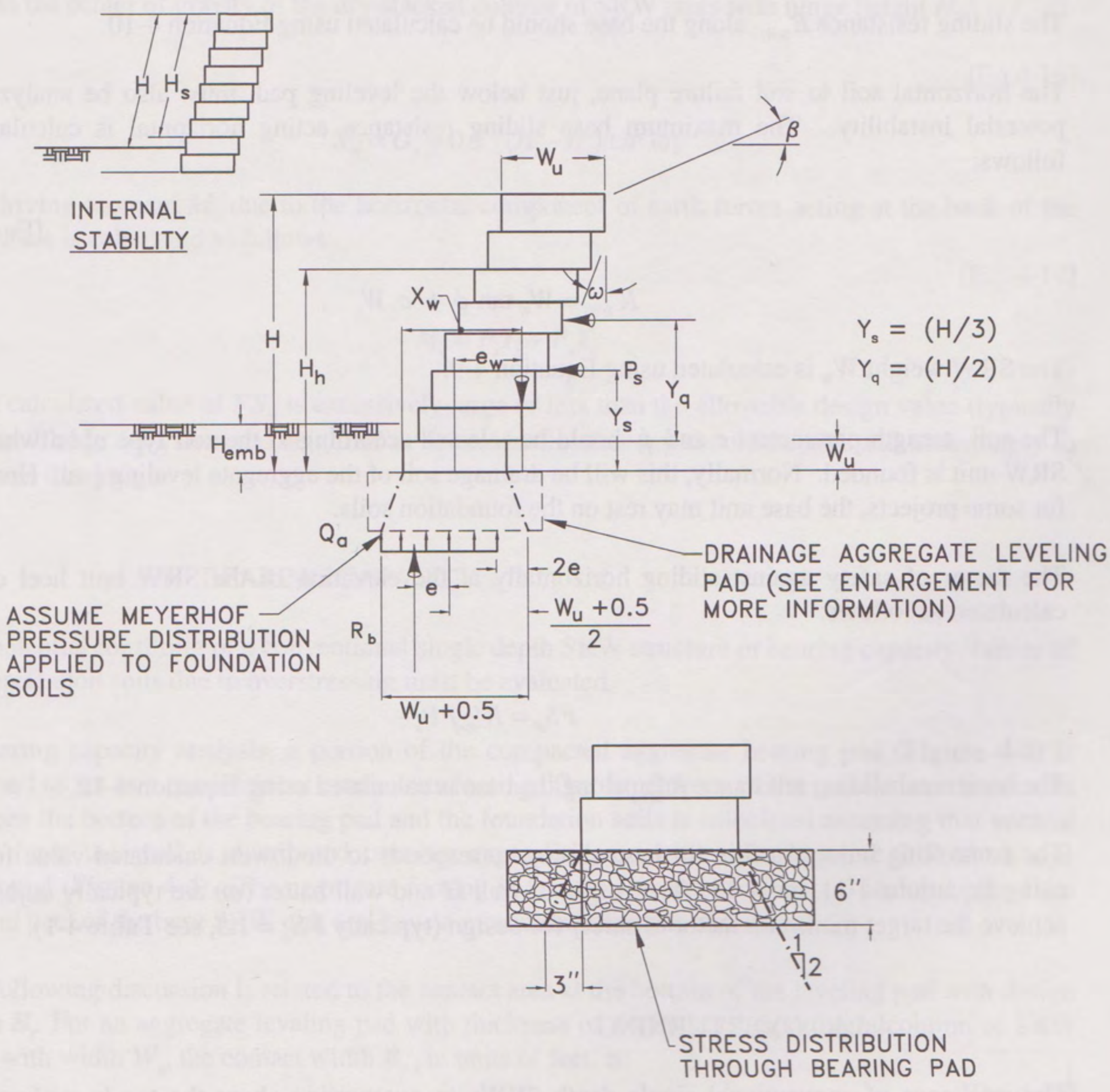


FIGURE 4–4: FREE BODY DIAGRAM OF EXTERNAL FORCES FOR CONVENTIONAL SINGLE DEPTH SRW

The factor of safety against sliding parallel to the base of the SRW unit can be calculated as follows:

[Eq. 4-11]

$$FS_{sl} = R_{S(W)} / P_a$$

The sliding resistance $R_{S(W)}$ along the base should be calculated using Equation 4-10.

The horizontal soil to soil failure plane, just below the leveling pad, must also be analyzed for potential instability. The maximum base sliding resistance acting horizontal is calculated as follows:

[Eq. 4-12]

$$R'_{S(W)} = W_W \tan \phi + c \; W_u$$

The SRW weight W_w is calculated using Equation 4-9.

The soil strength parameters c and ϕ should be selected according to the soil type upon which the SRW unit is founded. Normally, this will be drainage soil of the aggregate leveling pad. However, for some projects, the base unit may rest on the foundation soils.

The factor of safety against sliding horizontally at the elevation of the SRW unit heel can be calculated as follows:

[Eq. 4-13]

$$FS_{sl} = R'_{s(w)} / P_a$$

The horizontal sliding resistance $R'_{S(W)}$ along the base is calculated using Equation 4-12.

The controlling failure surface for base sliding corresponds to the lowest calculated value for FS_{sl} using Equations 4-11 and 4-13. The height of wall H and wall batter (ω) are typically adjusted to achieve the target minimum factor of safety for design (typically FS_{sl} = 1.5, see **Table 4-1**).

4.5.4 OVERTURNING

The resistance of conventional single depth SRWs to overturning about the toe is evaluated by calculating a factor of safety FS_{ot} that is the ratio of the sum of the resisting moments to the sum of the driving moments taken with respect to the toe of the wall (point 0 in **Figure 4-4**), hence:

[Eq. 4-14]

$$FS_{ot} = M_r / M_o$$

The resisting moment M_r can be calculated as follows:

[Eq. 4-15]

$$M_r = W_w X_w$$

where X_W is the resisting moment arm. The term X_W is calculated as the distance from the toe of the wall to the center of gravity of the dry-stacked column of SRW units with hinge height H_h.

[Eq.4-16]

$$X_W = G_u + 0.5 \{(H_h - H_u) \tan \omega\}$$

The driving moment M_o due to the horizontal component of earth forces acting at the back of the SRW face is calculated as follows:

[Eq. 4-17]

$$M_o = P_s Y_s + P_q Y_q$$

If the calculated value of FS_{ot} is excessively large or less than the allowable design value (typically 1.5, see **Table 4-1**), then the height H or facing inclination (ω) should be adjusted accordingly to optimize the design.

4.5.5 BEARING CAPACITY

The potential for tilting of a conventional single depth SRW structure or bearing capacity failure of the foundation soils due to overstressing must be evaluated.

In bearing capacity analysis, a portion of the compacted aggregate bearing pad (**Figure 4-4**) is assumed to act as a conventional continuous footing. The dimensions of the maximum contact area between the bottom of the bearing pad and the foundation soils is calculated assuming that vertical stress from the wall is distributed over an expanded area with side slopes at 2 vertical to 1 horizontal (**Figure 4-4**). The aggregate footing is assumed to extend at least 6 inches beyond the toe and heel of the base SRW unit and have a minimum thickness of 6 inches.

The following discussion is related to the contact area at the bottom of the leveling pad with design width B_f. For an aggregate leveling pad with thickness of six inches supporting a column of SRW units with width W_u, the contact width B_f, in units of feet, is:

[Eq. 4-18]

$$B_f = W_u + 0.5$$

The following assumptions are made in bearing capacity analysis:

1. The total footing load is distributed uniformly over a portion of the footing width B_e to account for base eccentricity, e, (the conventional Meyerhof approach to geotechnical footing design).

2. Inclination of net footing loads is ignored in order to avoid excess conservativeness in bearing capacity calculations and to be consistent with conventional practice for rigid gravity wall structures. The conservatism is created by the design assumption to ignore vertical component of $\boldsymbol{P_a}$. Similarly, corrections for footing shape and length are also ignored in this set of calculations.

3. The effect of embedment depth $\boldsymbol{H_{emb}}$ to the top of the aggregate pad (footing) is accounted for as a dead load surcharge. The soil in front of the wall is assumed to have no shear resistance in bearing capacity analyses. The assumption of a permanent surcharge mass must only be exercised if large excavations in front of the wall will not occur for the life of the structure. If excavations are made they should normally be restricted to short excavation lengths in order to minimize the potential loss of toe support at the wall.

The factor of safety $\boldsymbol{FS_{bc}}$ with respect to bearing capacity failure of the foundation soils can be expressed as follows:

[Eq. 4-19]

$$FS_{bc} = Q_{ult} / Q_a$$

where $\boldsymbol{Q_{ult}}$ represents the ultimate bearing capacity of the foundation soils and $\boldsymbol{Q_a}$ the applied bearing stress. The magnitude of $\boldsymbol{Q_{ult}}$ is calculated as follows:

[Eq. 4-20]

$$Q_{ult} = c_f N_c + 0.5\gamma_f B'_f N_\gamma + \gamma_f H_{emb} N_q$$

This is the classical bearing capacity equation for a continuous footing [Ref. 13]. The quantities N_c, N_γ and N_q are dimensionless bearing capacity coefficients that can be obtained from **Figure 4-5** using the peak friction angle of the foundation soil $\boldsymbol{\phi_f}$. The quantity $\boldsymbol{B'_f}$ is the equivalent footing width due to eccentric footing loads and is calculated as:

[Eq. 4-21]

$$B_f' = B_f - 2e$$

Eccentricity can be calculated by summing moments about the center of the footing base. The center of rotation is taken as the point located a horizontal distance $\boldsymbol{W_u}$ / 2 from the toe of the bottom SRW unit. Moments are considered positive in a counter-clockwise direction in this calculation set. Eccentricity is calculated as:

[Eq. 4-22]

$$e = [P_s Y_s + P_q Y_q - W_W e_W] / W_W$$

where:

[Eq. 4-23]

$$e_W = X_W - 0.5 W_u$$

ϕ^{+} (deg)	N_c	N_q	N_γ	N_q/N_c	$\tan\phi$
0	5.14	1.00	0.00	0.20	0.00
1	5.38	1.09	0.07	0.20	0.02
2	5.63	1.20	0.15	0.21	0.03
3	5.90	1.31	0.24	0.22	0.05
4	6.19	1.43	0.34	0.23	0.07
5	6.49	1.57	0.45	0.24	0.09
6	6.81	1.72	0.57	0.25	0.11
7	7.16	1.88	0.71	0.26	0.12
8	7.53	2.06	0.86	0.27	0.14
9	7.92	2.25	1.03	0.28	0.16
10	8.35	2.47	1.22	0.30	0.18
11	8.80	2.71	1.44	0.31	0.19
12	9.28	2.97	1.69	0.32	0.21
13	9.81	3.26	1.97	0.33	0.23
14	10.37	3.59	2.29	0.35	0.25
15	10.98	3.94	2.65	0.36	0.27
16	11.63	4.34	3.06	0.37	0.29
17	12.34	4.77	3.53	0.39	0.31
18	13.10	5.26	4.07	0.40	0.32
19	13.93	5.80	4.68	0.42	0.34
20	14.83	6.40	5.39	0.43	0.36
21	15.82	7.07	6.20	0.45	0.38
22	16.88	7.82	7.13	0.46	0.40
23	18.05	8.66	8.20	0.48	0.42
24	19.32	9.60	9.44	0.50	0.45
25	20.72	10.66	10.88	0.51	0.47
26	22.25	11.85	12.54	0.53	0.49
27	23.94	13.20	14.47	0.55	0.51
28	25.80	14.72	16.72	0.57	0.53
29	27.86	16.44	19.34	0.59	0.55
30	30.14	18.40	22.40	0.61	0.58
31	32.67	20.63	25.99	0.63	0.60
32	35.49	23.18	30.22	0.65	0.62
33	38.64	26.09	35.19	0.68	0.65
34	42.16	29.44	41.06	0.70	0.67
35	46.12	33.30	48.03	0.72	0.70
36	50.59	37.75	56.31	0.75	0.73
37	55.63	42.92	66.19	0.77	0.75
38	61.35	48.93	78.03	0.80	0.78
39	67.87	55.96	92.25	0.82	0.81
40	75.31	64.20	109.41	0.85	0.84
41	83.86	73.90	130.22	0.88	0.87
42	93.71	85.38	155.55	0.91	0.90
43	105.11	99.02	186.54	0.94	0.93
44	118.37	115.31	224.64	0.97	0.97
45	133.88	134.88	271.76	1.01	1.00
46	152.10	158.51	330.35	1.04	1.04
47	173.64	187.21	403.67	1.08	1.07
48	199.26	222.31	496.01	1.12	1.11
49	229.93	265.51	613.16	1.15	1.15
50	266.89	319.07	762.89	1.20	1.19

+ USE ϕ_r, THE PHI ANGLE FOR FOUNDATION SOILS

FIGURE 4-5: BEARING CAPACITY FACTORS (AFTER Vesic, Ref.13)

The applied bearing stress Q_a is calculated as follows using the equivalent footing width B_f' determined in Equation 4-21.

[Eq. 4-24]

$$Q_a = W_W / B_f'$$

For many projects, the site geotechnical engineer may have established an allowable bearing pressure that limits total and differential settlement, as well as precludes a general bearing capacity failure. Q_a should be less than the allowable bearing pressure.

The factor of safety FS_{bc} calculated from Equation 4-19 must satisfy minimum recommended values for design (typically 2.0, see **Table 4-1**). For a given retained soil and foundation soil the factor of safety can be adjusted to satisfy a minimum design value by changing the wall height H, wall batter (ω) and/or thickness of aggregate bearing pad.

4.5.5.1 Base Eccentricity

The base eccentricity (e) calculated using Equation 4-22 is not required to satisfy any criterion for the distribution of bearing pressure at the base of the bearing pad (footing) in this design methodology. The constraint adopted in some geotechnical engineering textbooks [Refs. 8, 13, 17, 20], requiring the total foundation load fall within the middle third distance of the bearing pad (footing), results in excessively conservative estimates of wall height.

4.5.5.2 Settlement

A gravity wall structure will typically impose pressures on the foundation soils in excess of pre-construction activities that lead to compression of foundation materials. For cohesionless soils (i.e., sands and gravels), these settlements are typically small and occur mostly during construction. Saturated cohesive soils, on the other hand, may exhibit large time-dependent deformations. For most routine structures, conventional one-dimensional consolidation theory as presented in most geotechnical engineering textbooks [Refs. 8, 13, 17, 20] will give an acceptable estimate of potential settlements. The calculation of total and differential settlements due to footing loads applied to the foundation soils is complex and requires a thorough knowledge of the consolidation properties of site soils. Therefore, it is recommended that the designer engage a professional engineer who is familiar with the site conditions and structure loadings to estimate SRW construction-induced settlements.

The dry-stack mortarless construction method for SRWs founded on an aggregate bearing pad creates a flexible gravity structure that can tolerate large total settlements and moderate differential settlement. For most standard SRW units (< 2 square feet face area) a differential settlement of 1% is acceptable. In situations where large settlement and/or greater differential settlement than 1% is expected special design steps should be taken.

4.5.5.3 Design and Construction Strategies to Minimize Settlements

Unsuitable foundation conditions at the planned base elevation of the wall can be improved by one or a combination of the following techniques:

- Excavate and replace unsuitable soils
- Locate base of SRW at competent soil
- Expand the aggregate leveling pad width and thickness
- Reinforce a thickened aggregate leveling pad with geogrid
- Reduce foundation stress by tiering wall section
- Preload the area prior to wall construction
- Preload wall prior to paving or building construction above wall
- Employ soil improvement techniques: vibrocompaction, stone columns, dynamic compaction

Prior to executing any of these foundation improvement strategies, enlist the assistance of a qualified geotechnical engineer familiar with site conditions to select the most efficient and cost effective solution.

Section 4.6
INTERNAL STABILITY

To resist lateral earth pressures, a dry-stacked column of SRW units must act as a coherent mass. The SRW units must have sufficient interface shear capacity to transfer all applied external forces to the base of the structure.

The analysis for internal shear capacity (**Figure 4-1B**) is similar to external base sliding stability calculations described in Section 4.5.3 and illustrated in **Figure 4-1A**. Internal sliding resistance, however, is developed by shear at the unit-to-unit interface. The height of the column of dry-stacked SRW units above the sliding surface is described by the intermediate height parameter $\boldsymbol{H_s}$ as illustrated in **Figure 4-4**.

Internal sliding analysis at SRW unit interface layers should proceed from the bottom of the wall to the top to expedite calculations. Lower elevation interfaces are typically more critical. This sliding analysis should be performed at each SRW unit interface elevation.

The portion of internal sliding resistance due to interface shear capacity V_u can be calculated from:

[Eq. 4-25]

$$V_u = a_u + W_W \tan \lambda_u$$

where:

$\boldsymbol{a_u}$ = apparent shear capacity adhesion (lb/ft)
$\boldsymbol{\lambda_u}$ = apparent peak interface friction angle between SRW units (deg)

W_w = total weight of column of dry-stacked SRW units between surface and hinge height calculated relative to the sliding surface (i.e., H_h vs. H_s) (lb/ft)

The interface shear capacity V_u has units of force/length to be consistent with the convention adopted for all stability calculations in this manual.

The weight term W_w is calculated using the hinge height H_h (Equations 4-1, 4-2) taken with respect to the internal sliding surface H_s. The maximum hinge height dimension is each intermediate height H_s. The vertical weight of the column of dry-stacked units above the sliding surface is calculated using Equation 4-9:

[Eq. 4-9]

$$W_W = H_h \gamma_u W_u$$

The hinge height H_h is calculated using Equation 4-1 in Section 4.2. If $H_h > H_s$, H_s is used in Equation 4-9.

The interface shear strength parameters a_u and λ_u must be established from appropriate full-scale testing as discussed in Section 3.2.3.2. The test method is described in NCMA test method SRWU-2 (Determination of Shear Strength between Segmental Concrete Units).

The horizontal component of the total sliding active earth force P_a located above the intermediate height internal sliding surface H_s is calculated as follows:

[Eq. 4-26]

$$P_a = K_a\,[0.5(\gamma_i)\;H_s^2 + (q_l + q_d)\,H_s]\cos(\delta_i - \omega)$$

The factor of safety for shear capacity FS_{sc} is the ratio of resisting forces to driving forces.

[Eq. 4-27]

$$FS_{sc} = V_u / P_a$$

The magnitude of FS_{sc} should satisfy minimum recommended design values for internal sliding stability (typically 1.5, see **Table 4-1**). For a given set of soils, backslope angle β and surcharge condition, the magnitude of FS_{sc} can be adjusted by lowering the wall height H, increasing wall facing batter (ω), or selecting an alternative SRW unit.

Section 4.7
GLOBAL STABILITY

The general mass movement of a conventional SRW structure and adjacent soil mass is called a global stability failure (**Figure 4-6**). Global stability failure may result from changes in grade, weak soil layers, increase in groundwater elevation, and/or the additional gravitational forces imposed on the site soils as a result of SRW construction. Analysis of the stability of soil masses comprising the gravity wall and adjacent soils is required as part of the design and analysis methodology recommended in this manual.

A detailed presentation of slope stability methods can be found in many geotechnical engineering text books [Refs. 4, 8, 17, 20, 25, 26, 27]. A brief discussion of slope stability methods of analysis is given in Section 2.5.5 and a sample slope stability analysis involving a gravity SRW structure is presented in Appendix A.

Wall heights satisfying minimum recommended factors of safety for internal and external stability using the hinge height concept (Section 4.2) compare favorably with wall heights satisfying a minimum factor of safety of 1.3 against toe failure using conventional slope stability methods of analysis. In other words, implementation of the hinge height criteria during internal and external stability calculations for routine structures will typically approximate the maximum design height based on slope stability methods of analysis on the same structure. This observation allows the designer to carry out preliminary analysis and design without the requirement to perform relatively tedious global stability analyses. **Nevertheless, any final design should be checked for potential global instability as part of the overall design methodology recommended in this manual, particularly when underlying weak soil layers or groundwater are present.**

In the event the structure does not satisfy a minimum design factor of safety against global instability, the wall height H or facing batter ω might have to be changed or geosynthetic reinforcement added. The minimum design factor of safety against global instability recommended in this manual ranges from 1.3 to 1.5 depending on the specific site conditions.

Section 4.8
EXAMPLE CALCULATIONS

An example analysis and design problem for a generic conventional single depth SRW is presented in Appendix A. The example serves to illustrate many of the analytical concepts presented in this design manual and to highlight important calculation steps.

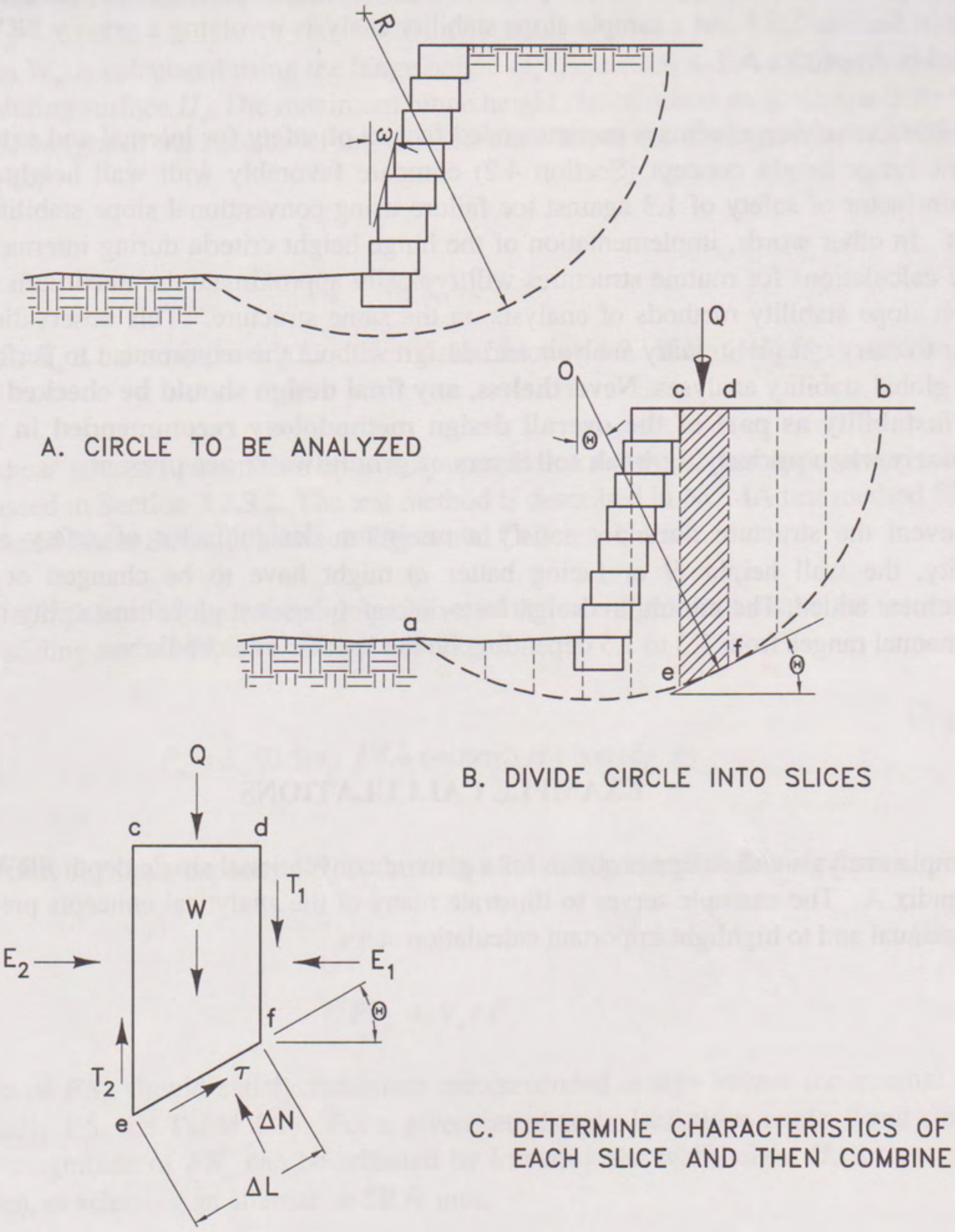


FIGURE 4–6: GLOBAL STABILITY ANALYSIS BY GENERAL METHOD OF SLICES

SECTION 5
REINFORCED SOIL SEGMENTAL RETAINING WALL DESIGN

Reinforced soil segmental retaining walls are gravity retaining walls with an expanded width created by a geosynthetic reinforced (infill) soil mass located behind a column of dry-stacked (SRW) units (**Figure 2-1B, 2-10**). The dry-stacked column of SRW units and the geosynthetic reinforced (infill) soil zone act together to resist the destabilizing forces generated by the retained soil (backfill) and surcharge loadings.

Geosynthetic reinforcement is used to create a significantly larger gravity mass than is possible with an isolated dry-stacked column of SRW units. Geosynthetic reinforced soil SRWs can be used to construct higher retaining wall structures and support greater surcharge loads than possible with conventional SRW structures.

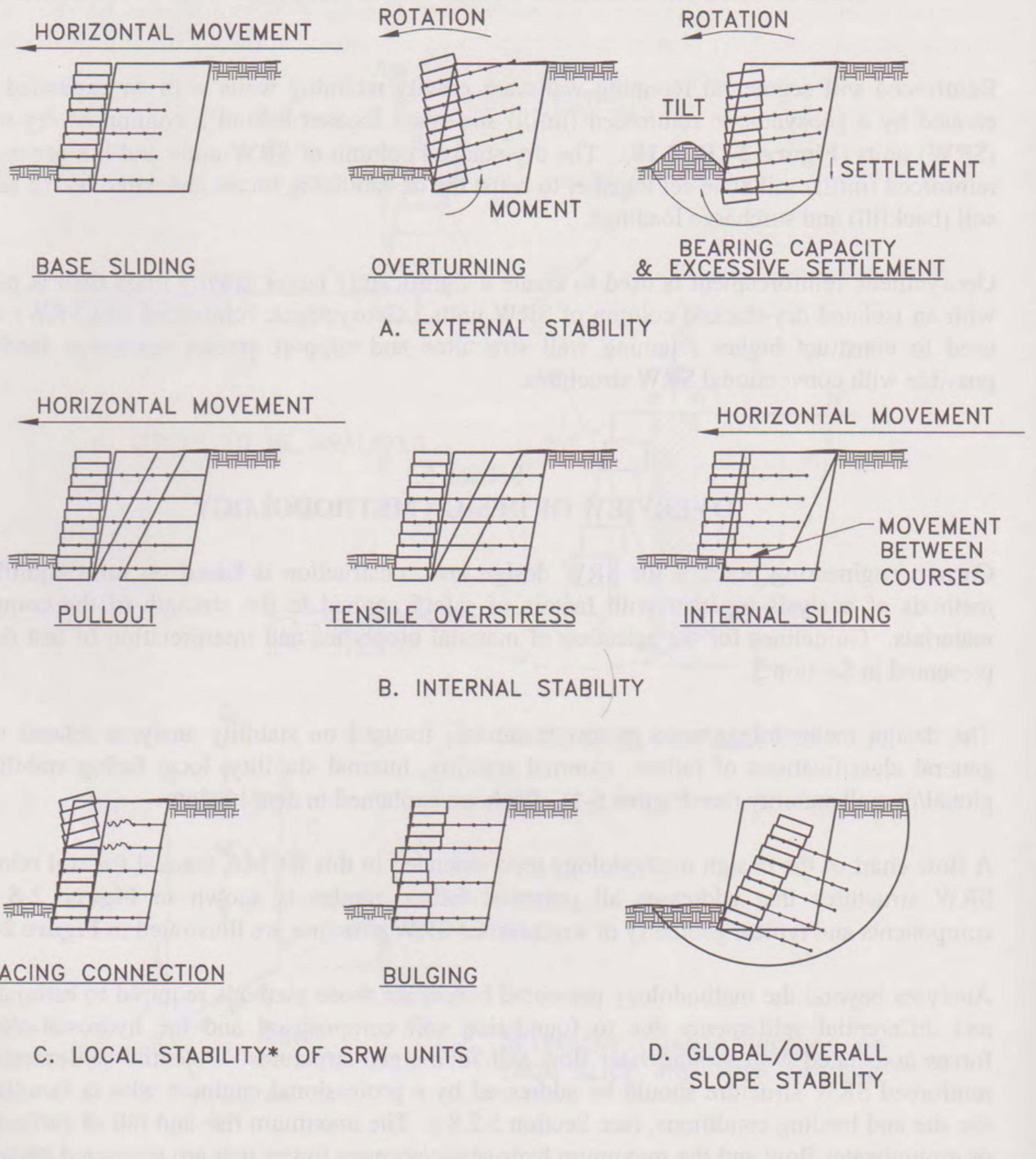
Section 5.1
OVERVIEW OF DESIGN METHODOLOGY

Current engineering practice for SRW design and construction is based on limit equilibrium-methods of analysis together with factors of safety applied to the strength of the component materials. Guidelines for the selection of material properties and interpretation of test data are presented in Section 3.

The design methodology used in this manual is focused on stability analyses related to four general classifications of failure: external stability, internal stability, local facing stability and global/overall stability (see **Figure 5-1**). Each are explained in detail below.

A flow chart of the design methodology recommended in this NCMA manual for soil reinforced SRW structures that addresses all potential failure modes is shown in **Figure 2-8**. The components and typical geometry of a reinforced SRW structure are illustrated in **Figure 2-10**.

Analyses beyond the methodology presented herein are those methods required to estimate total and differential settlements due to foundation soil compression and the hydrostatic/seepage forces associated with groundwater flow and submerged structures. Potential settlements for a reinforced SRW structure should be addressed by a professional engineer who is familiar with the site and loading conditions, (see Section 5.2.8.). The maximum rise and fall of surface water or groundwater flow and the maximum hydrostatic/seepage forces that are generated should also be determined by a professional engineer (see Sections 8.1 and 8.2).



* NOTE: THE MAXIMUM UNREINFORCED HEIGHT IS DETERMINED SIMILAR TO CONVENTIONAL SRWs, SEE FIGURE 2–6 A & B.

FIGURE 5–1: MAIN MODES OF FAILURE FOR REINFORCED SOIL SRWs

5.1.1 EXTERNAL STABILITY

External stability analyses examine the stability of the mass formed by the facing units and reinforced soil zone with respect to active earth forces generated by self-weight of the retained soils and distributed surcharge pressures beyond the reinforced zone (**Figure 5-1**). These analyses determine the minimum length ***L*** of geosynthetic reinforcement by checking:

- **Base Sliding:** Outward movement along the base of the reinforced soil mass due to insufficient shear resistance in the soil.

- **Overturning:** Rotation of the reinforced soil mass about the toe of the wall.

- **Bearing Capacity:** Shear failure or unacceptable deformation of the foundation soils due to excessive foundation pressures.

5.1.2 INTERNAL STABILITY

Internal stability analyses examine the effectiveness of the geosynthetic reinforcement in holding the reinforced soil mass together so the geosynthetic layers and soil function as a monolithic block (**Figure 5-1B**). The internal stability analyses determine the minimum strength, number and vertical spacing of reinforcement layers by examining:

- **Tensile Overstress:** When the tensile stress in the geosynthetic reinforcement exceeds an established product specific allowable working stress level.

- **Pullout:** Excessive movement of the geosynthetic through the reinforced soil zone, generally without rupture of the reinforcement.

- **Internal Sliding:** Geosynthetic reinforcement layers may create preferred planes of sliding at elevations along the height of the wall.

5.1.3 LOCAL STABILITY

The unique dry-stacked construction method using SRW units and its connection to geosynthetic reinforcement layers requires that stability analyses be carried out to ensure the column of SRW units remains intact and does not bulge excessively (**Figure 5-1C**). The following local stability analyses limit the vertical spacing of geosynthetic reinforcement:

- **Facing Connection:** The design strength of the facing connection between SRW units and the geosynthetic reinforcement must not be exceeded to ensure integrity of the composite system. To ensure serviceability of critical structures, this movement will be limited to 0.75 inches.

- **Bulging:** The vertical spacing of geosynthetic reinforcement layers must be restricted and the interface shear capacity between units must be adequate to prevent excessive shear deformation or shear failure between successive courses of facing units.

- **Maximum Unreinforced Height:** The unreinforced height of SRW units at the top of the structure must not lead to toppling (overturning) or sliding of the units near the crest of the wall. The analyses performed are the same as overturning and internal stability for conventional SRWs, Sections 4.5.4 and 4.6, respectively.

5.1.4 GLOBAL STABILITY

Global/overall slope stability failure is mass movement of the entire reinforced soil SRW structure including soil adjacent to the structure (**Figure 5-1D**). Generally, the reinforced soil SRW is assumed to act as a coherent structure in the overall rotating mass (**Figure 4-6**). General discussion on the analysis of this failure mechanism can be found in Sections 2.5.5 and 4.7. The responsibility for ensuring adequate global/overall slope stability rests with the site civil or geotechnical engineer and not the retaining wall designer. However, it is prudent for the wall design engineer to perform a confirming global stability analysis to ensure an adequate safety factor and notify all parties involved of any potential problems.

Section 5.2
DESIGN ASSUMPTIONS

The design and analysis of reinforced soil SRWs requires several assumptions and standards for consistent evaluation and uniform application of the generic methods presented. Additionally, several design assumptions germane to the analysis have been made to simplify calculations. The design assumptions made in this manual are outlined below.

5.2.1 COULOMB EARTH PRESSURE THEORY

Earth pressures and forces used in external, internal and local facing stability calculations are based on Coulomb earth pressure theory described in Section 3.4.5. Calculation of the coefficient of active earth pressure K_a is based on Equation 3-11 and its interpretation is consistent for external, internal and local facing stability analyses.

The reader is referred to Section 3.4.5 of this manual for details of the calculations and assumptions related to the determination of active earth pressures and forces in reinforced SRW stability design.

5.2.2 GROUNDWATER TABLE

The stability calculations presented in Section 5 assume that the groundwater table is located well below the base of the wall (i.e., greater than $0.66H$ where H is the height of the wall).

Hence, all stresses are computed as total stresses and do not have to be modified for the influence of porewater pressure. In addition, the destabilizing effect of hydrostatic pressures or seepage forces is not a concern when the groundwater table is at this depth. Specific details on handling groundwater in the analysis are presented in Sections 8.1 and 8.2.

5.2.3 HINGE HEIGHT

The hinge height H_h concept presented in Section 4.2 and illustrated in **Figure 4-2** is implemented in all local facing stability calculations. The hinge height limits the maximum weight applied to any SRW unit interface for calculation of shear capacity and facing connection strength based on the geometry of the system and shape of the SRW unit.

5.2.4 DIMENSIONS OF REINFORCED MASS

In this design manual the base of the reinforced (infill) soil zone (*L*) for external stability calculations includes the width of the dry-stacked column of SRW units. Additionally, the reinforcement length may change from top to bottom of a reinforced soil wall provided that each layer has a minimum length greater than *L* determined from external stability requirements. The lateral extent of the reinforced soil mass shall be taken as a line drawn parallel to the face of the stacked SRW units at a distance *L* from the face of the structure (**Figure 2-10**).

The results of external stability analyses for sliding and bearing capacity failure mechanisms are used to determine the minimum required base width *L*. The minimum base width *L* shall not be less than **0.6*H*** regardless of the results of stability calculations (*H* is the vertical height of the wall face). The purpose of this empirical constraint on *L* is to prevent the construction of unusually narrow reinforced retaining walls.

Selected layers of reinforcement may be lengthened at the top of a reinforced SRW to satisfy anchorage requirements. Any additional length of reinforcement beyond the base reinforcement length is not considered to modify the dimensions of the reinforced zone described above. In other words, for external stability calculations, the reinforced zone is always limited to the mass of soil located between parallel front face and back boundaries a distance *L* (base width) apart regardless of the number of reinforcement layers that may have to be locally extended at the top of a wall to satisfy internal stability requirements (i.e., pullout).

5.2.5 TREATMENT OF UNIFORMLY DISTRIBUTED DEAD LOADS AND LIVE LOADS

In this design manual only the dead load portion of any distributed surcharge pressure is considered in the expressions that calculate stabilizing forces or moments. For example, transient vehicular loads are not considered to assist the stability of the reinforced soil mass for base sliding, overturning and pullout calculations. They are considered to contribute to destabilizing forces only. This approach is adopted to ensure safe design and to avoid the use of surcharge pressures as contributions to structure stability when they will not be continuous for the life of the system.

5.2.6 SEISMIC LOADING

Reinforced SRWs subjected to seismic and/or dynamic loading will, in general, perform well due to their flexible nature and enhanced ductility. The design calculations associated with seismic loading are briefly outlined in Section 8.3. The reader is encouraged to utilize the referenced literature provided with this manual.

5.2.7 GLOBAL STABILITY

The general mass movement of a soil reinforced SRW structure and adjacent soil mass is called a global stability failure (**Figure 4-6**). Global stability failure may result from changes in grade, weak soil layers, increase in groundwater elevation, and/or the additional gravity forces imposed on the site soils as a result of SRW construction. However, stability analysis of soil masses comprising the gravity wall and adjacent soils is required as part of the design and analysis methodology recommended in this manual. The responsibility for ensuring adequate global/overall slope stability should rest with the site civil or geotechnical engineer. It is prudent to perform a confirming global stability analysis to ensure an adequate safety factor.

A detailed presentation of slope stability methods can be found in many geotechnical engineering text books [Refs. 4, 8, 17, 20, 24, 25, 26, 27]. A brief discussion of slope stability methods of analysis is given in Section 2.5.5 and 4.7, and a sample slope stability analysis involving a conventional SRW structure is presented in Appendix A.

5.2.8 SETTLEMENT

A reinforced soil SRW will typically impose pressures on the foundation soils in excess of pre-construction activities that lead to compression of foundation materials. For granular soils (i.e sands and gravels), these settlements are typically small and occur mostly during construction. Saturated cohesive soils, on the other hand, may exhibit large time-dependent deformations. For most routine structures, conventional one-dimensional consolidation theory as presented in most geotechnical engineering textbooks [Refs. 8, 13, 17, 20] will give an acceptable estimate of potential settlements. The calculation of total and differential settlements due to footing loads applied to the foundation soils is complex and requires a thorough knowledge of the consolidation properties of site soils. It is recommended the designer engage a professional engineer who is familiar with the site conditions and structure loadings to estimate SRW construction-induced settlements.

The discontinuous wall facing in reinforced soil SRW construction allows these structures to tolerate larger differential settlements than rigid systems. The dry-stack mortarless construction method for SRWs found on an aggregate leveling pad creates a flexible gravity structure that can tolerate large total settlements and moderate differential settlement. For most standard SRW units (< 2 square feet face area), differential settlement of one percent is acceptable. In situations where large settlement and/or greater than one percent is expected, special precautions should be taken.

5.2.8.1 Design and Construction Strategies to Minimize Settlement

Unsuitable foundation conditions at the planned base elevation of the wall can be improved by any one or combination of the following techniques:

- Excavate and replace unsuitable soils
- Locate base of SRW at competent soil
- Expand the aggregate leveling pad width and thickness
- Reinforce a thickened aggregate leveling pad with geogrid
- Reduce foundation stress by tiering wall section
- Preload the area prior to wall construction
- Preload wall prior to paving or building construction above wall
- Employ soil improvement techniques; vibrocompaction, stone columns, dynamic compaction.

Section 5.3
RECOMMENDED MINIMUM FACTORS OF SAFETY AND DESIGN CRITERIA

Selection of appropriate factors of safety should be based on the certainty with which design parameters and the consequences of failure are known. **Table 5-1** lists the recommended minimum safety factors for reinforced SRWs. Included in these recommended minimums are typical levels of uncertainty in wall geometry, imposed loadings, and the accuracy of design soil parameters. **Table 5-1** also provides other applicable design criteria for soil-reinforced SRWs.

TABLE 5-1

Recommended Minimum Factors of Safety and Design Criteria for Reinforced Soil SRWs		
<u>Failure Modes</u>		
Base Sliding	FS_{sl}	1.5
Overturning	FS_{ot}	2.0
Bearing Capacity	FS_{bc}	2.0
Global Stability	FS_{gl}	1.3 - 1.5
Tensile Overstress	FS_{to}	1.0
Pullout	FS_{po}	1.5
Facing Shear Capacity	FS_{sc}	1.5
Connection	FS_{cs}	1.5
<u>Design Criteria</u>		
Uncertainties	FS_{UNC}	1.5
Facing shear (serviceability criterion)		0.75 inch
Connection (serviceability criterion)(Note 3)		0.75 inch
Minimum Base Width	L	0.6H
Minimum Wall Embedment (Note 5)	H_{emb}	0.5 foot
Minimum Anchorage Length	L_a	1.0 foot

NOTES:

1. The minimum factors of safety given in this table assume that stability calculations are based on measured site-specific soil/wall data. Measured data are defined as the results of tests carried out on actual samples of soils and geosynthetic products at the proposed structure and actual samples of masonry concrete units (i.e., the same molds, forms, mix design and infill material or same broad soil classification type (e.g.G, S, if applicable).
2. When estimated data is used, the designer should use larger factors of safety than those shown in this table or conservative estimates of parameter values. Estimated data includes bulk unit weight and shear strength properties taken from the results of ASTM methods of testing (or similar protocols) carried out on samples of soil having the same USCS classification as the project soil and the same geosynthetic product. Estimated data for facing shear capacity and connection capacity analyses shall be based on laboratory tests carried out on the same masonry concrete unit type under representative surcharge pressures for the project structure (and the same broad soil classification type, e.g. G, S, if applicable).
3. Minimum factors of safety must be satisfied for pullout, facing connection, and shear capacity. Maximum recommended deformation for the connection is ¾ inch.
4. For maximum unreinforced wall height, implement factors of safety for conventional SRWs, (**Table 4-1** and Sections 4.5.4 and 4.6).
5. Wall embedment to be determined as per **Table 2-1** and must meet minimum requirement above.

Section 5.4

PROPERTIES FOR REINFORCED SRW DESIGN

Similar to gravity SRWs, the dimensional and mechanical properties of segmental units must be established prior to design. These parameters are:

H_u = segmental unit height (ft)
H_{cu} = segmental unit cap (coping) height (ft)
W_u = segmental unit width (ft)
γ_u = weight per unit volume of segmental unit as placed (includes stone fill if applicable) (pcf)
G_u = center of gravity of segmental unit from front face, using as placed weight (ft)
ω = unit inclination due to segmental unit setback (Δ_u) per course (deg)
μ_b = interface friction coefficient for base segmental unit sliding on bearing soils
a_u, a_u' = apparent minimum shear capacity (peak and serviceability) between segmental units (lb/ft)
λ_u, λ_u' = apparent angle of friction (peak and serviceability) between segmental units (deg)
Δ_U = setback per course (in.)
ω = wall inclination off vertical clockwise positive (deg)

Additionally, the attachment properties between the geosynthetic reinforcement and SRW units are required. The properties listed below should be determined by laboratory testing of the SRW unit and geosynthetic reinforcement to be utilized according to the proposed NCMA test method SRWU-1 (Determination of Connection Strength between Geosynthetics and Segmental Concrete Units) provided in Appendix C.

a_{cs}, a'_{cs} = apparent minimum connection strength (peak and serviceability) between geosynthetic reinforcement and SRW unit (lb/ft)
λ_{cs}, λ'_{cs} = apparent angle of friction for connection (peak and serviceability) of geosynthetic reinforcement to SRW unit (deg))
$T_{ultconn}$, T_{conn} @ $^3/_4$ = maximum connection strength (peak and serviceability) between geosynthetic reinforcement and SRW unit (lb/ft)

The performance properties for each type (*t*) of geosynthetic reinforcement utilized in a design should be determined through the appropriate laboratory and field testing as described in Section 3.5.

T_a = Allowable Strength of the geosynthetic as calculated by Equation 3-21 (lb/ft).
C_i = Coefficient of interaction for pullout of the geosynthetic from the reinforced soils to be used as calculated by Equation 3-22.
C_{ds} = Coefficient of direct sliding between the geosynthetic and reinforced soils to be used as calculated by Equation 3-23.

Section 5.5
EXTERNAL STABILITY OF REINFORCED SRWs

External stability calculations for reinforced SRW structures consider the reinforced zone of infill soil and the dry-stacked column of SRW units to act as a monolithic gravity mass. The forces and geometry associated with external stability calculations are summarized in **Figure 5-2**.

5.5.1 EXTERNAL EARTH PRESSURES AND FORCES.

The distribution of earth pressures acting at the back of the reinforced zone due to the retained soil self-weight and surcharge loadings are shown in **Figure 5-2**. The back of the reinforced soil zone is taken as a surface inclined at angle 90° - ω to the horizontal (i.e., parallel to the inclined wall) and located a distance L from the front face. Here L is the base width of the reinforced soil mass. The earth pressures generated by the retained soil zone are considered to act at the back of the reinforced zone over a height ($H + h$). To calculate the maximum height of the backslope h above the reinforced zone, the intersection L_β of the back of the reinforced (infill) soil zone and the backslope β must be determined. This can be related to L' the width of the reinforced zone at the wall crest and calculated as follows:

[Eq. 5-1]

$$L' = L - W_u$$

[Eq. 5-2]

$$L'' = \frac{L' \tan\beta \tan\omega}{1 - \tan\beta \tan\omega}$$

[Eq. 5-3]

$$L_\beta = L' + L''$$

[Eq. 5-4]

$$h = L_\beta \tan\beta$$

The active earth pressure coefficient K_a is calculated using Equation 3-11 and is based on values of the peak friction angle of the retained soil ϕ_r , backslope angle β, facing inclination angle ω, and the external interface friction angle δ_e (Equation 3-16). The lateral earth pressure is assumed to be applied in a direction perpendicular to the plane inclined at ω defining the back of the reinforced soil mass as altered by the interface friction angle δ_e (**Figure 5-2**). Since δ_e is assumed to mobilize full soil shearing resistance (Equation 3-16) along the back of the reinforced zone equal to the lower peak friction angle of the retained or reinforced (infill) soil, a downward inclined lateral pressure distribution results for most wall inclinations (i.e., typically $\delta_e > \omega$). Under these conditions, the vertical component of earth pressures can be safely is ignored to simplify stability calculations (i.e., the error is on the safe side).

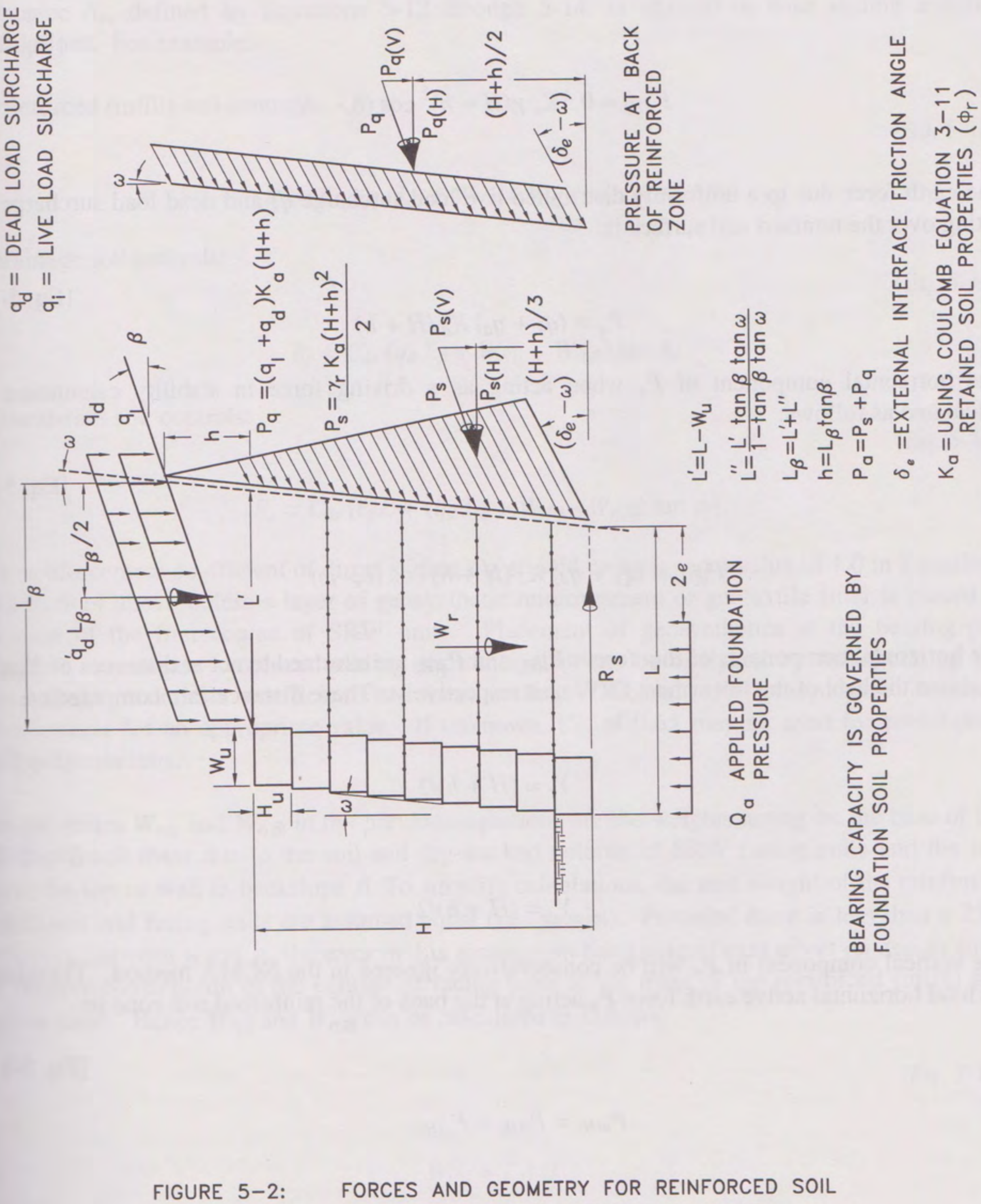


FIGURE 5–2: FORCES AND GEOMETRY FOR REINFORCED SOIL SRW EXTERNAL STABILITY CALCULATIONS

The earth force $\boldsymbol{P_s}$ due to the retained soil self-weight is calculated as follows:

[Eq. 5-5]

$$P_s = 0.5K_a\,\gamma_r\,(H + h)^2$$

The horizontal component of $\boldsymbol{P_s}$ is calculated as follows:

[Eq. 5-6]

$$P_{s(H)} = 0.5K_a\,\gamma_i(H + h)^2 \cos(\delta_e - \omega)$$

The earth force due to a uniformly distributed live load surcharge $\boldsymbol{q_l}$ and dead load surcharge $\boldsymbol{q_d}$ acting over the retained soil surface is:

[Eq. 5-7]

$$P_q = (q_l + q_d)\,K_a\,(H + h)$$

The horizontal component of $\boldsymbol{P_a}$ when acting as a driving force in stability calculation is calculated as follows:

[Eq. 5-8]

$$P_{q(H)} = (q_l + q_d)\,K_a\,(H + h)\cos(\delta_e - \omega)$$

The horizontal components of the forces $\boldsymbol{P_{s(H)}}$ and $\boldsymbol{P_{q(H)}}$ are assumed to act at distances of $\boldsymbol{Y_s}$ and $\boldsymbol{Y_q}$ above the heel of the lowermost SRW unit respectively. These distances are computed as:

[Eq. 5-9]

$$Y_s = (H + h)/3$$

[Eq. 5-10]

$$Y_q = (H + h)/2$$

The vertical component of $\boldsymbol{P_a}$ will be conservatively ignored in the NCMA method. Therefore, the total horizontal active earth force $\boldsymbol{P_a}$ acting at the back of the reinforced soil zone is:

[Eq. 5-11]

$$P_{a(H)} = P_{s(H)} + P_{q(H)}$$

5.5.2 BASE SLIDING

The horizontal stability of the reinforced zone is maintained by base sliding resistance $\boldsymbol{R_s}$ as shown in **Figure 5-3**. The magnitude of the base sliding resistance $\boldsymbol{R_s}$ is assumed to be controlled by the shear strength of the weakest soil at the base of the wall. Therefore the least resistance $\boldsymbol{R_s}$, defined by Equations 5-12 through 5-14, is utilized in base sliding stability calculations. For example:

If reinforced (infill) soil controls:

[Eq. 5-12]

$$R_s = C_{ds}\,(q_d\,L_\beta + W_{r(i)} + W_{r(\beta)})\tan\phi_i$$

If drainage soil controls:

[Eq. 5-13]

$$R_s = C_{ds}\,(q_d\,L_\beta + W_{r(i)} + W_{r(\beta)})\tan\phi_d$$

If foundation soil controls:

[Eq. 5-14]

$$R_s = C_{ds}\,[c_f\,L + (q_d\,L_\beta + W_{r(i)} + W_{r(\beta)})\tan\phi_f]$$

The reinforcement coefficient of direct sliding $\boldsymbol{C_{ds}}$ should be assigned a value of 1.0 in Equations 5-12 to 5-14 above unless a layer of geosynthetic reinforcement or geotextile filter is placed at the base of the first course of SRW units. Placement of geosynthetics at the bearing pad elevation may create a reduced sliding resistance that can be analyzed by incorporating the geosynthetic material's coefficient of direct sliding. Consult the geosynthetic materials manufacturer for an appropriate value. If unknown, $\boldsymbol{C_{ds}}$ of 0.65 may be used for preliminary design calculations.

The quantities $\boldsymbol{W_{r(i)}}$ and $\boldsymbol{W_{r(\beta)}}$ in the previous equations are the weights acting on the base of the reinforced soil mass due to the soil and dry-stacked column of SRW facing units and the soil above the top of wall in backslope $\boldsymbol{\beta}$. To simplify calculations, the unit weight of the reinforced (infill) soil and facing units are assumed equal (i.e $\gamma_u = \gamma_i$). Provided there is less than a 25% difference between γ_i and γ_u, the error in this assumption has no significant effect on design since the relative contribution of the column of facing units to the mass of the reinforced (infill) soil zone is small. Hence $\boldsymbol{W_{r(i)}}$ and $\boldsymbol{W_{r(\beta)}}$ can be calculated as follows:

[Eq. 5-15]

$$W_{r(i)} = L\gamma_i H$$

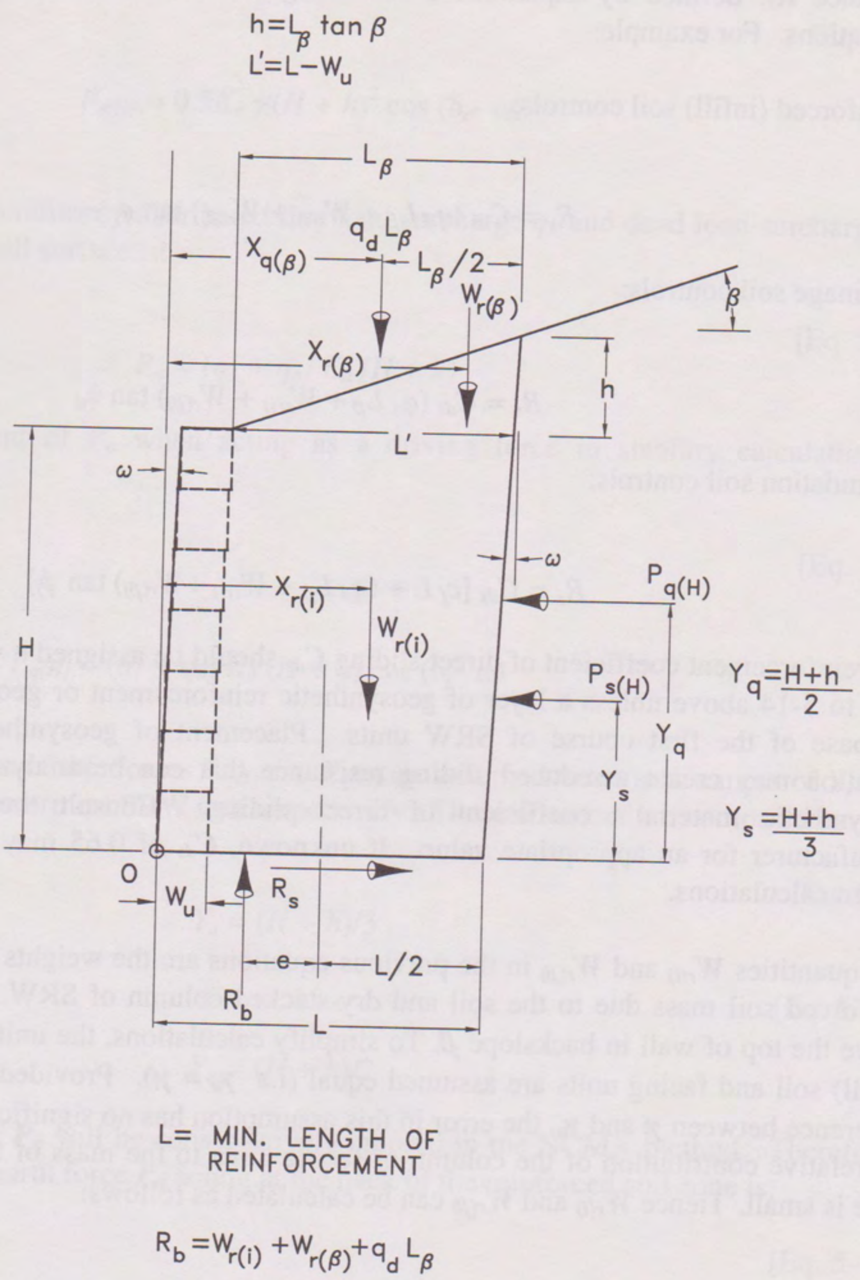


FIGURE 5-3: FREE BODY DIAGRAM FOR REINFORCED SOIL SRW EXTERNAL STABILITY CALCULATIONS

[Eq. 5-16]

$$W_{r(\beta)} = (L'\gamma_i\, h)\,/\,2 = (L'\gamma_i\; L_\beta\; \tan\beta)\,/\,2$$

Note that only the dead load portion of the distributed surcharge pressure is considered in the expressions (Equations 5-12 through 5-14) for base resistance (see Section 5.2.5).

The factor of safety $\boldsymbol{FS_{sl}}$ against base sliding is based on the smallest value of $\boldsymbol{R_s}$ calculated from Equations 5-12 through 5-14 and determined as follows:

[Eq. 5-17]

$$FS_{sl} = R_s\,/\,P_{a(H)}$$

If the factor of safety against sliding $\boldsymbol{FS_{sl}}$ is less than the target design value (typically 1.5, see **Table 5-1**) then the trial base reinforcement length $\boldsymbol{L}$ should be increased and the analysis repeated. For many routine structures, the length of the reinforced zone, and hence the minimum length of the geosynthetic reinforcement layers, will be controlled by this external base sliding failure mechanism.

5.5.3 OVERTURNING

Conventional engineering design practice for reinforced soil walls is to ensure the reinforced soil zone is stable with respect to overturning about the toe of the wall (point 0 in **Figure 5-3**). The flexibility of reinforced SRWs makes it unlikely that the gravity mass would actually overturn in practice; however, an adequate factor of safety against overturning will limit excessive outward tilting and distortion of the SRW face [Ref. 19].

The moments resisting overturning are due to the self-weight of the reinforced zone and any dead load surcharge $\boldsymbol{q_d}$ acting over the reinforced zone (i.e. over the length $\boldsymbol{L_\beta}$) as shown in **Figure 5-3**. The sum of the resisting moments $\boldsymbol{M_r}$ is calculated using Equation 5-18:

[Eq. 5-18]

$$M_r = W_{r(i)}\,X_{r(i)} + W_{r(\beta)}\,X_{r(\beta)} + q_d\,L_\beta\,X_{q(\beta)}$$

The moment arms (**X** terms in Equation 5-18) are the distances from the toe of the wall to the center of gravity of the resisting forces acting on the reinforced soil zone and are calculated as follows:

[Eq. 5-19]

$$X_{r(i)} = (L + H\;\tan\omega)/2$$

[Eq. 5-20]

$$X_{r((\beta)} = H \tan \omega + W_u + 2L'/3$$

[Eq. 5-21]

$$X_{q(\beta)} = L + [(H + h) \tan \omega] - (L_\beta /2)$$

The sum of the driving moments $\boldsymbol{M_o}$ due to the horizontal earth forces acting at the back of the reinforced soil zone are calculated as follows:

[Eq. 5-22]

$$M_o = P_{s(H)}\, Y_s + P_{q(H)}\, Y_q$$

The quantities $\boldsymbol{P_s}$, $\boldsymbol{P_q}$, $\boldsymbol{Y_s}$ and $\boldsymbol{Y_q}$ are calculated in Equations 5-6 through 5-10 and shown in **Figure 5-2**.

The factor of safety with respect to overturning $\boldsymbol{FS_{ot}}$ is calculated as follows:

[Eq. 5-23]

$$FS_{ot} = M_r/M_o$$

The magnitude of $\boldsymbol{FS_{ot}}$ is typically controlled in any design section by adjusting the length of the base reinforcement length $\boldsymbol{L}$. A typical minimum recommended value for $\boldsymbol{FS_{ot}}$ is 2.0 (see **Table 5-1**).

5.5.4 BEARING CAPACITY

Conventional bearing capacity analyses are carried out with respect to the base width $\boldsymbol{L}$ of the reinforced (infill) soil mass. The reinforced (infill) soil mass is assumed to act as a continuous strip footing and must have sufficient width $\boldsymbol{L}$ to prevent overstressing of the foundation soils that may lead to shear failure of the foundation soils or excessive settlement.

In this design manual, the conventional Meyerhof stress distribution approach is adopted. It is utilized to ensure a conservative estimate of applied bearing stress. The effect of eccentricity of the resultant bearing force (net foundation load) is to restrict compressive bearing pressures to an equivalent bearing area $\boldsymbol{B}$ calculated as:

[Eq. 5-24]

$$B = L - 2e$$

Here $\boldsymbol{e}$ is the eccentricity of the foundation load $\boldsymbol{R_b}$ (**Figure 5-3**). The quantity ($\boldsymbol{e}$) can be calculated by summing moments about the center of the base length ($\boldsymbol{L}$/2) with counter-clockwise being positive:

[Eq. 5-25]

$$e = \frac{P_{s(H)}Y_s + P_{q(H)}Y_q - W_{r(i)}\left(X_{r(i)} - L/2\right) - W_{r(\beta)}\left(X_{r(\beta)} - L/2\right) - q_d L_\beta\left(X_{q(\beta)} - L/2\right)}{W_{r(i)} + W_{r(\beta)} + q_d L_\beta}$$

The applied bearing pressure $\boldsymbol{Q_a}$ acting over the equivalent bearing width $\boldsymbol{B}$ is:

[Eq. 5-26]

$$Q_a = [W_{r(i)} + W_{r(\beta)} + (q_l + q_d)\, L_\beta]/B$$

For many projects, the site geotechnical engineer may have established an allowable bearing pressure for the foundation soils which include a settlement as well as bearing capacity criteria. The calculated $\boldsymbol{Q_a}$ should be less than the established allowable bearing pressure; if not, increase $\boldsymbol{L}$ or consult Section 5.2.8.1.

The ultimate bearing capacity $\boldsymbol{Q_{ult}}$ is calculated according to Equation 5-27:

[Eq. 5-27]

$$Q_{ult} = c_f N_c + 0.5\gamma_f B\, N_\gamma + \gamma_f H_{emb} N_q$$

The non-dimensional bearing capacity coefficients $\boldsymbol{N_c}$, $\boldsymbol{N_q}$, and $\boldsymbol{N_\gamma}$ can be found in **Figure 4-5**. The surcharge term $[\boldsymbol{\gamma_f H_{emb} N_q}]$ in the bearing capacity equation accounts for the benefits of deep ($\boldsymbol{H_{emb}}$) wall embedments. The assumption of a permanent surcharge mass must only be exercised if large excavations in front of the wall will not occur for the life of the structure. The stabilizing effects of wall embedment are applicable to bearing capacity since deformation requirements are significantly less stringent for vertical pressure rather than lateral pressure.

The factor of safety with respect to bearing capacity $\boldsymbol{FS_{bc}}$ is determined as follows:

[Eq. 5-28]

$$FS_{bc} = Q_{ult}/Q_a$$

If the value of $\boldsymbol{FS_{bc}}$ is less than the minimum design value (typically 2.0, see **Table 5-1**) the usual strategy is to incrementally increase the reinforced soil base width $\boldsymbol{L}$ and repeat the calculation set. Consult Section 5.2.8.1 for more strategies to handle difficult foundation conditions, if the required base length $\boldsymbol{L}$ is undesirable from a construction or cost perspective.

5.5.4.1 Base Eccentricity

This design manual utilizes base eccentricity $\boldsymbol{e}$ solely to calculate an equivalent footing width $\boldsymbol{B}$ to ensure a conservative calculation of applied bearing pressure. Throughout this NCMA method, the vertical stress at any point used to calculate lateral stress will be the conventional overburden stress $\boldsymbol{\sigma_v}$ (Equation 3-10), appropriate for Coulomb earth pressure theory. This assumption of a uniform vertical stress distribution is substantiated by data from instrumented test walls [Refs. 34, 36, 42, 43, 49, 51].

Section 5.6
INTERNAL STABILITY

Internal stability calculations are carried out to evaluate the integrity of the reinforced zone as a monolithic composite comprised of geosynthetic reinforcement, soil and SRW units. The tensile forces (**Figure 5-4**) to be resisted by horizontal reinforcement layers are calculated using Coulomb lateral earth pressure theory as discussed in Sections 5.2.1 and 3.4.5. Tensile overstress, pullout, and internal sliding failure modes as shown in **Figure 5-1B** must be examined.

The location of peak internal tensile load and anchorage zone within the reinforced (infill) soil mass is referenced to a failure plane that is assumed to propagate up into the reinforced (infill) soil mass from the heel of the lowermost SRW unit at an angle α_i from the horizontal (**Figure 5-5**). These assumptions are the essence of the "tied-back wedge method" of analysis and are a common element of most limit equilibrium based methods of internal stability analysis for geosynthetic reinforced soil walls [Refs. 7, 19, 39]. The tied-back wedge analysis refers to the lateral stability provided by the horizontal layers of geosynthetic reinforcement to anchor the wedge-shaped zone (IJK) of failed soil identified in **Figure 3-9**. All geosynthetic reinforcement layers must have sufficient length to develop adequate anchorage capacity beyond the internal plane of failure to prevent excessive deformation (pullout) of the reinforcement through the soil.

The tied-back wedge method for internal stability is used to determine the number, strength and vertical spacing of geosynthetic reinforcement layers in the reinforced zone. The length of the reinforced (infill) soil mass $\boldsymbol{L}$ determined from external stability calculations normally controls the length of reinforcement layers. However, in some instances the length of the uppermost geosynthetic layer(s) might have to be locally extended to provide adequate anchorage (pullout capacity). The local extension of one or more layers near the top of the wall does not affect the boundaries of the reinforced soil zone for external stability calculations (Section 5.2.4).

Incorporation of geosynthetic reinforcement into the reinforced zone may create a preferred path for outward sliding. Internal sliding analyses must be carried out to check that this failure mechanism is prevented. The potential for direct sliding increases for geosynthetics (usually geotextiles) that have a lower interface friction angle with the surrounding soil than the peak friction angle of the soil itself (i.e., $C_{ds} < 1.0$).

The location of the reinforcement layers is typically determined by a trial and error approach while recognizing that lateral earth pressures increase linearly with depth below the crest of the wall. Consequently, the vertical spacing between reinforcement layers can be expected to decrease with depth below the wall crest. The distribution of reinforcement layers will also be influenced by the method of geosynthetic facing connection (i.e., connections are made at interface elevations). The calculations, while tedious, lend themselves to implementation within computer programs.

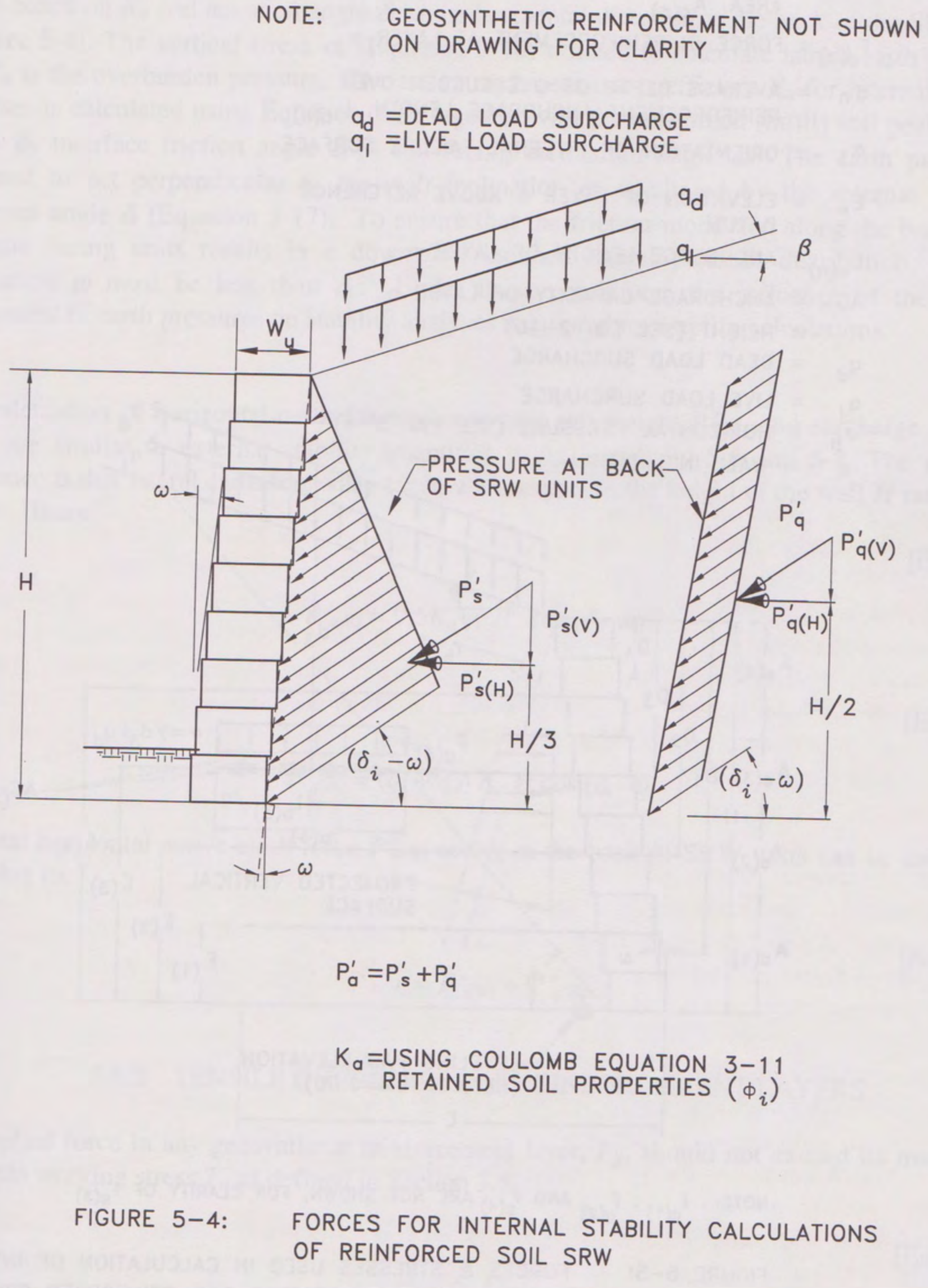


FIGURE 5–4: FORCES FOR INTERNAL STABILITY CALCULATIONS OF REINFORCED SOIL SRW

$A_{c(n)}$ = CONTRIBUTORY AREA TO DETERMINE FORCE IN REINFORCEMENT, $F_{g(n)}$

D_n = DEPTH TO MIDPOINT OF CONTRIBUTORY AREA, $A_{c(n)}$

$F_{g(n)}$ = FORCE IN REINFORCEMENT AT LAYER n

d_n = AVERAGE DEPTH OF OVERBURDEN OVER REINFORCEMENT ANCHORAGE LENGTH, $L_{a(n)}$

α_i = ORIENTATION OF INTERNAL FAILURE SURFACE

E_n = ELEVATION OF LAYER n ABOVE REFERENCE DATUM

$L_{a(n)}$ = ANCHORAGE LENGTH OF LAYER n

AC_n = ANCHORAGE CAPACITY OF LAYER n

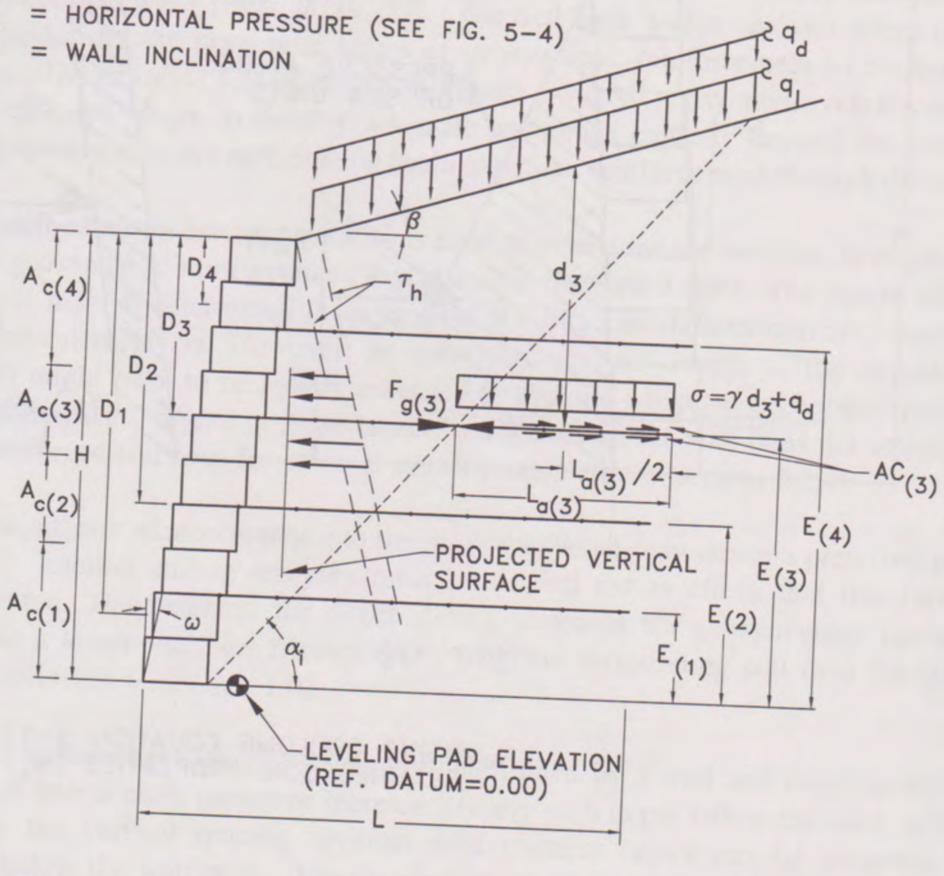
H = HEIGHT (SEE EQ. 2-10)

q_d = DEAD LOAD SURCHARGE

q_l = LIVE LOAD SURCHARGE

σ_h = HORIZONTAL PRESSURE (SEE FIG. 5-4)

ω = WALL INCLINATION



NOTE: $F_{g(1)}$, $F_{g(2)}$ AND $F_{g(4)}$ ARE NOT SHOWN, FOR CLARITY OF $F_{g(3)}$.

FIGURE 5-5: FORCES & STRESSES USED IN CALCULATION OF INTERNAL STABILITY FOR GEOSYNTHETIC SOIL REINFORCED SRW

5.6.1 EARTH PRESSURES

For internal stability calculations, the lateral earth pressure due to reinforced (infill) soil self-weight and imposed surcharge loadings (q_d and q_l) is assumed to be linearly distributed with depth based on K_a and act at an angle δ_i to the horizontal direction at the back of the SRW units (**Figure 5-4**). The vertical stress σ_v (Equation 3-10) utilized to calculate lateral earth pressures via K_a is the overburden pressure. The active earth pressure coefficient K_a for internal stability analyses is calculated using Equation 3-11 together with the reinforced (infill) soil peak friction angle ϕ_i, interface friction angle δ_i, and facing inclination angle ω. The earth pressure is assumed to act perpendicular to the wall inclination ω as altered by the internal interface frictional angle δ_i (Equation 3-17). To ensure that the friction mobilized along the back of the concrete facing units results in a downward inclined lateral pressure distribution, the wall inclination ω must be less than δ_i. Under these conditions, the influence of the vertical component of earth pressures on stability analyses is ignored to simplify calculations.

The calculation of horizontal earth forces due to soil self-weight $P'_{s(H)}$ and surcharge loadings $P'_{q(H)}$ are similar to external stability quantities using Equations 5-6 and 5-8. The principle difference is that lateral earth forces are taken with respect to the height of the wall H rather than $H + h$. Hence:

[Eq. 5-29]

$$P'_{s(H)} = 0.5 K_a \gamma_i H^2 \cos(\delta_i - \omega)$$

and

[Eq. 5-30]

$$P'_{q(H)} = (q_l + q_d) K_a H \cos(\delta_i - \omega)$$

The total horizontal active earth force $P'_{a(H)}$ acting at the back of SRW units can be calculated according to:

[Eq. 5-31]

$$P'_{a(H)} = P'_{s(H)} + P'_{q(H)}$$

5.6.2 TENSILE OVERSTRESS OF REINFORCEMENT LAYERS

The applied force in any geosynthetic reinforcement layer, F_g, should not exceed its maximum allowable working stress T_a as defined in Section 3.5.

[Eq. 5-32]

$$F_{g(n)} \leq T_{a(n)}$$

5.6.2.1 Minimum Number of Reinforcement Layers

Estimating the minimum number of reinforcement layers N_{min} required to satisfy horizontal equilibrium at the back of the dry-stacked column of SRW units is the initial design step. The value calculated using the following expressions should be rounded up to the nearest whole number.

For designs using a single reinforcement type:

$$N_{min} = P'_{a(H)} / T_a \quad \text{[Eq. 5-33]}$$

For multiple reinforcement types:

$$N_{min} = N_1 + N_2 + \ldots + N_t \quad \text{[Eq. 5-34]}$$

where:

$$P'_{a(H)} = T_{a(1)}(N_1) + T_{a(2)}(N_2) + \ldots + T_{a(n)}(N_n) \quad \text{[Eq. 5-35]}$$

The quantity $\boldsymbol{P'_{a(H)}}$ is the active earth force acting over the height of the wall $\boldsymbol{H}$. The quantity $\boldsymbol{T_{a(t)}}$ is the long-term allowable design strength of reinforcement type $\boldsymbol{t}$ and $\boldsymbol{N_t}$ the total number of layers for that type $\boldsymbol{t}$. Equation 5-34 allows any combination of design strengths (types $\boldsymbol{t}$), but the common strategy is to use a single reinforcement type to facilitate construction. Alternatively, reinforcement materials that decrease in design strengths with increasing elevation above the bearing pad can be used to help keep vertical reinforcement layer spacings constant. $\boldsymbol{N_{min}}$ should always be rounded up to the nearest whole number of layers. Throughout this manual the subscripts $\boldsymbol{n}$ and $\boldsymbol{N}$ indicate the reinforcement layer under consideration and the total number of reinforcement layers to be analyzed, respectively.

5.6.2.2 Vertical Spacing of Reinforcement Layers

The vertical location (elevation $\boldsymbol{E_{(n)}}$) and number of reinforcement layers should be selected based on $\boldsymbol{N_{min}}$, SRW unit height $\boldsymbol{H_u}$ and recognizing that earth pressures will increase linearly with depth. If a wall project comprises an alignment with variable footing grade an effort is often made to preserve reinforcement elevations across wall sections to facilitate construction. If one or more reinforcement materials are used, it is recommended that the stronger materials be placed at the lower elevations. The reinforcement layers, $\boldsymbol{n}$, and types, $\boldsymbol{t}$, should increase numerically from the base of the wall up.

5.6.2.3 Effective Elevation of Reinforcement Layer

The convention in this manual is to use $\boldsymbol{E_{(n)}}$ to denote elevations in a vertical plane. The datum is the elevation of the heel of the lowermost SRW unit (i.e., leveling pad elevation, see **Figure 5-5**). The effect of battered wall and actual joint location is ignored to simplify analysis (the effect can be shown to be negligible.).

5.6.2.4 Calculation of Applied Tensile Load

The tensile load developed in a layer of geosynthetic reinforcement is based on the contributory area $A_{c(n)}$ of the layer and the integration of lateral pressure over the effective height of the wall defined by the contributory area (**Figure 5-5**). The total applied tensile force in the geosynthetic reinforcement $F_{g(n)}$ can be calculated using the average horizontal pressure at the midpoint of the contributory area as follows:

[Eq. 5-36]

$$F_{g(n)} = [\gamma_i\, D_n + q_l + q_d]\, K_a\, A_{c(n)} \cos(\delta_i - \omega)$$

a.) Reinforcement Contributory Area: The contributory area $A_{c(n)}$ for any reinforcement elevation is defined as the midpoint between adjacent reinforcement elevations or the between the top and bottom of the wall. Hence, for the lowermost layer:

[Eq. 5-37]

$$A_{c(1)} = (E_{(2)} + E_{(1)})\,/\,2$$

For any intermediate layer n:

[Eq. 5-38]

$$A_{c(n)} = [(E_{(n+1)} + E_{(n)})\,/\,2] - [(E_{(n)} + E_{(n-1)})\,/\,2]$$

which simplifies to:

[Eq. 5-39]

$$A_{c(n)} = (E_{(n+1)} - E_{(n-1)})\,/\,2$$

For the topmost layer N:

[Eq. 5-40]

$$A_{c(n)} = H - [(E_{(n)} + E_{(n-1)})\,/\,2]$$

b.) Midpoint of Contributory Area: To calculate the force in a geosynthetic reinforcement layer $F_{g(n)}$, the depth D_n below the crest of the wall to the midpoint of the contributory area $A_{c(n)}$ must be determined for that layer n to calculate the average pressure. Generally, for non-uniform vertical spacing, the midpoint of the contributory area will be different from the placement elevation $E_{e(n)}$. The D_n to the midpoint of a contributory area $A_{c(n)}$ can be calculated as follows for the lowermost layer.

[Eq. 5-41]

$$D_1 = (H + h) - (A_{c(1)}/2)$$

For any intermediate layer $\boldsymbol{n}$:

[Eq. 5-42]

$$D_n = (H + h) - A_{c(1)} - A_{c(2)} - \ldots - A_{c(n-1)} - (A_{c(n)}/2)$$

For the uppermost layer $\boldsymbol{N}$:

[Eq. 5-43]

$$D_N = (A_{c(N)}/2)$$

5.6.3 PULLOUT OF REINFORCEMENT

The applied tensile force $\boldsymbol{F_{g(n)}}$ in the geosynthetic reinforcement must be transferred to the soil through the development of an anchorage capacity beyond the active wedge of soil movement defined by a failure surface inclined to the horizontal at α_i (**Figure 5-5**). Pullout of reinforcement layers is prevented by sufficient anchorage capacity which maintains a coherent mass of soil in the reinforced SRW. The ratio of the developed anchorage capacity $\boldsymbol{AC_n}$ to the applied force $\boldsymbol{F_{g(n)}}$ in any geosynthetic reinforcement layer is designated by the factor of safety against pullout $\boldsymbol{FS_{po}}$. This represents a reasonable assessment of the geosynthetic material's potential to resist pullout from the soil. Routinely, the uppermost layer of reinforcement is most critical due to less overburden pressure and anchorage length.
The factor of safety against pullout failure is calculated as follows:

[Eq. 5-44]

$$FS_{po} = AC_n / F_{g(n)}$$

The factor of safety against reinforcement pullout should be greater than the minimum required for design (typically 1.5, see **Table 5-1**). If the factor of safety of any layer does not satisfy the specified design value, it can usually be increased in length or placed at a lower level to accommodate the design. There is no requirement in this manual for reinforcement layers to have uniform length, with the exception that each be equal to or greater than the minimum base width length $\boldsymbol{L}$ established from external stability calculations.

5.6.3.1 Anchorage Capacity of Reinforcement

The anchorage capacity $\boldsymbol{AC_n}$ of geosynthetic reinforcement is related directly to the available soil shear strength through the coefficient of interaction for pullout parameter, $\boldsymbol{C_i}$. The magnitude of anchorage capacity is controlled by the anchorage length $\boldsymbol{L_{a(n)}}$, which is the portion of length for a specific geosynthetic layer beyond the failure surface α_i (see **Figure 5-5**). The minimum anchorage length for structures is one foot. The anchorage capacity $\boldsymbol{AC_n}$ of the geosynthetic reinforcement is assumed to be proportional to the anchorage length $\boldsymbol{L_{a(n)}}$, peak shear strength of anchorage soil (tan ϕ_i), coefficient of interaction for pullout $\boldsymbol{C_i}$ and the depth of soil overburden $\boldsymbol{d_n}$ acting over the anchorage length.

The anchorage capacity AC_n is calculated as follows:

[Eq. 5-45]

$$AC_n = 2\, L_{a(n)}\, C_i\, (d_n\, \gamma_i + q_d) \tan \phi_i$$

Only the dead load component of the uniform surcharge distribution enters the calculation above in order to be consistent with treatment of the dead and live surcharge loads described in Section 5.2.5. The coefficient 2 in the above equation is typical of many anchorage models and reflects resistance being mobilized on both the top and bottom of the geosynthetic layer and is consistent with the definition of C_i in Section 3.5.2.

a.) Anchorage Length of Geosynthetic: The anchorage length $L_{a(n)}$ for any geosynthetic layer within the reinforced zone ϕ_i can be calculated as follows:

[Eq. 5-46]

$$L_{a(n)} = L - W_u - E_{(n)} \tan (90 - \alpha_i) + E_{(n)} \tan \omega$$

b.) Depth of Overburden on Anchorage Length: The available soil shear strength that can be transferred to the geosynthetic reinforcement to resist pullout is controlled by the normal overburden stress acting over the anchorage length $L_{a(n)}$. The average normal overburden pressure is proportional to the average overburden depth d_n (i.e., at the midpoint of the anchorage length) that is calculated accordingly:

[Eq. 5-47]

$$d_n = (H - E_{(n)}) + [(E_{(n)} / \tan \alpha_i) - H \tan \omega + (L_{a(n)}/2)] \tan \beta$$

5.6.4 INTERNAL SLIDING FAILURE

The potential for an internal sliding failure to propagate along the surface of a reinforcement layer must be examined for each reinforcement elevation $E_{(n)}$ (**Figure 5-6**). The potential for this type of failure mechanism increases as the shear resistance between the soil and reinforcement material decreases. The shear resistance available at a geosynthetic surface is described by parameter R'_{sn} (Equation 5-49) in this manual and is proportional to the coefficient of direct sliding C_{ds} of the geosynthetic material. The method of test to determine C_{ds} is described in Section 3.5.2.2. The maximum value of C_{ds} is 1.0. The failure plane generated along the surface of a geosynthetic layer will also have to propagate through the interface between SRW units. This interface will provide some additional shear capacity described by parameter $V_{u(n)}$.

The potential for an internal sliding failure should be examined for each reinforcement elevation. The driving force $P_{a(H,n)}$ for the internal sliding failure is calculated using Equations 5-1 through 5-11, as provided in Section 5.5.1 and substituting $L'_{s(n)}$ for L', and $(H - E_{(n)} + h)$ for $(H + h)$. The factor of safety against sliding FS_{sl} can then be calculated as follows:

[Eq. 5-48]

$$FS_{sl(n)} = [R'_{s(n)} + V_u] / P_{a(H,n)}$$

The $FS_{sl(n)}$ at each geosynthetic reinforcement level should be greater than that required (typically 1.5, see **Table 5-1**). For substandard FS_{sl}, the overall length of geosynthetic reinforcement L must be increased for the layer analyzed and all reinforcement layers above it. Alternatively, the reinforcement placement elevations may be altered to reduce the sliding force $P_{a(H,n)}$ and/or increase the length L'_s over which sliding occurs .

5.6.4.1 Sliding Resistance Over Geosynthetic Reinforcement

The sliding resistance over the geosynthetic reinforcement $R'_{s(n)}$ is calculated as follows:

[Eq. 5-49]

$$R'_{s(n)} = C_{ds}\,(q_d\,L_{\beta(n)} + W'_{r(i,n)} + W'_{r(\beta,n)})\tan\phi_i$$

In this equation, $W'_{r(i,n)}$ and $W'_{r(\beta,n)}$ are the weights of soil at the sliding surface above and below the top of wall. $L_{\beta(n)}$ is the length of the geosynthetic reinforcement over which the uniform dead load surcharge q_d and backslope β are active.

a.) <u>Length of Geosynthetic for Sliding:</u> The length of geosynthetic reinforcement $L'_{s(n)}$ over which sliding can occur is controlled by the vertical spacing between layers (**Figure 5-6**).

The geosynthetic sliding surface length $L'_{s(n)}$ can be calculated as:

[Eq. 5-50]

$$L'_{s(n)} = L - (W_u) - \Delta L$$

[Eq. 5-51]

$$\Delta L = (E_{(n+1)} - E_{(n)}) / \tan\alpha_e$$

The reduction in resisting length ΔL is calculated using the external failure surface orientation α_e (Equation 3-14). The reinforced structure width $L_{\beta(n)}$ over which the backslope β and surcharge q_d are active is calculated as follows:

[Eq. 5-52]

$$L''_{s(n)} = \frac{L'_{s(n')}\tan\beta\tan\omega}{1-\tan\beta\tan\omega}$$

[Eq. 5-53]

$$L_{\beta(n)} = L'_{s(n)} + L''_{s(n)}$$

[Eq. 5-54]

$$h_{(n)} = L_{\beta(n)}\tan\beta$$

$P_{a(n)}$ = TOTAL ACTIVE EARTH FORCE GENERATING SLIDING ON GEOSYNTHETIC n

$W'_{r(n,i)}$ = WEIGHT OF SOIL ON GEOSYNTHETIC n

$L_{\beta(n)}$ = LENGTH OF REINFORCEMENT IN INTERNAL SLIDING ACTED UPON BY BACKSLOPE

$L'_{s(n)}$ = LENGTH OF REINFORCEMENT USED IN INTERNAL STABILITY ANALYSIS

L = LENGTH OF REINFORCEMENT

α_e = ORIENTATION OF EXTERNAL FAILURE PLANE

$R'_{s(n)}$ = SHEAR RESISTANCE ALONG GEOSYNTHETIC

$E_{(n)}$ = ELEVATION OF LAYER n ABOVE REFERENCE DATUM

V_h = SHEAR CAPACITY OF SRW UNIT

q_l = LIVE LOAD UNIFORM SURCHARGES

q_d = DEAD LOAD UNIFORM SURCHARGES

$W'_{r(\beta,n)}$ = WEIGHT OF SLOPE ABOVE THE REINFORCED ZONE

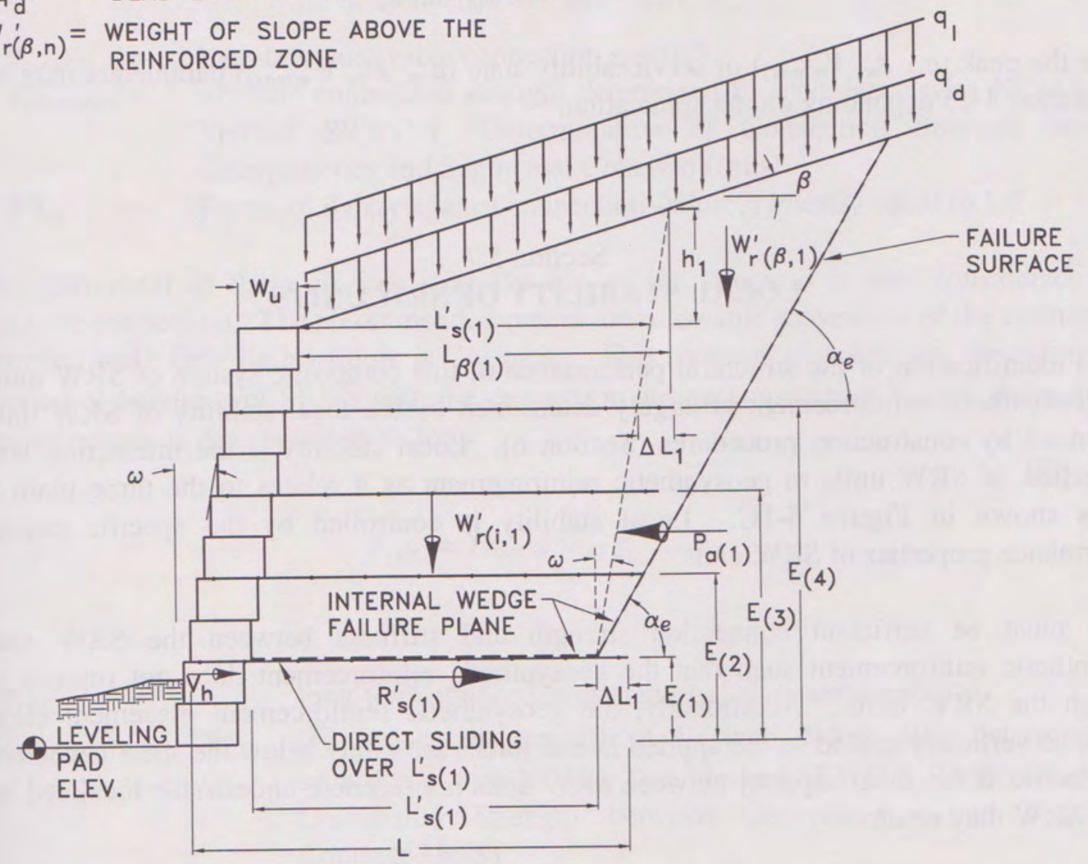


FIGURE 5-6: INTERNAL SLIDING ANALYSIS FOR REINFORCED SOIL SRWs

b.) Weight of Soil on Geosynthetic Reinforcement: The weight of soil, $W'_{r(i,n)}$ and $W'_{\beta(r,n)}$, acting on $L'_{s(n)}$ is calculated as follows using the geometry illustrated in **Figure 5-6**:

[Eq. 5-55]

$$W'_{r(i,n)} = L'_{s(n)}\,(H - E_{(n)})\,\gamma_i$$

[Eq. 5-56]

$$W'_{r(\beta,n)} = (\gamma_i\, L_{\beta(n)}\; L'_{s(n)} \tan\beta)/2$$

5.6.4.2 Shear Capacity Between SRW Units

The shear capacity $V_{u(n)}$ at the interface between SRW units which will resist the internal sliding failure can be calculated by using Equation 4-25 in Section 4.6. The shear capacity will be controlled by the weight of SRW units W_w acting on the interface and can be determined using Equation 4-9 while implementing the hinge height Equations 4-1 or 4-2, if applicable.

[Eq. 4-25]

$$V_{u(n)} = a_u + W_{W(n)} \tan\lambda_u$$

Either the peak (a_u, λ_u, $V_{u(max)}$) or serviceability state (a'_u, λ'_u, $V'_{u(max)}$) parameters may be used in Equation 4-25 depending on the application.

Section 5.7
LOCAL STABILITY OF SRW UNITS

Visual identification of the structural performance of this composite system of SRW units, soil and geosynthetic reinforcement is largely determined by the local stability of SRW units and influenced by construction procedures (Section 6). Local stability is the interaction with and connection of SRW units to geosynthetic reinforcement as it relates to the three main failure modes shown in **Figure 5-1C**. Local stability is controlled by the specific engineering performance properties of SRW units.

There must be sufficient connection strength and stiffness between the SRW unit and geosynthetic reinforcement such that the geosynthetic reinforcement does not rupture or pull through the SRW units. Additionally, the geosynthetic reinforcement placement elevations should be vertically spaced so the applied lateral forces are safely below the shear capacity of the SRW units. If the shear capacity between SRW units is exceeded, undesirable localized bulging of the SRW may result.

The SRW units near the top of the wall must be examined to ensure they are stable as a free standing retaining wall above the highest reinforcement placement elevation. The examination of the upper unreinforced SRW height for sliding and overturning failure modes is performed in the same manner as the gravity SRW analysis (Sections 4.5.3 and 4.5.4).

5.7.1 FACING CONNECTION STRENGTH

The facing between the geosynthetic reinforcement and SRW unit at each reinforcement placement elevation $E_{(n)}$ must have sufficient connection strength to preclude rupture or slippage of the reinforcement due to the applied tensile force.

There are two criteria which should be addressed when designing the connection of a geosynthetic reinforced SRW. The limit state strength of the connection at failure, and a serviceability state, the strength of the connection at a specified deformation. By considering both these criteria the connection will have the required long-term strength and will have acceptable deformation.

The ultimate strength of the connection is evaluated using the limit state connection strength as determined below:

[Eq. 5-57]

$$T_{cA(n)} = T_{ultconn(n)} / FS_{cs} \leq T_{a(n)}$$

where:

$T_{cA(n)}$ = long-term allowable connection strength

$T_{ultconn(n)}$ = ultimate connection strength determined in accordance with NCMA Test Method SRWU-1 "Determination of Connection Strength between Geosynthetics and Segmental Concrete Units"

FS_{cs} = Factor of Safety against connection failure, typically equal to 1.5.

Limiting movement of the wall face over the life of the structure is also considered when designing the connection. The recommended maximum allowable movement of the connections to minimize wall face deformation is $^3/_4$ inch. This criteria is based on the successful construction of hundreds of SRWs with the connection design considering $^3/_4$ inch deformation. The service criteria is determined as follows:

[Eq. 5-58]

$$T_{cs(n)} = T_{conn\ @\ 3/4(n)} \leq T_{a(n)}$$

where:

$T_{cs(n)}$ = long-term connection strength based on serviceability

$T_{conn\ @\ 3/4(n)}$ = the connection strength at $^3/_4$ inch deformation determined in accordance with NCMA Test Method SRWU-1 (Determination of Connection Strength between Geosynthetics and Segmental Concrete Units)

The connection strength ($T_{ultconn(n)}$) relationship between any specific combination of geosynthetic reinforcement and height of SRW units can be determined through laboratory testing as defined in Section 3.2.3.1 and Appendix C . Laboratory testing is required to establish the design facing connection strength relationship parameters, a_{cs} and λ_{cs}, to relate connection strength $T_{ultconn(n)}$ and applied normal load. Both the limit (a_{cs}, λ_{cs}, $T_{ultconn}$) and service (a'_{cs}, λ'_{cs}, T_{conn} @ ¾$_{(n)}$) state parameters should be determined.

The connection strength will be influenced by the weight of SRW units $W_{w(n)}$ (Equation 4-9) acting on the interface (H - $E_{(n)}$) and the hinge height H_h (Equations 4-1 and 4-2). The connection strength $T_{ultconn(n)}$ at each geosynthetic reinforcement placement elevation $E_{(n)}$ can be calculated as:

[Eq. 5-59]

$$T_{ultconn(n)} = a_{cs} + W_{w(n)} \tan \lambda_{cs}$$

The serviceability connection strength established at $^3/_4$ inch deformation is calculated as:

[Eq. 5-60]

$$T_{conn} \text{ @ } {}^{3/4}{}_{(n)} = a'_{cs} + W_{w(n)} \tan \lambda'_{cs}$$

The allowable connection strength $T_{aconn(n)}$ shall be the least of the limit state connection strength service state connection strength and the allowable strength of the geosynthetic (T_a).

The connection must be able to resist the tensile force that is transferred from the reinforced soil mass to the connection of a SRW system ($F_{g(n)}$). The tensile force that must be resisted by the connection is a function of the vertical spacing of the reinforcement and proximity of the internal failure surface to the connection (**Figure 5-5**). The maximum tensile force that the connection must resist is the applied tensile force $F_{g(n)}$.

The facing connection between the geosynthetic reinforcement and SRW units at each reinforcement placement elevation $E_{(n)}$ must have sufficient connection strength to preclude rupture or slippage of the reinforcement due to the applied tensile force $F_{g(n)}$.

5.7.2 RESISTANCE TO BULGING

Bulging of a SRW in the vertical plane occurs when a SRW unit does not maintain its relative position with respect to the SRW units above and below it. The relative position of one course to the next is maintained by shear resistance. Therefore, for reinforced soil SRWs, all units must possess sufficient shear capacity to resist the theoretical horizontal earth pressure being applied between layers of geosynthetic reinforcement. Consequently, resistance to bulging is controlled by the magnitude of applied pressure, vertical spacing of geosynthetic reinforcement, and shear capacity between SRW units.

The shear force applied to the SRW units varies with location along the wall as shown in **Figure 5-7A**. For analysis of bulging, the dry-stacked SRW units are modeled as a continuous beam subjected to a continuous distributed load (i.e., earth pressure). Using a simplified equivalent beam method [Ref. 15] as proposed for flexible tied-back steel sheet pile retaining walls, a shear force diagram can be generated. The shear force diagram example shown in **Figure 5-7A** illustrates that the theoretical maximum shear forces occur at geosynthetic reinforcement elevations.

The shear force diagram is constructed by summing the out-of-balance horizontal forces above each interface elevation $E_{(n)}$ starting from the top of the wall and proceeding to the bottom of the wall. The out-of-balance force at any interface elevation $E_{(n)}$ must be carried by mobilized shearing resistance at that elevation. The maximum applied shear (out-of-balance) force at any interface is the difference between the horizontal pressure and the available geosynthetic reinforcement tension above that interface.

The shear capacity $V_{u(n)}$ at any interface is calculated using Equation 4-25 in Section 4.6. The shear capacity will be controlled by the weight of SRW units $W_{w(n)}$ acting on the interface which can be determined using Equation 4-9 and implementing the hinge height Equation 4-1.

[Eq. 4-25]

$$V_{u(n)} = a_u + W_{w(n)} \tan \lambda_u$$

Either the peak (a_u, λ_u, $V_{u(max)}$) or service state (a'_u, λ'_u, $V'_{u(max)}$) parameters may be used in Equation 4-25, depending on the structure.

The maximum applied tensile force in each geosynthetic reinforcement layer $F_{g(n)}$ may be taken as that calculated by Equation 5-35 in Section 5.6.2.4.

The total horizontal earth force $P'_{a(H)}$ is calculated using Equation 5-31 for each intermediate wall height $H_{(n)}$ along with Equations 5-29 and 5-30 to determine $P'_{s(H)}$ and $P'_{q(H)}$. The distribution of earth pressure shown in **Figures 5-7** and **5-4**, are governed by the reinforced (infill) soil properties ϕ_i. The active earth pressure coefficient K_a is calculated using Equation 3-11, the backslope angle β and the reinforced (infill) soil properties ϕ_i, δ_i.

The factor of safety against horizontal sliding FS_{sl} and/or shear capacity FS_{sc} would apply to the calculation of bulging resistance. Since the resistance component of greatest concern is shear capacity, calculate bulging resistance as:

[Eq. 5-61]

$$FS_{sc(n)} = (V_{u(n)}) / [P'_{a(H,n)} - (F_{g(n+1)} + F_{g(n+2)} + \ldots)]$$

The factor of safety against shear capacity $FS_{sc(n)}$ (bulging) should be greater than the required minimum (typically 1.5, see **Table 5-1**) for peak load criterion using parameters a_u and λ_u. Additionally, to reduce lateral deformations at SRW unit facing the $FS_{sc(n)}$ at each reinforcement elevation should meet the minimum specified design value for service state criterion using parameters a'_u and λ'_u. The magnitude of $FS_{sc(n)}$ may be increased by either altering the vertical spacing and/or increasing the number of layers of geosynthetic reinforcement. Since the applied shear force is highly dependent on vertical spacing of the reinforcement (**Figure 5-7A**) each reinforcement placement elevation should be analyzed.

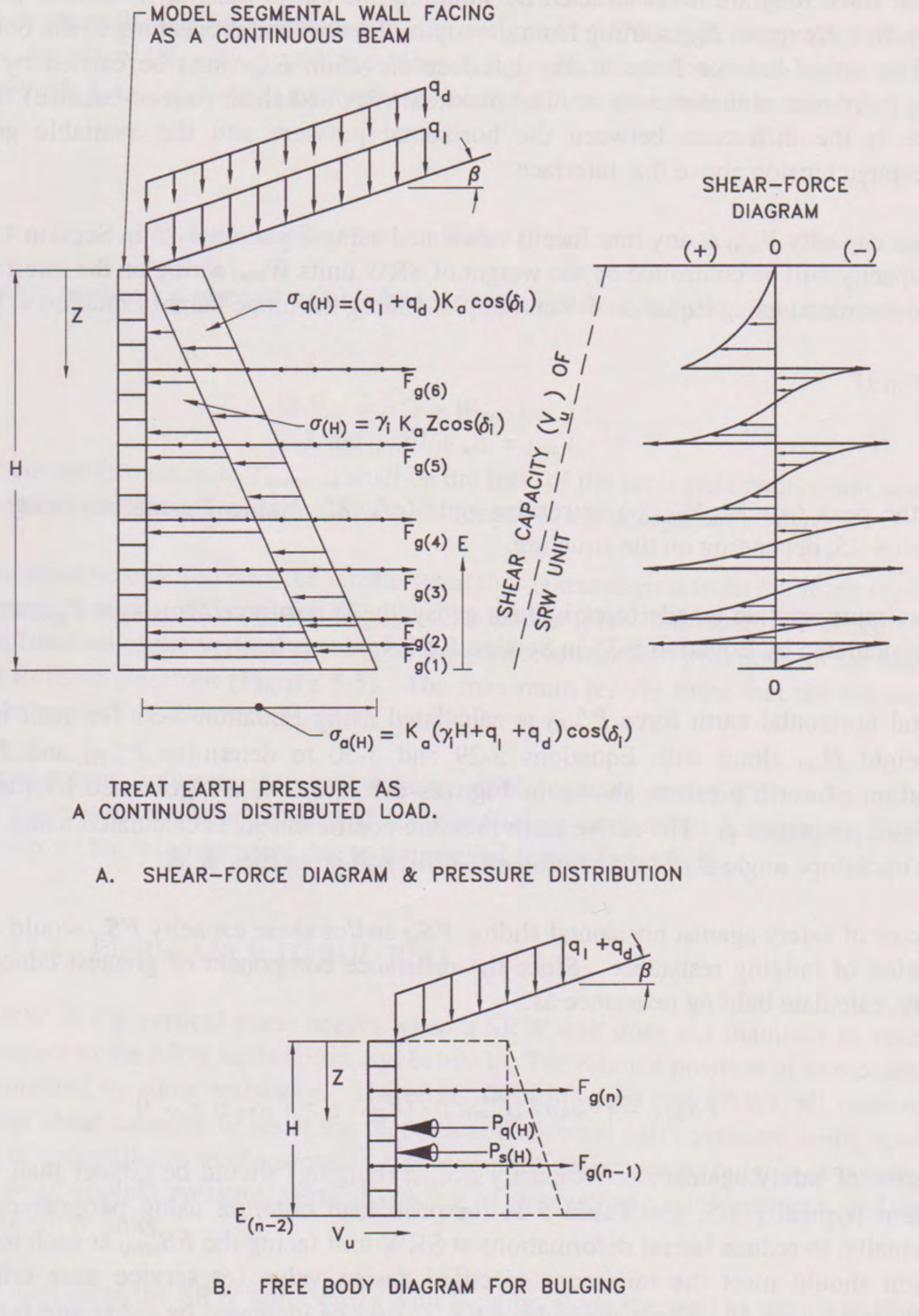


FIGURE 5-7: SHEAR-FORCE ANALYSIS FOR BULGING OF SRW UNITS

5.7.3 MAXIMUM UNREINFORCED SRW HEIGHTS

The SRW units above the highest reinforcement placement elevation must be examined to ensure they will perform as a free standing retaining wall. The examination of the upper unreinforced SRW height for sliding (shear) and overturning failure modes is done in the same manner as the conventional SRW analysis, Sections 4.5.3 and 4.5.4 respectively, using the minimum safety factors established for critical structures in **Table 4-1**.

The shear capacity $V_{u(n)}$ at any interface should be calculated using Equation 4-25 in Section 4.6. The shear capacity will be controlled by the weight of SRW units $W_{w(n)}$ acting on the interface which can be determined using Equation 4-9 and implementing the hinge height (Equation 4-1).

[Eq. 4-25]

$$V_{u(n)} = a_u + W_{w(n)} \tan \lambda_u$$

Either the peak (a_u, λ_u, $V_{u(max)}$) or service state (a'_u, λ'_u, $V'_{u(max)}$) parameters may be used in Equation 4-25, depending on the structure.

The reinforced SRW design is complete when the FS_{sl} and FS_{ot} for the intended maximum unreinforced height exceed the minimum required safety factors (**Table 4-1**). Otherwise, if an unacceptable FS_{sl} or FS_{ot} is identified, the maximum unreinforced height should be reduced by incorporating an additional layer of reinforcement near the top of the wall or adjusting the vertical spacing of the existing reinforcement layout.

Section 5.8
EXAMPLE CALCULATIONS

An example calculation using this design methodology to analyze a typical design problem for a generic reinforced soil SRW is presented in Appendix B. The example calculation serves to illustrate many of the consequences of certain design property assumptions and important intermediate calculation steps.

SECTION 6
CONSTRUCTION OF SEGMENTAL RETAINING WALLS

In addition to proper engineering, long-term structural performance of SRWs is directly influenced by the construction procedures. By adhering to good construction practices SRWs can provide a long service life both functionally and aesthetically. Provided throughout this section are guidelines for the construction of SRWs which, by experience, have proven successful.

Section 6.1
CONSTRUCTION DRAWINGS AND SPECIFICATIONS

A successful project always begins with appropriate planning and scheduling. The preceding sections of this manual provide details on the engineering design of SRWs. By performing those engineering analyses, a set of construction plans can be generated detailing the specifics to build the SRW which should include:

- plan location of SRWs, including location limits of top and bottom of walls
- profile dimensions, including elevations of top and bottom of wall and, for reinforced soil walls, elevations of reinforcement
- typical cross-sections
- drainage details, both surface and subsurface features
- details (i.e., bearing pad, geosynthetic reinforcement to SRW unit connection for reinforced soil walls), wall abutment to other structures, wall termination, and geosynthetic layout around utilities or other obstructions, as applicable
- construction specifications, consistent with the construction drawings.

Guide specifications for SRWs are provided in Section 7 of this design manual.

Section 6.2
CONSTRUCTION DETAILS

The layout of curves and corners for SRWs requires planning by both the design engineer and contractor. The varying horizontal setback per course (ω) among different types of SRW units must be considered prior to construction. This variable will dictate actual layout in plan and elevation. Leveling pad location will step up and back as elevation increases due to the horizontal setback per course (ω). The setback and inclination angles also create larger or smaller radii (lengths of curved wall) as the SRW increases in height, depending upon either a concave or convex orientation. These potential changes in length and elevations must be accounted for in plan and field construction layout of the wall to assure the minimum radius is not encroached upon and that project requirements are met.

End of Section

It is recommended the leveling pad beneath the SRW units be constructed of drainage fill. An advantage of using drainage fill is that it is free draining and allows placement of the drain pipe in the lower, inward corner of the pad. Caution should be exercised in leveling the drainage fill to ensure intimate contact between the units and aggregate. A sand material is easier to level and ensure intimate contact with SRW units. The sand should be a well draining material (i.e., less than fifteen percent passing #200 sieve) and be in contact with a geotextile wrapped discharge pipe. The drain fill above a sand bearing pad may be more permeable (i.e., capable of carrying more water) which could result in water flowing out the face of the wall. A designer or a contractor may opt to use an unreinforced concrete leveling pad on some projects. Use of unreinforced concrete should be limited to sites with firm foundations. The potential disadvantages of using unreinforced concrete for the leveling pad are difficulty in layout of vertical and horizontal steps; maintaining intimate contact between the leveling pad and SRW unit; and a loss of flexibility.

For reinforced soil walls, specific details on placement of geosynthetic reinforcement at wall corners should be provided in the construction drawings. Two typical details are provided as **Figures 6-1** and **6-2**.

Occasionally, SRWs will encounter utilities or other features which must pass through, under, or within the infill soil zone. Special details should be presented in the construction drawings which incorporate the encroaching utility or feature. If the site planning process and time permits, routing utilities around the SRW tends to be most cost effective.

Section 6.3
CONSTRUCTION PLANNING

The execution of construction operations for SRWs is dependent on quality surveying information, both to plan and field locate its proper position. The existing and proposed finish grades shown on the drawings should be verified in the field to ensure the planned design heights are in agreement with topographic information from the project grading plan. Once located in the field, it is good practice to have the retaining wall location verified by the owner or the owner's engineer. Any changes in wall location made in the field should be duly noted on as-built drawings prior to finishing the project.

Delivery and storage of all retaining wall materials should be coordinated to ensure maximum access to the work area and availability during construction. This is particularly important for soil used to construct the wall which may have to be placed with a specified moisture content. Likewise, the geosynthetic reinforcement should be stored according to the manufacturer's recommendations.

There are two basic topographical conditions in which SRWs may be constructed; "cut" and "fill". The differences between the two are illustrated in **Figure 6-3**. The construction approach, schedule, and cost will be dictated by the type of wall that is required at a site. Additionally, the effects of construction on existing nearby structures and parking areas must be carefully considered for "cut" walls so that foundation support of those structures is not undermined or encroached upon in any way.

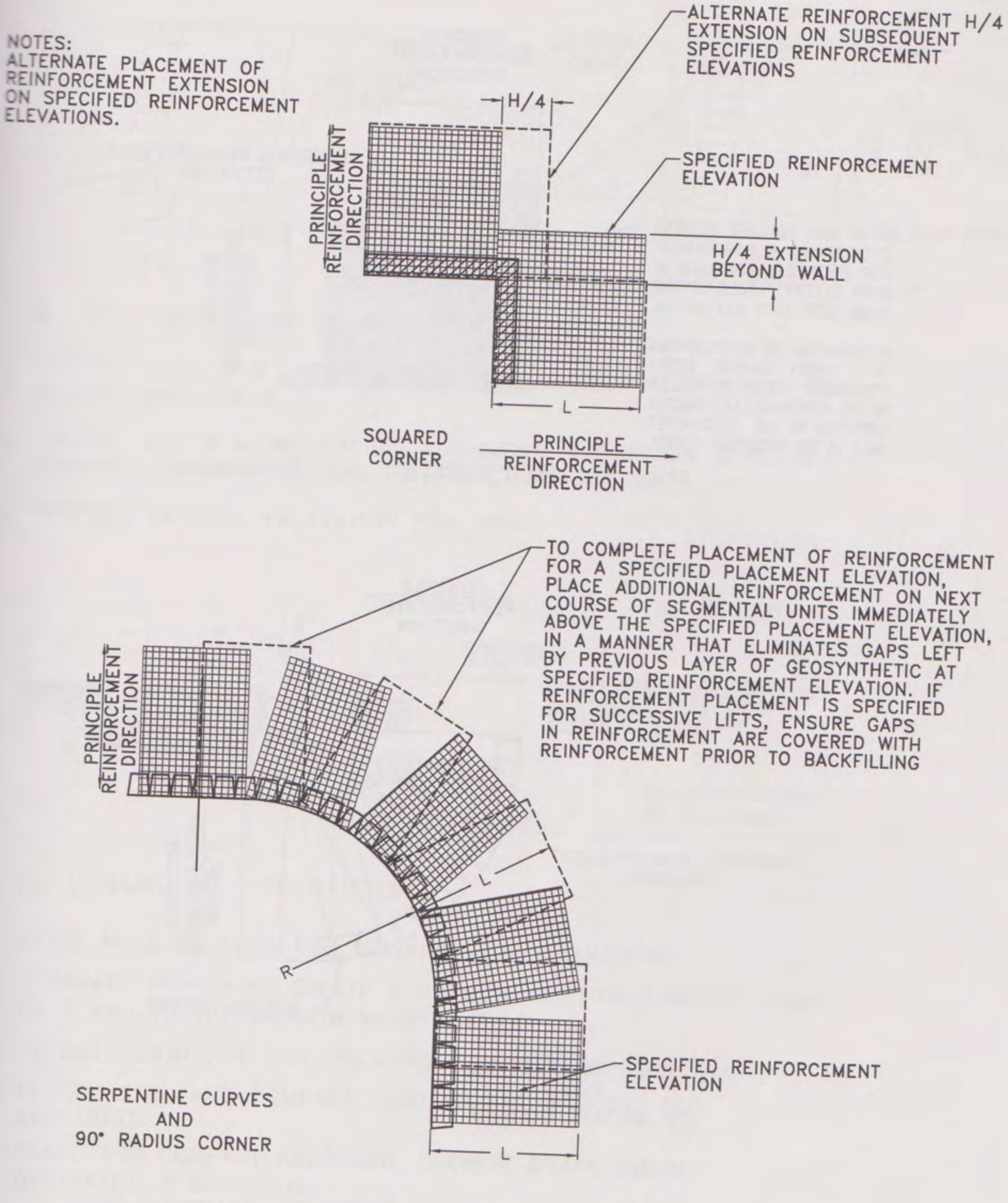
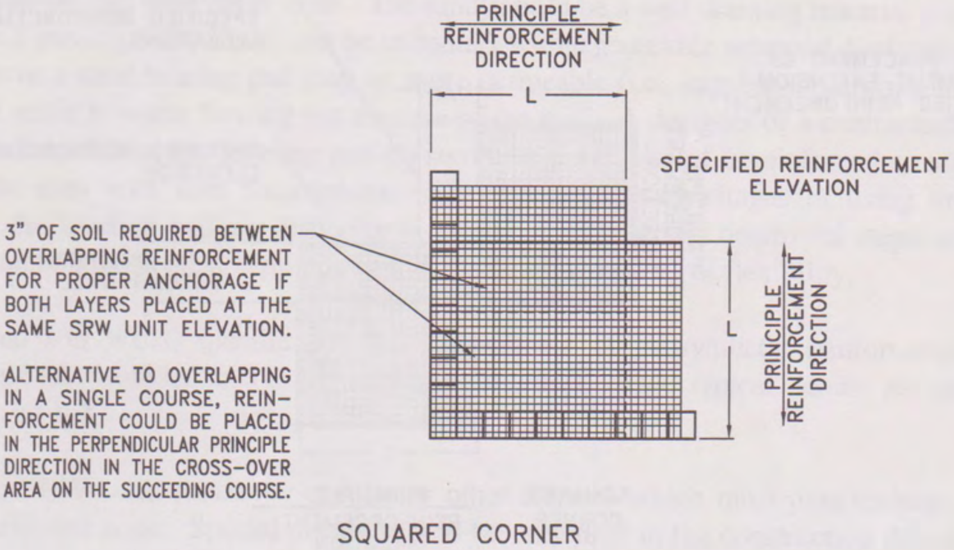


FIGURE 6–1: REINFORCEMENT PLACEMENT FOR CONCAVE CORNERS



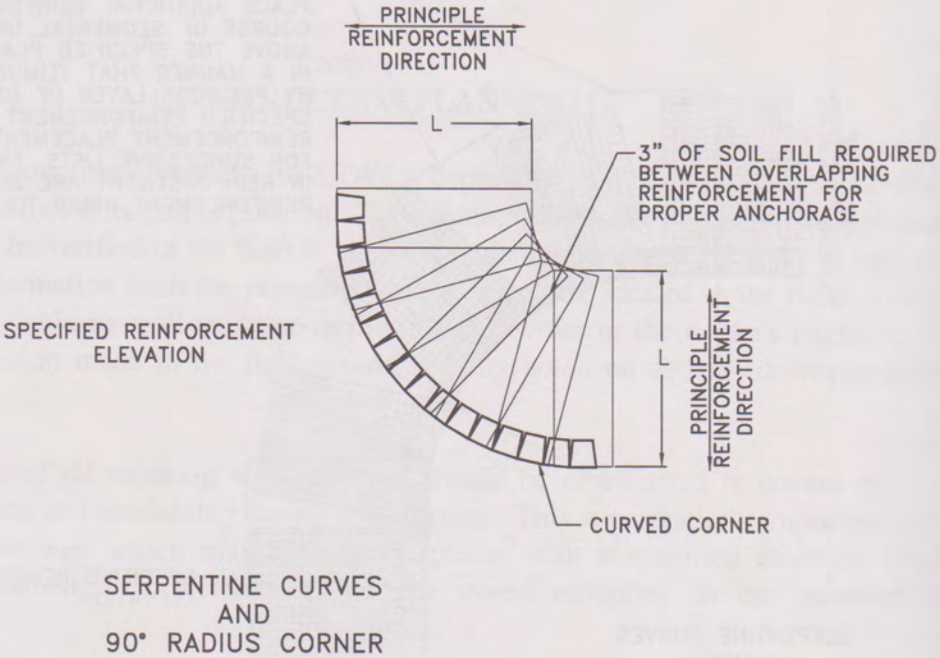
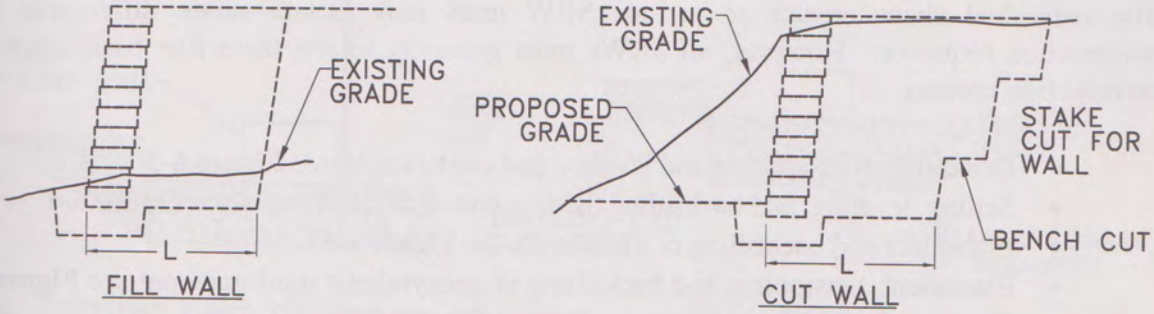
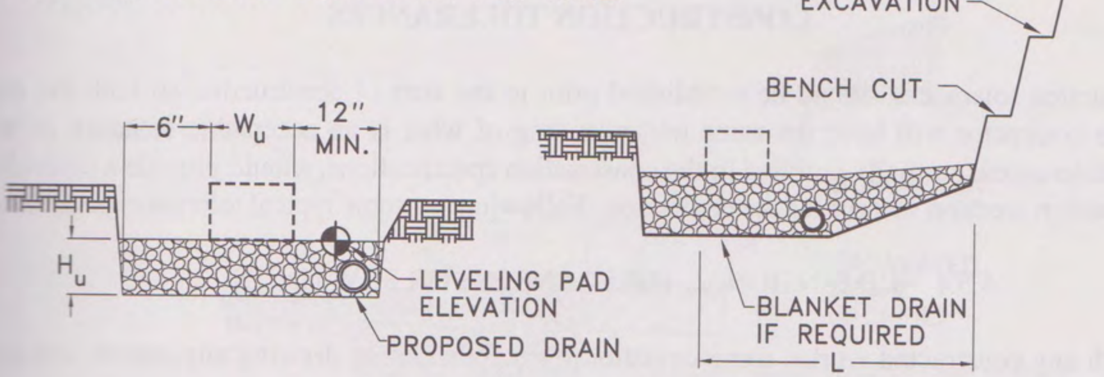


FIGURE 6–2: REINFORCEMENT PLACEMENT FOR CONVEX CORNERS



A. WALL LAYOUT AND GENERAL EXCAVATION

1. SURVEY STAKE SRW LOCATION AND GENERAL EXCAVATION LIMITS FOR WALL CONSTRUCTION.
2. ENSURE SRW IS ALONG PROPER ALIGNMENT, AND WITHIN APPROPRIATE PROPERTY BOUNDARIES, AND CONSTRUCTION EASEMENTS.
3. PERFORM GENERAL EXCAVATION FOR WALL.



B. LEVELING PAD CONSTRUCTION

1. STAKE WALL LOCATION FOR LEVELING PAD EXCAVATION.
2. EXCAVATE TRENCH TO CREATE A MINIMUM LEVELING PAD THICKNESS OF 6″ AND TO THE MINIMUM WIDTH SHOWN.
3. INSTALL DRAIN PIPE WITH POSITIVE GRAVITY FLOW TO OUTLET.
4. PLACE, LEVEL AND COMPACT LEVELING PAD MATERIAL FOR SRW UNITS.
5. PLACE AND COMPACT AGGREGATE BLANKET DRAIN, INSTALL GEOTEXTILE IF REQUIRED.

FIGURE 6–3: CONSTRUCTION SEQUENCE – STEP 1 EXCAVATION AND LEVELING PAD

Section 6.4
CONSTRUCTION SEQUENCE

The individual characteristics of various SRW units may dictate subtle differences in the construction sequence. However, all SRWs must generally follow these five basic steps in the construction process.

- General wall excavation and leveling pad construction-see **Figure 6-3**
- Setting, leveling, and backfilling the first course of SRW units-see **Figure 6-4**
- Placement and backfilling of SRW units-see **Figure 6-5**
- Placement, tensioning, and backfilling of geosynthetic reinforcement-see **Figure 6-6**
- Capping the SRW and finish grading-see **Figure 6-7**

By following these basic procedures, SRWs can be expected to perform well for the intended design life of the structure. However, to quantitatively evaluate the quality of the constructed SRWs, construction tolerances should be established.

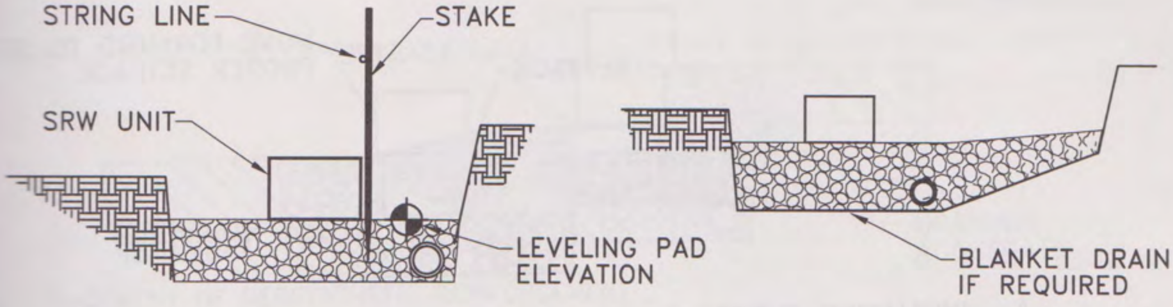
Section 6.5
CONSTRUCTION TOLERANCES

Construction tolerances should be established prior to the start of construction so both the owner and the contractor will have the same understanding of what is an acceptable standard of work. These tolerances, normally outlined in the construction specifications, should provide a controllable construction erection margin for the contractor. Following are some typical tolerance guidelines.

6.5.1 DIMENSIONAL TOLERANCES FOR SRW UNITS

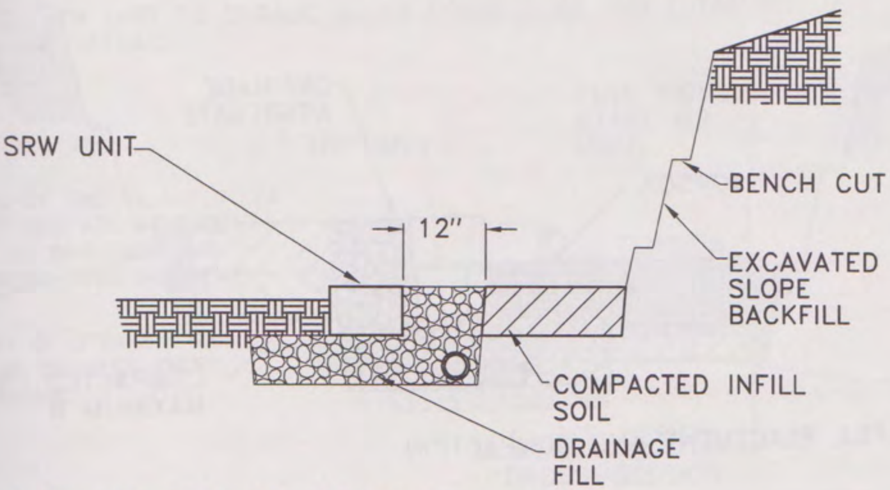
As with any constructed works, some deviation from construction drawing alignments will occur. Variability in construction of SRWs is approximately equal to that of cast-in-place concrete retaining walls. As opposed to cast-in-place concrete walls, alignment of SRWs can be simply corrected or modified during construction. Based upon examination of numerous completed SRWs, the following recommended minimum tolerances can be achieved with good construction techniques:

- Vertical control
  - ± 1.25 inches maximum over a 10 ft distance; 3 inches maximum
- Horizontal location control
  - straight lines: ± 1.25 inches over a 10 ft distance; 3 inches maximum
- Rotation
  - from established plan wall batter: 2.0°
- Bulging
  - 1.0 inch over a 10 ft distance



A. SETTING FIRST COURSE OF SRW UNITS

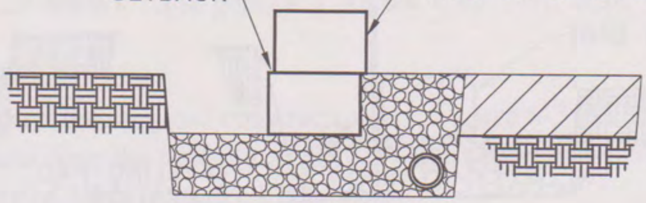
1. CHECK LEVELING PAD ELEVATION AND SMOOTH LEVELING PAD SURFACE.
2. STAKE AND STRINGLINE THE WALL LOCATION, PAY CLOSE ATTENTION TO EXACT LOCATION OF CURVES, CORNERS, VERTICAL AND HORIZONTAL STEPS. STRING LINE MUST BE ALONG A MOLDED FACE OF THE SRW UNIT, AND NOT ALONG A BROKEN BLOCK FINISH SURFACE.
3. INSTALL FIRST COURSE OF SRW UNITS, CHECKING LEVEL AS PLACED.



B. BACKFILLING FIRST COURSE OF SRW UNITS

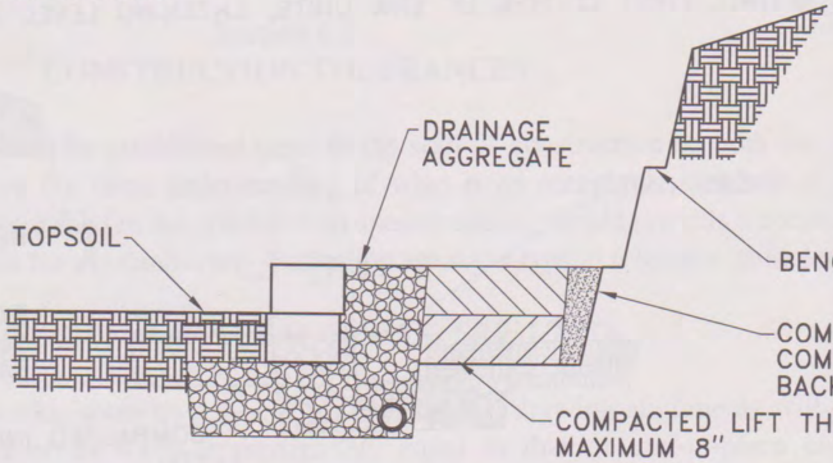
1. RECHECK WALL LOCATION.
2. USE DRAINAGE FILL TO FILL ANY OPENINGS IN AND BETWEEN SRW UNITS, AS REQUIRED.
3. CAREFULLY PLACE DRAINAGE FILL BEHIND AND UP TO THE HEIGHT OF SRW UNIT TO CREATE WALL FACE DRAIN. INSTALL GEOTEXTILE IF REQUIRED.
4. PLACE AND COMPACT INFILL SOIL BEHIND WALL DRAIN.
5. PLACE FILL SOIL IN FRONT OF SRW UNIT.
6. COMPACT DRAINAGE FILL AND INFILL SOIL.

FIGURE 6–4: CONSTRUCTION SEQUENCE – STEP 2
FIRST COURSE OF SRW UNITS



A. INSTALLING SUCCESSIVE COURSES OF SRW UNITS

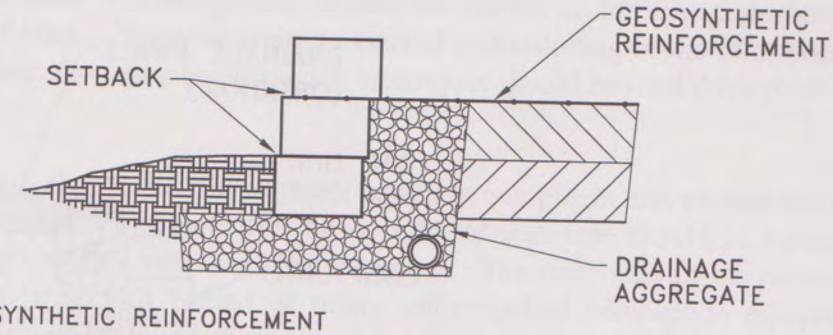
1. ENSURE THE DRAINAGE AGGREGATE IS LEVEL WITH, OR SLIGHTLY BELOW THE TOP OF SRW UNIT BELOW.
2. CLEAN DEBRIS OFF TOP OF UNIT.
3. PLACE SRW UNIT SHEAR CONNECTORS IF APPLICABLE.
4. MOVE SRW UNIT TO ENGAGE SHEAR CONNECTORS AND ESTABLISH PROPER SETBACK, CONSISTENT WITH MANUFACTURER'S RECOMMENDATIONS.



B. FILL PLACEMENT AND COMPACTION

1. USE DRAINAGE AGGREGATE TO FILL OPENINGS IN AND BETWEEN SRW UNITS AS REQUIRED.
2. PLACE DRAINAGE AGGREGATE BEHIND AND UP TO HEIGHT OF SRW UNIT TO CONTINUE WALL FACE DRAIN. INSTALL GEOTEXTILE IF REQUIRED.
3. PLACE AND COMPACT INFILL SOIL BEHIND WALL DRAIN.
4. COMPACT DRAINAGE AGGREGATE AND INFILL SOIL.

FIGURE 6–5: CONSTRUCTION SEQUENCE – STEP 3
PLACEMENT & BACKFILLING OF SRW UNITS

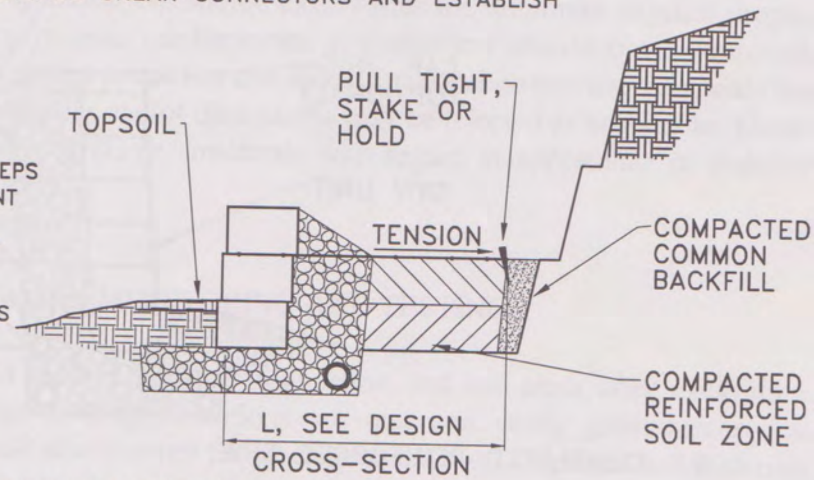


A. PLACEMENT OF GEOSYNTHETIC REINFORCEMENT

1. ENSURE WALL FACE DRAINAGE AGGREGATE IS LEVEL WITH, OR SLIGHTLY ABOVE THE TOP OF SRW UNIT.
2. CLEAN DEBRIS OFF TOP OF UNIT.
3. CUT GEOSYNTHETIC REINFORCEMENT TO DESIGN LENGTH L AS SHOWN ON PLANS AND INSTALL WITH STRENGTH DIRECTION PERPENDICULAR TO WALL FACE.
4. PLACE SHEAR CONNECTORS, IF APPLICABLE, AS RECOMMENDED BY THE MANUFACTURER.
5. PLACE SRW UNIT ON TOP OF GEOSYNTHETIC.
6. MOVE SRW UNIT TO ENGAGE SHEAR CONNECTORS AND ESTABLISH PROPER SETBACK.

NOTES:

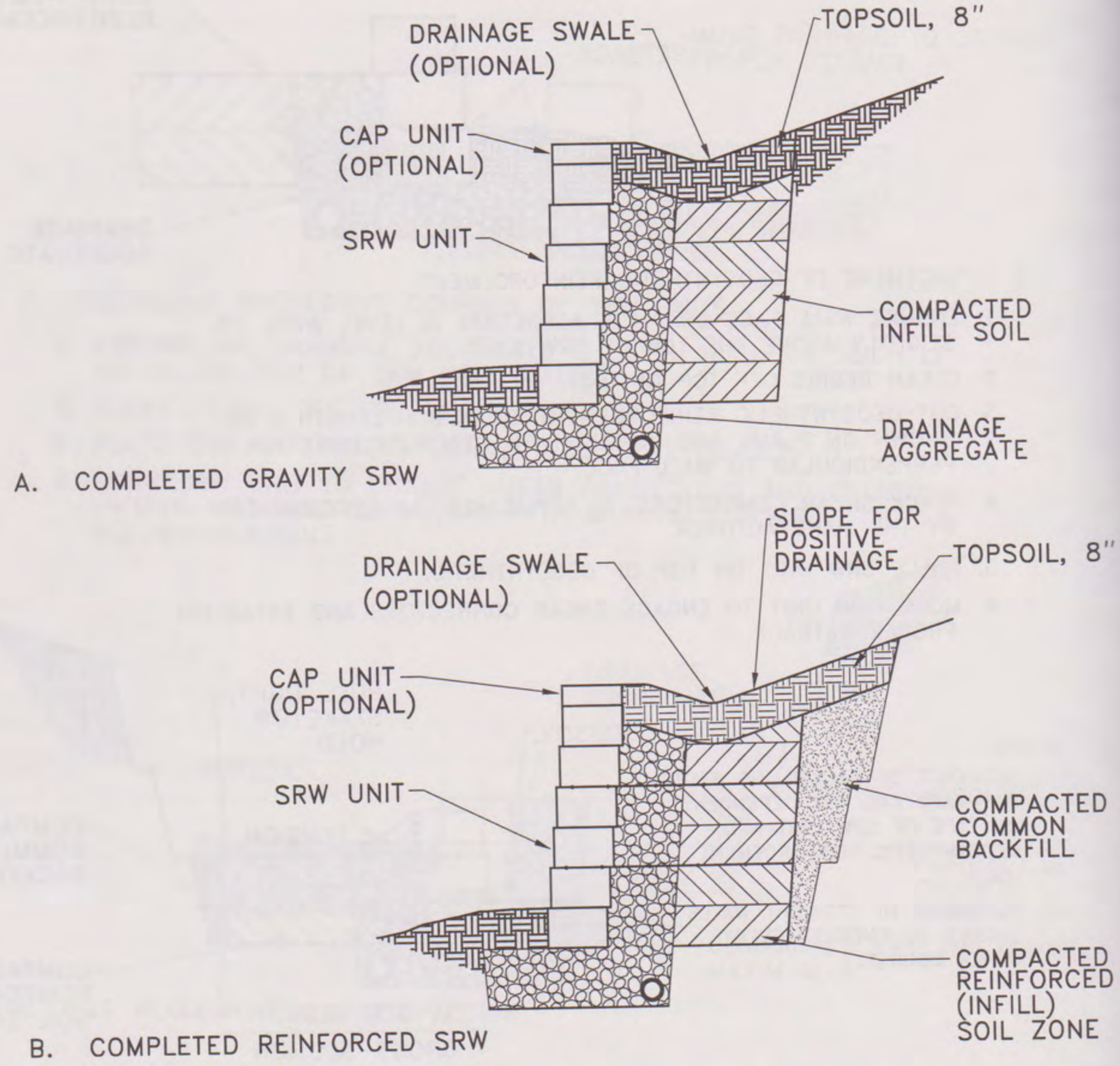
1. SEQUENCE OF BACKFILLING STEPS MAY VARY AND ARE DEPENDENT ON TYPE OF SRW UNIT AND GEOSYNTHETIC REINFORCEMENT USED.
2. ALIGNMENT OF STRAIGHT WALLS SHOULD BE CHECKED EVERY OTHER COURSE.



B. BACKFILLING OVER GEOSYNTHETIC REINFORCEMENT

1. PULL GEOSYNTHETIC REINFORCEMENT TAUT, USING UNIFORM TENSION, HOLD OR STAKE TO MAINTAIN TENSION THROUGHOUT FILL PLACEMENT PROCESS.
2. PLACE DRAINAGE AGGREGATE FOR WALL FACE DRAIN IN AND BETWEEN SRW UNITS AS REQUIRED.
3. PLACE INFILL SOIL.
4. COMPACT INFILL SOIL.
5. COMPACT DRAINAGE AGGREGATE.
6. PLACE REMAINDER OF AGGREGATE DRAIN.

FIGURE 6–6: CONSTRUCTION SEQUENCE – STEP 4
GEOSYNTHETIC REINFORCEMENT INSTALLATION



1. CONTINUE WALL TO FULL HEIGHT USING STEPS A AND B FROM FIGURES 6-5 AND 6-6.
2. INSTALL SRW CAP/COPING UNIT (OPTIONAL), SECURE PER MANUFACTURER'S RECOMMENDATIONS.
3. PLACE AND COMPACT FINAL BACKFILL.
4. FINISH GRADE FOR POSITIVE DRAINAGE AWAY FROM WALL FACE, DRAINAGE SWALE IS OPTIONAL.
5. PLACE TOPSOIL AND VEGETATE SLOPES ABOVE AND AROUND WALL TERMINATIONS.

FIGURE 6-7: CONSTRUCTION SEQUENCE – STEP 5 CAPPING AND GRADING

Horizontal and vertical control can be maintained by surveying the wall during construction. Control of wall rotation and bulging during construction can be influenced by SRW unit dimension tolerances, type of soil fill utilized, soil compaction techniques and the uniformity in geosynthetic tension applied during backfilling. Non uniformity in manual pretensioning of the reinforcement may result in localized bulging. Consistent construction techniques should be used throughout wall erection.

Careful planning and attention should be paid to the compaction equipment and procedures used during construction. Compaction within three feet of the front of wall face should be limited to hand operated equipment, preferably a vibrating plate or tamper. The remainder of the reinforced soil zone can be compacted with walk-behind or riding self-propelled compaction equipment, depending upon soil type and available operating area. Non uniform compaction procedures can result in vertical and horizontal alignment control problems. Upon completion of the wall, landscaping equipment and other vehicles should be kept at least five feet behind the wall face.

6.5.2 MATERIALS ACCEPTANCE

SRW units and geosynthetic reinforcement materials delivered to the site should be accompanied with a manufacturer's certification indicating the material meets or exceeds specified minimum physical properties. The SRW specifications should clearly state the minimum physical properties of the SRW units and the geosynthetic reinforcement manufacturer should have submitted an established correlation between design properties and index / physical properties. Materials below the required strengths, index properties and/or dimensions may be rejected as unsuitable. There are provisions in NCMA TEK 2-4 to evaluate materials with regard to appearance or inability to perform or be utilized in construction.

6.5.3 EARTHWORK MONITORING AND TESTING

A geotechnical engineer should inspect the wall foundation and cut areas after excavation is completed to assure that design bearing conditions are met and verify groundwater design assumptions. The engineer should also monitor fill placement and test compacted soil materials to ensure proper soil type and compaction specifications are being achieved. Typically, compaction requirements are 95 percent of maximum standard Proctor dry density (ASTM D 698 or AASHTO T-99) or 90 percent of maximum modified Proctor dry density (ASTM D 1557 or AASHTO T-180). The moisture content for compaction should be controlled within minus three percent to plus one percent of the optimum moisture content (see Section 3.4.4.1). Finished lift thickness should not exceed the height of unit H_u and should be limited to 12 inches. For units smaller than 6 inches, utilize some convenient multiple of unit height without exceeding the maximum.

Tolerances for the soils related construction operations of SRWs are usually controlled to meet a minimum requirement as stated above. Since the entire SRW design is predicated on the soil strength parameters, complete and thorough earthwork construction is essential to achieve satisfactory long-term performance of the SRW system. The geotechnical engineer should have final control over the suitability of soil and earthwork operations for the SRW construction.

6.5.4 GEOSYNTHETIC REINFORCEMENT

The owner's engineer should verify placement of the geosynthetic reinforcement in addition to ensuring that the material meets or exceeds specified minimum property requirements. Since geosynthetic reinforcement type, grade, or lengths may change across the extent and height of the wall, competent construction monitoring should verify that the contractor's placement of geosynthetic reinforcement is in accordance with construction plans and specifications.

SECTION 7
SAMPLE SPECIFICATION

The following sample specifications illustrate various methods to incorporate a SRW into a construction project. Traditional method and materials specifications designating material and installation requirements are presented in Section 7.1. Specifications for segmental retaining wall units, geosynthetic reinforcement and drain aggregate are presented. The method specification approach requires that a site specific design be performed by the owner's engineer. Designs should be performed such that specified SRW and geosynthetic material properties can be met by a number of manufacturers, and with properties of the on-site soil. SRW and geosynthetic properties are then specified as the minimum properties that must be met. The advantage of this type of specification is that the owner's engineer is in control of design. A potential disadvantage to the owner is that engineering costs are incurred prior to construction.

The end result specification in Section 7.2 can be used to solicit proposals from various segmental retaining wall suppliers using a "line and grade" or systems approach. Each retaining wall supplier will be required to furnish a project-specific stamped engineering design for its system that meets the requirements set forth in this manual. The efficiencies of various combinations of material components, or systems can be compared by mandating use of the standardized NCMA design approach. Manufacturers may be allowed to deviate from the NCMA approach only if these deviations are so stated in their bid proposal. This end result specification provides for an equitable assessment of different wall systems, or combinations of material components. The advantage with this type of specification is that one source, experienced in the design and construction of SRW's, will be responsible for the wall and it will be built economically due to competition.

Section 7.1
MATERIAL SPECIFICATION

A method specification for construction of segmental retaining walls follows. The SRW unit and drainage fill specifications are applicable to both conventional and reinforced soil walls. The geosynthetic reinforcement specification is only applicable to reinforced soil walls.

Shaded areas of text indicate project-specific data should be inserted at these locations by the specifier/designer and the values used in calculation of the design tables.

SPECIFICATION FOR SEGMENTAL RETAINING WALL MATERIALS

PART 1: GENERAL

1.01 Description

Work shall consist of furnishing all materials, labor, equipment, and supervision to install a segmental retaining wall system in accordance with these specifications and in reasonably close conformity with the lines, grades, design and dimensions shown on the plans or as established by the Owner or Owner's Engineer.

1.02 Related Work

A. Section ________ - Site Preparation
B. Section ________ - Earthwork

1.03 Reference Standards

A. Engineering Design
   1. NCMA Design Manual for Segmental Retaining Walls
   2. NCMA TEK 2-4 - Specifications for Segmental Retaining Wall Units
   3. NCMA SRWU-1 - Determination of Connection Strength between Geosynthetics and Segmental Concrete Units
   4 NCMA SRWU-2 - Determination of Shear Strength between Segmental Concrete Units

B. Segmental Retaining Wall Units
   1. ASTM C 140 - Sampling and Testing Concrete Masonry Units
   2. ASTM C 1262 - Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units and Related Concrete Units

C. Geosynthetic Reinforcement
   1. ASTM D 4595 Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
   2. ASTM D 5262 -Test Method for Evaluating the Unconfined Creep Behavior of Geosynthetics
   3. GRI GG-1: Single Rib Geogrid Tensile Strength
   4. GRI GG-5: Geogrid Pullout
   5. GRI GT-6: Geotextile Pullout

D. Soils
   1. ASTM D 698 - Moisture Density Relationship for Soils, Standard Method
   2. ASTM D 422 - Gradation of Soils
   3. ASTM D 424 - Atterberg Limits of Soils
   4. ASTM D G51 - Soil pH

E. Drainage Pipe
   1. ASTM D 3034 - Specification for Polyvinyl Chloride (PVC) Plastic Pipe
   2. ASTM D 1248 - Specification for Corrugated Plastic Pipe

F. Where specifications and reference documents conflict, the Owner's Engineer shall make the final determination of applicable document.

1.04 Approved Segmental Retaining Wall Systems

A. Suppliers of segmental retaining wall system material components shall have demonstrated experience in the supply of similar size and types of segmental retaining walls on previous projects, and shall be approved by the Owner's Engineer. The supplier must be approved two weeks prior to bid opening. Suppliers currently approved for this work are:

   Segmental Wall Units
   1. [redacted]
   2.
   3.

   Geosynthetic Reinforcements
   1. [redacted]
   2.
   3.

1.05 Submittals

A. Material Submittals - The Contractor shall submit manufacturer's certifications, 30 days prior to the start of work, stating that the SRW units, the geosynthetic reinforcement, and the drainage aggregate meet the requirements of section 2.0 of this specification. The Contractor shall provide a list of successful projects with references showing that the installer for the segmental retaining wall is qualified and has a record of successful performance.

1.06 Delivery, Storage, and Handling

A. The Contractor shall inspect the materials upon delivery to assure that proper type and grade material has been received.

B. The Contractor shall store and handle materials in accordance with manufacturer's recommendations.

C. The Contractor shall protect the materials from damage. Damaged material shall not be incorporated into the segmental retaining wall.

PART 2: MATERIAL

2.01 Concrete Segmental Retaining Wall Units

A. Concrete segmental units shall conform to the requirements of NCMA TEK 2-4 and have a minimum 28 days compressive strength of 3000 psi and a maximum absorption of 10 pcf as determined in accordance with ASTM C 140. For areas subject to detrimental freeze-thaw cycles as determined by the Owner or Owner's Engineer the concrete shall have adequate freeze/thaw protection and meet the requirements of ASTM C1262.

B. All units shall be sound and free of cracks or other defects that would interfere with the proper placing of the unit or significantly impair the strength or permanence of the construction. Any cracks or chips observed during construction shall fall within the guidelines outlined in NCMA TEK2-4.

C. SRW units dimensions shall not differ more than $\pm$ 1/8 inch except height, which shall not differ more than, $\pm$ 1/16 inch, as measured in accordance with ASTM C140.

D. SRW units shall match the color, surface finish and dimension for height, width, depth and batter as shown on the plans.

E. If pins are used by the retaining wall supplier to interconnect SRW units, they shall consist of a nondegrading polymer or galvanized steel and be made for the express use with the SRW units supplied.

F. Cap adhesive shall meet the requirements of the SRW unit manufacturer.

2.02 Geosynthetic Reinforcements

A. Geosynthetic Reinforcements shall consist of high tenacity geogrids or geotextiles manufactured for soil reinforcement applications. The type, strength and placement location of the reinforcing geosynthetic shall be as shown on the plans. The design properties of the reinforcement shall be determined according to the procedures outlines in this specification and the NCMA Design Manual for Segmental Retaining Walls (1996 Revision.) Detailed test data shall be submitted to the Owner's Engineer for approval at least 30 days prior to construction and shall include tensile strength (ASTM D 4595 or GRI GG-1), creep (ASTM D 5262) site damage and durability (GRI GG-4) pullout (GRI GG-5 or GRI GT-6) and connection (NCMA SRWU-1) test data.

Included with the raw test data shall be a report that will show that the proposed geosynthetic reinforcements have the following minimum properties:

Property	Geosynthetic Reinforcement Type 1	Type 2	Type 3
Allowable Reinforcement Tension - T_a (lb/ft)			
Coefficient of Interaction C_i			
Coefficient of Direct Sliding - CDs			

Calculation of the allowable reinforcement tension shall use the following method:

Allowable Reinforcement Tension:

The allowable reinforcement tension, T_a, at the end of the service life shall consider the time-temperature creep characteristics of the reinforcement, environmental degradation, construction induced damage and an overall factor of safety.

$$T_a = \frac{T_{ult}}{RF_D \bullet RF_{ID} \bullet RF_{CR} \bullet FS_{UNC}}$$

where:

T_{ult} = Ultimate (or yield tensile strength) from wide width tensile strength tests (ASTM D 4595 or GR1 GG-1 for geogrids), based on minimum average roll value (MARV) for the product.

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

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

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, and can vary typically from 1.50 to 5.0.

FS_{UNC} = Overall factor of safety or load factor to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads, and shall be no less than 1.5.

In no case shall the product RF_{ID} x RF_D x RF_{CR} be less than 2.0.

2.03 Drainage Pipe

A. The drainage collection pipe shall be a perforated or slotted, PVC or corrugated HDPE pipe. The pipe and drainage aggregate may be wrapped with a geotextile that will function as a filter.

B. Drainage pipe shall be manufactured in accordance with ASTM D 3034 and/or ASTM D 1248

2.04 Drainage Aggregate

A. Drainage aggregate shall be a clean crushed stone or granular fill meeting the following gradation as determined in accordance with ASTM D 422:

Sieve Size	Percent Passing
1 inch	100
$^3/_4$ inch	75 - 100
No. 4	0 - 60
No. 40	0 - 50
No. 200	0 - 5

2.05 Reinforced Backfill

A. The reinforced backfill shall be free of debris and consist of one of the following inorganic USCS soil types: GP, GW, SW, SP, SM, meeting the following gradation as determined in accordance with ASTM D 422.

Sieve Size	Percent Passing
4 inch	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 35

The maximum size should be limited to $^3/_4$ inch for reinforced soil SRWs unless tests have been performed to evaluate potential strength reduction in the geosynthetic due to installation damage.

The plasticity of the fine fraction of the reinforced soil shall be less than 20.

B. The pH of the backfill material shall be between 3 and 9 when tested in accordance with ASTM G-51.

2.06 Geotextile Filter

A. Drainage geotextile shall have the following minimum properties or shall meet the criteria recommended by the Wall Design Engineer.

AOS	ASTM D 4751
Grab Tensile	ASTM D 4632
Trap Tear	ASTM D 4533
Water Flow Rate	ASTM D 4491
Puncture	ASTM D 4833

PART 3: CONSTRUCTION

3.01 Inspection

A. The Owner or Owner's Engineer is responsible for verifying the materials supplied by the contractor meets all the requirements of the specification. This includes all submittals and proper installation of the system.

B. As requested by the Owner's Engineer or Contractor, the segmental retaining wall system supplier shall provide a qualified and experienced representative on site for up to 3 days to assist the Contractor regarding proper wall installation. Contractor site assistant shall be provided at no additional cost to the Owner.

C. The Contractor's field construction supervisor shall have demonstrated experience and be qualified to direct all work at the site.

3.02 Excavation

A. The Contractor shall excavate to the lines and grades shown on the project grading plans. The Contractor shall take precautions to minimize over-excavation. Excavation support, if required, shall be designed by the Contractor.

3.03 Foundation Preparation

A. Following excavation for the leveling pad and the reinforced soil zone foundation soil shall be examined by the Owner's Engineer to assure the actual foundation soil strength meets or exceeds the assumed design bearing strength. Soils not meeting the required strength shall be removed and replaced with soil meeting the design criteria, as directed by the Owner's Engineer.

3.04 Leveling Pad Preparation

A. A minimum 6 inch thick layer of compacted granular material shall be placed for use as a leveling pad up to the grades and locations as shown on the construction drawings. The granular base shall be compacted to provide a firm, level bearing pad on which to place the first course of concrete segmental retaining wall units. Compaction should be performed using a lightweight compactor, such as a mechanical plate compactor to obtain a minimum of 95% of the maximum standard Proctor density (ASTM D 698).

3.05 SRW and Geosynthetic Reinforcement Placement

A. All materials shall be installed at the proper elevation and orientation as shown in the wall details on the construction plans or as directed by the Owner's Engineer. The concrete segmental wall units and geosynthetic reinforcement shall be installed in general accordance with the manufacturer's recommendations. The drawings shall govern in any conflict between the two requirements.

B. Overlap of the geosynthetic in the design strength direction shall not be permitted. The design strength direction is that length of geosynthetic reinforcement perpendicular to the wall face and shall consist of one continuous piece of material. Adjacent sections of geosynthetic shall be placed in a manner to assure that the horizontal coverage shown on the plans is provided.

C. Geosynthetic reinforcement should be installed under tension. A nominal tension shall be applied to the reinforcement and maintained by staples, stakes, or hand tensioning until the reinforcement has been covered by at least 6 inches of soil fill.

D. The overall tolerance relative to the wall design verticality or batter shall not exceed ± 1.25 inches maximum over a 10 ft distance; 3 inch maximum.

E. Broken, chipped, stained or otherwise damaged units shall not be placed in the wall unless they are repaired and the repair method and results are approved by the Engineer.

3.06 Backfill Placement

A. The reinforced backfill shall be placed as shown in construction plans in maximum compacted lift thickness of 10 inches and shall be compacted to a minimum 95% of standard Proctor density (ASTM D 698) at a moisture content within 2% of optimum. Backfill shall be placed, spread and compacted in such a manner that eliminates the development of wrinkles or movement of the geosynthetic reinforcement and the wall facing units.

B. Only hand-operated compaction equipment shall be allowed within 3 feet of the front of the wall face. Compaction within 3 feet of the back face of the facing units shall be achieved by at least three (3) passes of a lightweight mechanical tamper, plate or roller. Soil density in this area shall not be less than 90% standard Proctor density.

C. Tracked construction equipment shall not be operated directly on the geosynthetic reinforcement. A minimum backfill thickness of 6 inches is required prior to operation of tracked vehicles over the geosynthetic reinforcement. Turning of tracked vehicles should be kept to a minimum to prevent displacing the fill and damaging or moving the geosynthetic reinforcement.

D. Rubber-tired equipment may pass over the geosynthetic reinforcement, if in accordance with the manufacturer's recommendations, at slow speeds less that 10 mph. Sudden braking and sharp turning should be avoided.

E. At the end of each day's operation, the contractor shall slope the last level of backfill away from the wall facing to direct runoff of rainwater away from the wall face. In addition, the contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

3.07 Drainage Fill Placement

A. Drainage fill shall be placed to the minimum finished thickness and widths shown on the construction plans or as modified by the Owner's Engineer.

B. Drainage collection pipes shall be installed to maintain gravity flow of water outside of the reinforced soil zone. The drainage collection pipe should daylight into a storm sewer manhole or along a slope at an elevation lower than the lowest point of the pipe within the aggregate drain.

C. The main collection drain pipe, just behind the block facing, shall be a minimum of 3 inches in diameter. The secondary collection drain pipes should be sloped a minimum of two percent to provide gravity flow into the main collection drain pipe. Drainage laterals shall be spaced at a maximum 50 feet spacing along the wall face.

3.8 Cap Block Placement

A. The cap block and/or top SRW unit shall be bonded to the SRW units below using cap adhesive described in Section 2.01F .

PART 4: MEASUREMENT AND PAYMENT

4.01 Measurement

A. The unit of measurement for furnishing the segmental retaining wall system shall be the vertical square foot of wall surface from the top of the leveling pad to the top of the wall or wall coping. The quantity to be paid shall include supply and installation of the segmental retaining wall system. Excavation of unsuitable materials and replacement with select fill, as directed and approved in writing by the Owner or Owner's Engineer shall be paid for under separate pay items.

4.02 Payment

A. The accepted quantities of segmental retaining wall system will be paid for per vertical square foot in place as measured from the top of the leveling pad to the top of wall or coping block. The quantities of the segmental retaining wall system as shown on plans or as approved by the Owner or Owner's Engineer shall be used to determine the area supplied. Payment will be made under:

Pay Item	Pay Unit
Geosynthetic Reinforced SRW	SQ. FT.

Section 7.2

END RESULT SPECIFICATION

An end result specifications for construction of segmental retaining walls follows. The goal in use of this specification is to purchase design, materials and construction from a single source. Contractors will typically bid the project. The stamped engineering design, SRW units and geosynthetic reinforcement material (if applicable) may be purchased by the Contractor as a package from a wall "system" supplier, or the Contractor may individually purchase these components.

Shaded items indicate selections which must be inserted by the designer/specifier.

SPECIFICATION FOR SEGMENTAL RETAINING WALL SYSTEMS

PART 1: GENERAL

1.01 Description

Work shall consist of designing, furnishing all materials, labor, equipment, and supervision and placement of a segmental retaining wall system in accordance with these specifications and in reasonably close conformity with the lines, grades, design and dimensions shown on the plans or as established by the Owner or Owner's Engineer.

1.02 Related Work

A. Section ________ - Site Preparation
B. Section ________ - Earthwork

1.03 Reference Standards

A. Engineering Design
1. NCMA Design Manual for Segmental Retaining Walls
2. NCMA TEK 2-4 - Specifications for Segmental Retaining Wall Units
3. NCMA SRWU-1 - Determination of Connection Strength between Geosynthetics and Segmental Concrete Units
4 NCMA SRWU-2 - Determination of Shear Strength between Segmental Concrete Units

B. Segmental Retaining Wall Units
1. ASTM C 140 - Sampling and Testing Concrete Masonry Units
2. ASTM C 1262 - Evaluating the Freeze-Thaw Durability of Manufactured Concrete Masonry Units and Related Concrete Units

C. Geosynthetic Reinforcement
1. ASTM D 4595 Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
2. ASTM D 5262 -Test Method for Evaluating the Unconfined Creep Behavior of Geosynthetics
3. GRI GG-1: Single Rib Geogrid Tensile Strength
4. GRI GG-5: Geogrid Pullout
5. GRI GT-6: Geotextile Pullout

D. Soils
1. ASTM D 698 - Moisture Density Relationship for Soils, Standard Method
2. ASTM D 422 - Gradation of Soils
3. ASTM D 424 - Atterberg Limits of Soils
4. ASTM D G51 - Soil pH

E. Drainage Pipe
   1. ASTM D 3034 - Specification for Polyvinyl Chloride (PVC) Plastic Pipe
   2. ASTM D 1248 - Specification for Corrugated Plastic Pipe

F. Where specifications and reference documents conflict, the Owner's Engineer shall make the final determination of applicable document.

1.04 Approved Segmental Retaining Wall Systems

A. Suppliers of segmental retaining wall systems shall have demonstrated experience in the construction of similar size and types of segmental retaining walls on previous projects, and shall be approved by the Owner's Engineer. The supplier must be approved two weeks prior to bid opening. Suppliers currently approved for this work are:

   1. [redacted]
   2.
   3.

1.05 Submittals

A. Material Submittals - The Contractor shall submit manufacturer's certifications, 30 days prior to the start of work, stating that the SRW units, the geosynthetic reinforcement, and the drainage aggregate meet the requirements of Section 2 of this specification. The Contractor shall provide a list of successful projects with references showing that the installer for the segmental retaining wall is qualified and has a record of successful performance.

B. Design Submittal: The Contractor shall submit 3 sets of detailed design calculations, construction drawings, and shop drawings for approval at least 30 days prior to the beginning of reinforced segmental retaining wall construction. A detailed explanation of the design properties for the geosynthetic reinforcements shall be submitted with the design. All computer generated calculations and drawings shall be prepared and sealed by a professional engineer, licensed in the State or Province where the wall is to be built.

1.06 Delivery, Storage, and Handling

A. The Contractor shall inspect the materials upon delivery to assure that proper type and grade material has been received.

B. The Contractor shall store and handle all materials in accordance with manufacturer's recommendations and in a manner to prevent deterioration or damage due to moisture, temperature changes, contaminants, corrosion, breaking, chipping or other causes.

C. The Contractor shall protect the materials from damage. Damaged material shall not be incorporated into the segmental retaining wall.

PART 2: MATERIAL

2.01 Concrete Segmental Retaining Wall Units

A. Concrete segmental units shall conform to the requirements of NCMA TEK 2-4 and have a minimum 28 days compressive strength of 3000 psi and a maximum absorption of 10 pcf as determined in accordance with ASTM C 140. For areas subject to detrimental freeze-thaw cycles as determined by the Owner or Owner's Engineer the concrete shall have adequate freeze/thaw protection and meet the requirements of ASTM C1262.

B. All units shall be sound and free of cracks or other defects that would interfere with the proper placing of the unit or significantly impair the strength or permanence of the construction. Any cracks or chips observed during construction shall fall within the guidelines outlined in NCMA TEK2-4.

C. SRW units dimensions shall not differ more than ± 1/8 inch except height, which shall not differ more than, ± 1/16 inch, as measured in accordance with ASTM C140.

D. SRW units shall match the color, surface finish and dimension for height, width, depth and batter as shown on the plans.

E. If connectors are used by the retaining wall supplier to interconnect SRW units, they shall meet the requirements of the manufacturer.

F. Cap adhesive shall meet the requirements of the SRW unit manufacturer.

2.02 Geosynthetic Reinforcements

A. Geosynthetic Reinforcements shall consist of geogrids or geotextiles manufactured for soil reinforcement applications. The type, strength and placement location of the reinforcing geosynthetic shall be determined by the Engineer providing the wall design. The design properties of the reinforcement shall be determined according to the procedures outlines in this specification and the NCMA Design Manual for Segmental Retaining Walls (1996 Revision.) Detailed test data shall be submitted to the Owner's Engineer for approval at least 30 days prior to construction and shall include tensile strength (ASTM D 4595 or GRI GG-1), creep (ASTM D 5262) site damage and durability (GRI GG-4) and pullout (GRI GG-5 or GRI GT-6) and connection (NCMA SRWU-1) test data.

2.03 Drainage Pipe

A. The drainage collection pipe shall be a perforated or slotted, PVC or corrugated HDPE pipe. The pipe may be covered with a geotextile sock that will function as a filter.

B. Drainage pipe shall be manufactured in accordance with ASTM D 3034 and/or ASTM D 1248

2.04 Drainage Aggregate

A. Drainage aggregate shall be a clean crushed stone or granular fill meeting the following gradation as determined in accordance with ASTM D 422:

Sieve Size	Percent Passing
1 inch	100
$^3/_4$ inch	75 - 100
No. 4	0 - 60
No. 40	0 - 50
No. 200	0 - 5

2.05 Reinforced Backfill

A. The reinforced backfill shall be free of debris and consist of one of the following inorganic USCS soil types: GP, GW, SW, SP, SM meeting the following gradation as determined in accordance with ASTM D 422.

Sieve Size	Percent Passing
4 inch	100 - 75
No. 4	100 - 20
No. 40	0 - 60
No. 200	0 - 35

The maximum size should be limited to $^3/_4$ inch for reinforced soil SRWs unless tests have been performed to evaluate potential strength reduction in the geosynthetic due to installation damage.

The plasticity of the fine fraction of the reinforced soil shall be less than 20.

B. The pH of the backfill material shall be between 3 and 9 when tested in accordance with ASTM G 51.

2.06 Geotextile Filter

A. Drainage geotextile shall have the following minimum properties or shall meet the criteria recommended by the Wall Design Engineer.

AOS	ASTM D 4751
Grab Tensile	ASTM D 4632
Trap Tear	ASTM D 4533
Water Flow Rate	ASTM D 4491
Puncture	ASTM D 4833

PART 3: WALL DESIGN CRITERIA

Retaining walls shall be designed in accordance with recommendations of the NCMA Design Manual for Segmental Retaining Walls (revised 1996.) The following sections summarize the minimum design criteria for this project.

The design by the wall system supplier shall consider the internal stability of the reinforced soil mass and shall be in accordance with acceptable engineering practice and these specifications. External stability including global stability, and total and differential settlement is the responsibility of the Owner or the Owner's Geotechnical Engineering Consultant. The design life of the structure shall be 75 years unless otherwise specified by the Owner.

3.01 Design Height

The structures' design height, H, shall be measured from the top of the leveling pad to the top of the wall where the ground surface intercepts the wall facing.

3.02 Soil Reinforcement Length

The minimum soil reinforcement length shall be as required to achieve a minimum width of structure, B, measured from the front face of wall to the end of the soil reinforcements, greater than or equal to 60 percent of the design height, ***H***. The length of the reinforcements at the top of the wall may be increased beyond the minimum length required to increase pullout resistance.

3.03 Inclination of Failure Surface:

A. A Coulomb failure surface passing through the base of the wall behind the facing units up to the ground surface at or above the top of wall shall be assumed in design of walls.

3.04 Soil

Design parameters: The following soil parameters shall be assumed for the design unless otherwise shown on the plans or specified by the Engineer.

Reinforced fill:	unit weight =		pcf, ϕ =		, C = 0
Random backfill:	unit weight =		pcf, ϕ =		, C = 0
Foundation soils:	unit weight =		pcf, ϕ =		, C = 0

3.05 Minimum Factors of Safety for Internal Stability:

Reinforcement yield or reinforcement rupture: FS_{UNC} = 1.5 @ end of service life.
Reinforcement pullout: ***FS*** = 1.5 against ultimate pullout (GRI GG-5 or GRI GT-6)

3.06 Allowable Reinforcement Tension:

The allowable reinforcement tension, T_a, at the end of the service life shall consider the time-temperature creep characteristics of the reinforcement, environmental degradation, construction induced damage and an overall factor of safety.

$$T_a = \frac{T_{ult}}{RF_D \bullet RF_{ID} \bullet RF_{CR} \bullet FS_{UNC}}$$

where:

T_{ult} = Ultimate (or yield tensile strength) from wide width tensile strength tests (ASTM D 4595 or GR1 GG-1 for geogrids), based on minimum average roll value (MARV) for the product.

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

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

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, and can vary typically from 1.50 to 5.0.

FS_{UNC} = Overall factor of safety or load factor to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and externally applied loads, and can typically vary from 1.35 to 1.5.

In no case shall the product of RF_{ID} x RF_D x RF_{CR} be less than 2.0

3.07 Minimum Factors of Safety for external stability:

Sliding of the mass:	$FS = 1.5$
Overturning of the mass:	$FS = 2.0$
Bearing capacity:	$FS = 2.0$
Eccentricity:	***L-2e*** Shall fall within the rear two thirds of the structure.

3.08 Connection Strength

The allowable connection strength of reinforcements to facing units, T_{ac} shall be the lesser of:

$$T_{c\ell} = T_{ultconn} / FS_c < T_a$$

$$T_{cs} = T_{conn} @ \tfrac{3}{4} < T_a$$

where:

$T_{ultconn}$ = ultimate connection strength (NCMA SRWU-1).
$T_{c\ell}$ = long-term allowable connection strength.
T_{cs} = long-term connection strength based on serviceability.
T_{conn} @ ¾ = the connection strength at ¾ inch deformation determined in accordance with NCMA Test Method SRWU-1.
FS_c = Factor of Safety against connection failure typically equal to 1.5.

3.09 State of Stress:

A. The lateral earth pressure to be resisted by the reinforcements *at each reinforcement layer shall be calculated using the Coulomb coefficient of earth* pressure, K_a, times the vertical stress at each reinforcement layer.

$$K_a = \frac{\cos^2(\phi+\omega)}{\cos^2\omega\cos(\omega-\delta)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\omega-\delta)\cos(\omega+\beta)}}\right]^2}$$

B. The vertical soil stress at each reinforcement layer shall be taken equal to the unit weight of soil times the depth to the reinforcement layer below the finished grade behind the facing units. A coefficient of active earth pressure, K_a, shall be used from top to bottom of wall. The coefficient of active earth pressure, K_a, shall be assumed independent of all external loads except sloping fills. For sloping fills, the coefficient of active earth pressure, K_a, appropriate for the sloping condition, using Coulomb earth pressure, shall be used in the analysis.

3.10 Minimum embedment: the minimum wall embedment shall be the greater of 0.5 feet or the following:

Level Slope in Front	$H'/20$
Level Slope (abutments)	$H'/10$
3H:1V Slope in Front	$H'/10$
2H:1V Slope in Front	$H'/7$

where H' is the exposed height of the wall

3.11 Settlement control: It is the responsibility of the Owner or the Owner's Geotechnical Engineering Consultants to determine if the foundation soils will require special treatment to control total and differential settlement.

PART 4: CONSTRUCTION

4.01 Inspection

A. The Owner or Owner's Engineer is responsible for verifying that the contractor meets all the requirements of the specification. This includes all submittals for materials and design, qualifications and proper installation of the system.

B. As requested by the Owner's Engineer or Contractor, the segmental retaining wall system supplier shall provide a qualified and experienced representative on site for up to 3 days to assist the Contractor regarding proper wall installation. Contractor site assistant shall be provided at no additional cost to the Owner.

C. The Contractor's field construction supervisor shall have demonstrated experience and be qualified to direct all work at the site.

4.02 Excavation

A. The Contractor shall excavate to the lines and grades shown on the project grading plans. The Contractor shall take precautions to minimize over-excavation. Excavation support, if required, shall be designed by the Contractor.

4.03 Foundation Preparation

A. Following excavation for the leveling pad and the reinforced soil zone foundation soil shall be examined by the Owner's Engineer to assure the actual foundation soil strength meets or exceeds the assumed design bearing strength. Soils not meeting the required strength shall be removed and replaced with soil meeting the design criteria, as directed by the Owner's Engineer.

4.04 Leveling Pad Preparation

A. A minimum 6 inch thick layer of compacted granular material shall be placed for use as a leveling pad up to the grades and locations as shown on the construction drawings. The granular base shall be compacted to provide a firm, level bearing pad on which to place the first course of concrete segmental retaining wall units. Compaction should be performed using a lightweight compactor, such as a mechanical plate compactor to obtain a minimum of 95% of the maximum standard Proctor density (ASTM D 698).

4.05 SRW and Geosynthetic Reinforcement Placement

A. All materials shall be installed at the proper elevation and orientation as shown in the wall details on the construction plans or as directed by the Owner's Engineer. The concrete segmental wall units and geosynthetic reinforcement shall be installed in general accordance with the manufacturer's recommendations. The drawings shall govern in any conflict between the two requirements.

B. Overlap of the geosynthetic in the design strength direction shall not be permitted. The design strength direction is that length of geosynthetic reinforcement perpendicular to the wall face and shall consist of one continuous piece of material. Adjacent sections of geosynthetic shall be placed in a manner to assure that the horizontal coverage shown on the plans is provided.

C. Geosynthetic reinforcement should be installed under tension. A nominal tension shall be applied to the reinforcement and maintained by staples, stakes, or hand tensioning until the reinforcement has been covered by at least 6 inches of soil fill.

D. The overall tolerance relative to the wall design verticality or batter shall not exceed ± 1.25 inches maximum over a 10 ft distance; 3 inches maximum.

E. Broken, chipped, stained or otherwise damaged units shall not be placed in the wall unless they are repaired and the repair method and results are approved by the Engineer.

4.06 Backfill Placement

A. The reinforced backfill shall be placed as shown in construction plans in maximum compacted lift thickness of 10 inches and shall be compacted to a minimum 95% of standard Proctor destiny (ASTM D 698) at a moisture content within 2% of optimum. Backfill shall be placed, spread and compacted in such a manner that eliminates the development of wrinkles or movement of the geosynthetic reinforcement and the wall facing units.

B. Only hand-operated compaction equipment shall be allowed within 3 feet of the front of the wall face. Compaction within 3 feet of the back face of the facing units shall be achieved by at least three (3) passes of a lightweight mechanical tamper, plate or roller. Soil density shall not be less than 90% standard Proctor density.

C. Tracked construction equipment shall not be operated directly on the geosynthetic reinforcement. A minimum backfill thickness of 6 inches is required prior to operation of tracked vehicles over the geosynthetic reinforcement. Turning of tracked vehicles should be kept to a minimum to prevent displacing the fill and damaging or moving the geosynthetic reinforcement.

D. Rubber-tired equipment may pass over the geosynthetic reinforcement, if in accordance with the manufacturer's recommendations, at slow speeds less that 10 mph. Sudden braking and sharp turning should be avoided.

E. At the end of each day's operation, the contractor shall slope the last level of backfill away from the wall facing to direct runoff of rainwater away from the wall face. In addition, the contractor shall not allow surface runoff from adjacent areas to enter the wall construction site.

4.07 Drainage Fill Placement

A. Drainage aggregate shall be placed to the minimum finished thickness and widths shown on the construction plans or as modified by the Owner's Engineer.

B. Drainage collection pipes shall be installed to maintain gravity flow of water outside of the reinforced soil zone. The drainage collection pipe should daylight into a storm sewer manhole or along a slope at an elevation lower than the lowest point of the pipe within the aggregate drain.

C. The main collection drain pipe, just behind the block facing, shall be a minimum of 3 inches in diameter. The secondary collection drain pipes should be sloped a minimum of 2% to provide gravity flow into the main collection drain pipe. Drainage laterals shall be spaced at a maximum 50 feet spacing along the wall face.

4.08 Cap Block Placement

A. The cap block and/or top SRW unit shall be bonded to the SRW units below using cap adhesive described in Section 2.01F .

PART 5: MEASUREMENT AND PAYMENT

5.01 Measurement

A. The unit of measurement for furnishing and fabricating the segmental retaining wall system shall be the vertical square foot of wall surface from the top of the leveling pad to the top of the wall or wall coping. The quantity to be paid shall include design, supply, and installation of the segmental retaining wall system. Excavation of unsuitable materials and replacement with select fill, as directed and approved in writing by the Owner or Owner's Engineer shall be paid for under separate pay items.

5.02 Payment

A. The accepted quantities of segmental retaining wall system will be paid for per vertical square foot in place as measured from the top of the leveling pad to the top of wall or coping block. The quantities of the segmental retaining wall system as shown on plans or as approved by the Owner or Owner's Engineer shall be used to determine the area supplied. Payment will be made under:

Pay Item	Pay Unit
Geosynthetic Reinforced SRW	SQ. FT.

SECTION 8
SPECIAL DESIGN CONSIDERATIONS

Occasionally, conventional and reinforced soil SRWs are planned for projects that require special considerations to ensure the structure is appropriately designed. Although these special conditions require modifications to the design/analysis of SRWs, the overall methodology provided in the previous sections is still applicable. The two most common special considerations are rise in groundwater table and seismic loading.

Section 8.1
INFLUENCE OF GROUNDWATER CONDITIONS ON WALL DESIGN

Seasonal fluctuations in the groundwater table are to be anticipated. Groundwater rise to the leveling pad elevation may generate a significant loss in available soil shear in resistance that can lead to instability. Therefore, when groundwater conditions dictate that either internal drainage Case 2 or 3 (i.e., groundwater at or above base of wall, Section 3.3.1) are required, a modification of the design soil parameters is necessary to perform a partial effective stress analysis.

For drainage Case 2, foundation and drainage soil parameters should be changed to effective stress parameters (i.e., c'_f, ϕ'_f, and γ'_f).

[Eq. 8-1]

$$\gamma' = \gamma_{sat} - 62.4\ pcf$$

For drainage Case 3, the drainage material and foundation soil parameters are expressed in terms of effective stress parameters. However, the reinforced (infill) and retained soil zones of the SRW system can still be analyzed using total stress parameters provided the destabilizing forces exerted by the retained soil include the hydrostatic pressure and seepage forces generated by the groundwater location. It is recommended the magnitude of these groundwater forces be calculated by a qualified professional geotechnical engineer familiar with site conditions.

Section 8.2
SUBMERGED SRW DESIGN

For applications requiring any portion of a SRW to be placed permanently or periodically in a body of water, several special precautions are required as outlined below.

To account for the free flow of water through the SRW system, the blanket drain shown in internal drainage Case 3 (**Figure 3-5**) should be increased in thickness. The top of the blanket drain should be raised to an elevation one foot above the maximum anticipated water level. The gradation of the drainage material shall be designed to preclude piping (washing) of the drainage material through the segmental units and maximum permissible spacing between units. This drainage material must be protected by a geotextile or natural soil filter to prevent contamination and/or piping of either the retained and foundation soils.

The influence of this submerged condition on the soil parameters for analysis is severe. Effective stress soil parameters (Section 8.1), including buoyant unit weight, should be used for all four major soil zones; foundation soil, retained soil, drainage media and infill soil (reinforced zone), if applicable. Secondly, any additional destabilizing forces due to hydrostatic pressure shall be incorporated in both external and internal stability calculations [Ref. 38]. The magnitude and location of the hydrostatic pressure and seepage forces should be determined after a qualified engineer has established both the mean low water elevation and the maximum anticipated high water elevation. It is recommended that the hydrostatic pressure seepage forces and soil parameters be determined by a qualified professional geotechnical engineer familiar with the project conditions.

The evaluation of the global stability of the submerged SRW system should be performed for the most severe combination of groundwater flow, water feature elevation and surcharge loadings. Additionally, a qualified engineer must address potential scour beneath the SRW system, and the effects of any possible wave action.

The structural durability and long-term aesthetics of the SRW unit, due to exposure to water, should be addressed by the manufacturer. This should include effects of freeze/thaw and ice action on the SRW unit.

Section 8.3.
SEISMIC DESIGN

Under a seismic loading, a reinforced SRW is subjected to dynamic forces in addition to the static forces. The allowable tensile stress of the geogrid reinforcement may be increased for short-term seismic loading conditions [Refs. 1, 16, 19, 24]. The target safety factor is typically taken as greater than or equal to 1.1 for these potential failure modes.

Design of SRWs for seismic loading should be based on dynamic analysis of the SRW and soil mass or on a pseudo-static analysis in which dynamic seismic effects are determined by an equivalent static force model. The magnitude of the pseudo-static force coefficient will typically be dictated by local codes and/or practice. A detailed map of seismic risk is presented in the AASTHO Bridge Manual [Ref. 39] or NEHRP [Ref. 58].

Use of pseudo-static dynamic earth pressures per the Mononobe-Okabe procedure may be acceptable for SRWs [Refs. 16, 19]. This pseudo-static analysis was developed for retaining walls and assumes that the reinforced soil behind the wall behaves as a rigid body. The factor of safety against failure by outward sliding should be greater than or equal to 1.1. The potential for liquefaction or excessive subsidence of the foundation soils supporting the SRW must be analyzed separately. Pseudo-static techniques might not be appropriate for areas subject to high seismic loadings or SRWs adjacent to critical structures. Comprehensive dynamic analysis procedures should be utilized for these cases.

SECTION 9
REFERENCES

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CONVENTIONAL SRW EXAMPLE CALCULATIONS

Problem Statement And Parameter Selection:

This sample calculation demonstrates the procedure outlined in the *NCMA Design Manual for Segmental Retaining Walls, 2nd Ed.*, for analysis of conventional segmental retaining walls. The analysis examines a wall's external stability as the resistance against sliding, overturning, and bearing capacity failure. The internal stability is examined for shear resistance between the segmental units. The following example considers an exposed wall height of six feet eight inches (6.67') which correlates to ten courses of an eight inch high segmental unit. The assumed embedment depth (H_{emb}) will be one unit or eight inches (8") for a total height of seven feet four inches (7.34'). A level backfill without a surcharge will be included in this non-critical application. The leveling pad is assumed to be compacted drainage aggregate that extends six inches laterally from the heel and toe of the lowermost SRW unit.

Soils:

Infill Soil - $\phi_i = 34^o$, $\gamma_i = 120$ lb/ft^3, $c_i = 0$ lb/ft^2, $\delta_i = (2\phi_i/3) = 22.7^o$
Retained Fill - $\phi_r = 34^o$, $\gamma_r = 120$ lb/ft^3, $c_r = 0$ lb/ft^2
Foundation Soil - $\phi_f = 34^o$, $\gamma_f = 120$ lb/ft^3, $c_f = 0$ lb/ft^2
Drainage Aggregate Leveling Pad - $\phi_d = 40^o$, $\gamma_d = 120$ lb/ft^3, $c_d = 0$ lb/ft^2

Segmental Concrete Unit:

$H_{cu} = 0.00'$ (no cap units), $H_u = 8" = 0.67'$
$W_u = 24" = 2.00'$, $\gamma_u = 120$ lb/ft^3, $G_u = 1.00'$
Segmental Unit Setback = 1.5" per course

$\therefore$ Wall inclination or batter, $\omega = \tan^{-1}(1.5"/8") = 10.6^o$ use 10^o
Flat Interface Unit with $a_u = 400$ lb/ft, $\lambda_u = 30^o$
Coefficient of Interaction (segmental units and drainage aggregate), $\mu_b = 0.70$

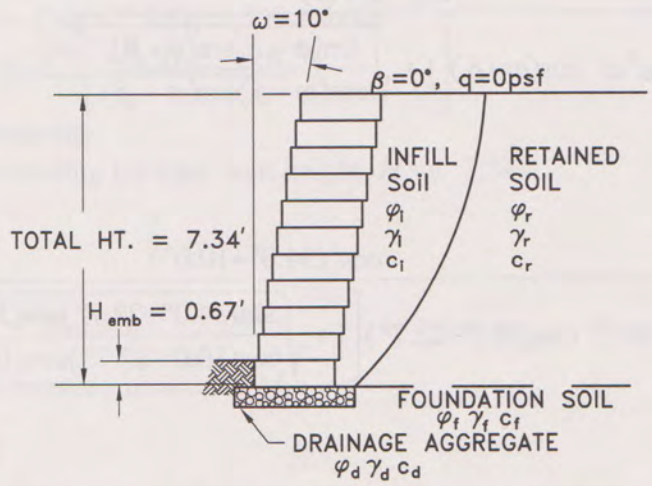


FIGURE A-1: CONVENTIONAL SRW GEOMETRY

EXTERNAL STABILITY CALCULATIONS

1. Calculate earth forces acting on the wall:

[Eq. 4- 4]

$P_a = P_s + P_q$ Note: In this example $\boldsymbol{P_q} = 0$ and is ignored

[Eq. 4- 5]

$P_s = 0.5\,(K_a)(\gamma_i)(H)^2 \cos\,(\delta_i - \mapsto)$

Confirm analysis procedure is applicable:

[Eq. 3-18]

$\omega \leq \delta_i$ For this example: $\omega = 10^{\circ}$ and $\delta_i = 22.7^{\circ}$ $\therefore$O.K.

Confirm whether the total height, $\boldsymbol{H}$, or the hinge height, $\boldsymbol{H_h}$, should be used in Eq. 4-5:

[Eq. 4- 1]

$H_h = 2(W_u - G_u) / \tan\omega$

$H_h = 2(2.00' - 1.00') / \tan 10^{\circ}$

$H_h = 11.34'$

Since $H_h > H$ use $H = 7.34'$

Calculate $\boldsymbol{K_a}$:

[Eq. 3-11]

$$K_a = \frac{\cos^2(\phi + \omega)}{\cos^2\omega\ \cos(\omega - \delta_i)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta_i)\sin(\phi - \beta)}{\cos(\omega - \delta_i)\cos(\omega + \beta)}}\right]^2}$$

$$K_a = \frac{\cos^2(34.0^{\circ}+10.0^{\circ})}{\cos^2 10.0^{\circ}\ \cos(10.0^{\circ}-22.7^{\circ})\left[1 + \sqrt{\dfrac{\sin(34.0^{\circ}+22.7^{\circ})\sin(34.0^{\circ}-0.00^{\circ})}{\cos(10.0^{\circ}-22.7^{\circ})\cos(10.0^{\circ}+0.00^{\circ})}}\right]^2}$$

$K_a = 0.190$

Therefore,

$P_a = P_s = 0.5(0.190)(120\ \text{lb/ft}^3)(7.34')^2 \cos(22.7^\circ - 10^\circ)$
$P_a = 599.2\ \text{lb/ft}$

Determine the location of the resultant force $\boldsymbol{P_a = P_s}$:

[Eq. 4- 7]

$Y_s = H/3$
$Y_s = 7.34'/3 = 2.45'$

2. Analyze Base Sliding

Determine the sliding resistance $\boldsymbol{RS_{srw}}$

[Eq. 4-10]

$R_{s(w)} = \mu_b\ [W_u (\tan \phi_d) + c\,(W_u)]$ Note: In this example **c** $= 0.00\ \text{lb/ft}^2$

[Eq. 4- 9]

$W_w = (H)(\gamma_u)(W_u)$
$W_w = (7.34')(120\ \text{lb/ft}^3)(2.00')$
$W_w = 1761.6\ \text{lb/ft}$
$\therefore\ R_{s(w)} = 0.7[1761.6\ \text{lb/ft}\ (\tan 40^\circ)]$
$R_{s(w)} = 1034.7\ \text{lb/ft}$

Calculate the Factor of Safety against sliding, $\boldsymbol{FS_{sl}}$:

[Eq. 4-11]

$FS_{sl} = R_{s(w)} / P_a$
$FS_{sl} = (1034.7\ \text{lb/ft}) / (599.2\ \text{lb/ft})$
$\underline{FS_{sl} = 1.73 > 1.50\ \therefore\ \text{O.K.}}$

3. Analyze overturning:
Check overturning for total wall height, $\boldsymbol{H}$, of 7.34':

[Eq. 4-14]

$FS_{ot} = M_r / M_o$

Determine resisting moments, $\boldsymbol{M_r}$:

[Eq. 4-15]

$M_r = W_w\,(X_w)$

Calculate the resisting moment arm, X_w :

[Eq. 4-17]

$X_w = G_u + 0.5\ [(H - H_u) \tan \omega]$
$X_w = 1.00' + 0.5\ [(7.34' - 0.67') \tan 10^o]$
$X_w = 1.59'$

From sliding analyses:

$W_w = 1761.6$ lb/ft
$\therefore\ M_r = (1761.6$ lb/ft$)\ (1.59')$
$M_r = 2800.9$ lb-ft/ft

Determine overturning moments, M_o

[Eq. 4-17]

$M_o = P_s(Y_s) + P_q(Y_q)$ Note: In this example $P_q = 0$ and is ignored

Calculate the overturning moment arm, Y_s:

[Eq. 4- 7]

$Y_s = H/3$
$Y_s = (7.34')/3$
$Y_s = 2.45'$

From sliding analyses:

$P_s = 599.2$ lb/ft
$\therefore\ M_o = (599.2$ lb/ft$)\ (2.45')$
$M_o = 1468.0$ lb-ft/ft

Calculate the Factor of Safety against overturning, FS_{ot}:

[Eq. 4-11]

$FS_{ot} = M_r / M_o$
$FS_{ot} = (2800.0$ lb-ft/ft$) / (1468.0$ lb-ft/ft$)$
$\underline{FS_{ot} = 1.91 > 1.50\ \therefore\ \text{O.K.}}$

4. Analyze bearing capacity

Check bearing capacity for a SRW with an exposed height of 6.67' and an embedment of 0.67':

[Eq. 4-19]

$$FS_{bc} = Q_{ult} / Q_a$$

Calculate the ultimate bearing capacity:

[Eq. 4-20]

$$Q_{ult} = c_f N_c + 0.5\gamma_f (B'_f)(N_\gamma) + \gamma_f (H_{emb})(N_q)$$ Note: $c_f = 0$ lb/ft^2

From Figure 4-5 the bearing capacity factors for $\phi_f = 34^o$ are:

$N_\gamma = 41.06$ and $N_q = 29.44$

Compute the equivalent footing width, B'_f:

[Eq. 4-21]

$$B'_f = B_f - 2e$$

[Eq. 4-18]

$$B_f = W_u + 0.50'$$
$$B_f = 2.00' + 0.50'$$
$$B_f = 2.50'$$

Determine the eccentricity, e, for an eccentric footing load:

[Eq. 4-22]

$$e = \frac{P_s Y_s + P_q Y_q - W_w e_w}{W_u}$$ Note: In this example $P_q = 0$ and is ignored

[Eq. 4-23]

$$e_w = X_w - 0.5(W_u)$$
$$e_w = 1.59' - 0.5(2.00')$$
$$e_w = 0.59'$$

$$e = \frac{(599.2 \text{ lb/ft})(2.45') - (1761.6 \text{ lb/ft})(0.59')}{1761.6 \text{ lb/ft}}$$
$$e = 0.243'$$

$$\therefore \quad B'_f = 2.50' - 2(0.243')$$
$$B'_f = 2.01'$$

and

$$Q_{ult} = 0.5(120 \text{ lb/ft}^3)(2.01')(41.06) + (120 \text{ lb/ft}^3)(0.67')(29.44)$$
$$Q_{ult} = 7319 \text{ lb/ft}^2$$

Calculate the applied bearing stress, Q_a:

[Eq. 4-24]

$$Q_a = W_w / B'_f$$
$$Q_a = (1761.6 \text{ lb/ft})/(2.01')$$
$$Q_a = 876.4 \text{ lb/ft}$$

Calculate the Factor of Safety against bearing capacity failure, FS_{bs} :

[Eq. 4-19]

$$FS_{bc} = Q_{ult} / Q_a$$
$$FS_{bc} = (7319 \text{ lb/ft}^2) / (876.4 \text{ lb/ft}^2)$$
$$\underline{FS_{bc} = 8.35 > 2.0 \quad \therefore \text{ O.K.}}$$

INTERNAL STABILITY CALCULATIONS

1. Analyze shear resistance

Calculate the shear capacity of the segmental concrete units using the "generic" shear capacity envelope for flat interface units, i.e. $a_u = 400$ lb/ft and $\lambda_u = 30^\circ$.

[Eq. 4-25]

$$V_u = a_u + W_w \tan \lambda_u$$
$$V_u = 400 \text{ lb/ft} + W_w \tan 30^\circ$$

Begin from the bottom of the wall since the deepest shear interface below the top of the wall should be the most critical, for this problem as it will have the largest active earth force, P_a :

[Eq. 4- 9]

$$W_u = H_h\,(\gamma_d)(W_u)$$

Confirm whether the hinge height,Hh, or an intermediate sliding height at the top of the bottom segmental concrete unit, H_s, is appropriate:

since $H_s = H - H_u = 7.34' - 0.67' = 6.67'$
and $H_h = 11.34'$ from previous calculations
use $H_s = 6.67'$ in Eq. 4-9

$$\therefore \quad W_u = (6.67')(120 \text{ lb/ft}^3)(2.00')$$
$$W_u = 1600.1 \text{ lb/ft}$$

and

$V_u = 400 \text{ lb/ft} + (1600.1 \text{ lb/ft}) \tan 30^o$

$V_u = 1323.8 \text{ lb/ft}$

Determine the active earth force, $\boldsymbol{P_a}$, for $\boldsymbol{H_s} = 6.67'$:

[Eq. 4-26]

$P_a = 0.5(K_a)(\gamma_i)(H_s)^2\cos(\delta_i - \mapsto)$ Note: In this example $\boldsymbol{P_q} = 0$ and is ignored

$P_a = 0.5(0.190)(120 \text{ lb/ft}^3)(6.67')^2\cos(22.7^o - 10^o)$

$P_a = 494.8 \text{ lb/ft}$

Calculate the Factor of Safety against shear capacity failure:

[Eq. 4-27]

$FS_{sc} = V_u / P_a$

$FS_{sc} = (1323.8 \text{ lb/ft}) / (494.8 \text{ lb/ft})$

$FS_{sc} = 2.68 > 1.5 \quad \therefore$ O.K.

$\boldsymbol{P_a}$ decreases proportionately with $\boldsymbol{H_s}^2$ and $\boldsymbol{V_u}$ decreases proportionately with H for this example problem. Since the lowest segmental concrete unit interface has an adequate $\boldsymbol{FS_{sc}}$ then all other shear interfaces will adequately resist the associated active earth forces.

Summary of Stability Calculations:

External:		Internal:
FS_{sl}	= 1.73	$FS_{sc} = 2.68$
FS_{ot}	= 1.91	
FS_{bc}	= 8.35	

CONVENTIONAL SRW
GLOBAL STABILITY
EXAMPLE CALCULATIONS

The following example examines the global stability of an conventional eight foot (8') wall. The method of slices used to calculate the factor of safety for global stability of the conventional SRW assumes the forces acting on the sides of any slice have a zero resultant in the vertical direction. The analysis below is for a slip circle with a radius of eleven feet (11').

Wall definition:

- Total Wall Height = 8.0'
- Wall Batter = 10^o
- Segmental Unit Width, $W_u = 2.0'$
- Angle of Internal Friction (All Soils), $\phi = 34^o$
- Unit weight of soil and segmental unit, $\gamma_{soil} = \gamma_{unit} = 120$ lb/ft^3
- Cohesion (All Soils), $c = 0$ lb/ft^2
- Pore Pressure = 0 lb/ft^2
- Δx_i = width of slice and Θ_i =angle at bottom of slice

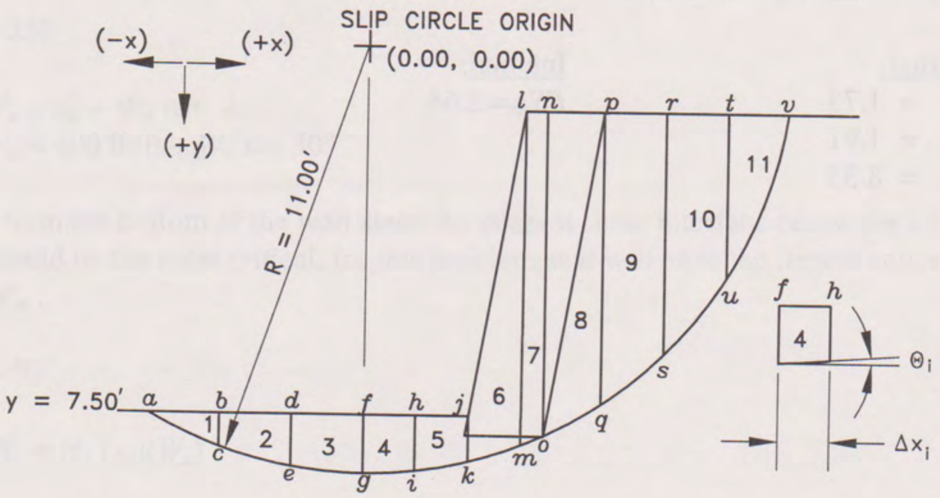


FIGURE A−2: GLOBAL STABILITY ANALYSIS USING CIRCULAR SLIP PLANE AND METHOD OF SLICES

Determine the coordinates of points ***a*** through ***t***:

Given:
$$f = (0.00',\ 9.50')$$
$$g = (0.00',\ 11.00')$$
$$l = (4.00',\ 2.00')$$
$$p = (6.00',\ 2.00')$$

and the general equation for a right triangle: $(x)^2 + (y)^2 = (R)^2$

Determine points where the circle intersects the grade:

Point a $\quad y = 7.50'$
$$(x)^2 + (9.50')^2 = (11.00')^2$$
$$(x)^2 = 30.75\ ft^2$$
$$x = 5.54'$$
$$\therefore\ a = (-5.54',\ 9.50')$$

Point v $\quad y = 2.00'$
$$(x)^2 + (2.00')^2 = (11.00')^2$$
$$(x)^2 = 117.00\ ft^2$$
$$x = 10.82'$$
$$\therefore\ v = (10.82',\ 2.00')$$

For points between ***a*** and ***f*** divide horizontal distance into three sections:

$$a = (-5.54',\ 9.50') \text{ and } f = (0.00',\ 9.50') \Rightarrow [0.00' - (-5.54')]/3 = 5.54'/3 = 1.85'$$
$$\therefore\ b = (-3.70',\ 9.50') \text{ and}$$
$$d = (-1.85',\ 9.50')$$

For points between ***p*** and ***v*** divide horizontal distance into three sections:

$$p = (6.00',\ 2.00') \text{ and } v = (10.82',\ 2.00') \Rightarrow [10.82' - (6.00')]/3 = 4.82'/3 = 1.61'$$

$$\therefore\ r = (7.61',\ 2.00') \text{ and}$$
$$t = (9.22',\ 2.00')$$

For points associated with wall geometry find coordinates ***j*** and ***k***:

Determine the x coordinate of the toe of the exposed wall from known point ***l*** = (4.00', 2.00') :

$$\Delta x = H \tan\ \omega = 7.50' \tan 10^{o} = 1.32'$$
$$\text{x coordinate of toe} = l_x - \Delta x = 4.00' - 1.32' = 2.68'$$

$$\therefore\ j = (2.68',\ 9.50')$$

Find the y coordinate of point ***k*** using $j_x = 2.68'$

$$x = 2.68' \quad (2.68')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 113.82\ ft^2$$
$$y = 10.67'$$

$\therefore\ k = (2.68', 10.67')$

For points *h* divide the horizontal distance between *f* and *j* in half:

$f = (0.00', 9.50')$ and $j = (2.68', 9.50') \Rightarrow [2.68' - (0.00')]/2 = 2.68'/2 = 1.34'$

$\therefore\ h = (1.34', 9.50')$

Find the y coordinate of point ***i*** using $h_x = 1.34'$

$$x = 1.34' \quad (1.34')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 119.20\ ft^2$$
$$y = 10.92'$$

$\therefore\ i = (1.34', 10.92')$

Find the coordinates for points ***n*** and ***o*** as follows:

Determine the x coordinate of the back of the wall base unit (point *o*) from point $p = (6.00', 2.00')$

$$\Delta x = H \tan \omega = 8.00' \tan 10^{o} = 1.41'$$
$$o_x = p_x - \Delta x = 6.00' - 1.41' = 4.59'$$
$$o_y = l_y + H = 2.00' + 8.00' = 10.00'$$

$\therefore\ o = (4.59', 10.00')$

Point ***n*** shares the same x coordinate as point ***o***, and the same y coordinate as point ***p***:

$\therefore\ n = (4.59', 2.00')$

Determine the remaining points along the circular slip surface:

Point ***c*** has the same x coordinate as point ***b*** = (-3.70', 9.50'):

$$x = -3.70' \quad (-3.70')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 107.31\ ft^2$$
$$y = 10.36'$$

$\therefore\ c = (-3.70', 10.36')$

Point ***e*** has the same x coordinate as point ***d*** = (-1.85', 9.50'):

$$x = -1.85' \qquad (-1.85')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 117.58\ ft^2$$
$$y = 10.84'$$

$\therefore\ e = (-1.85', 10.84')$

Point ***m*** has the same x coordinate as point l = (4.00', 2.00'):

$$x = 4.00' \qquad (4.00')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 105.00\ ft^2$$
$$y = 10.24'$$

$\therefore\ m = (4.00', 10.24')$

Point ***q*** has the same x coordinate as point ***p*** = (6.00', 2.00'):

$$x = 6.00' \qquad (6.00')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 85.00\ ft^2$$
$$y = 9.22'$$

$\therefore\ q = (6.00', 9.22')$

Point ***s*** has the same x coordinate as point ***r*** = (7.61', 2.00'):

$$x = 7.61' \qquad (7.61')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 63.09\ ft^2$$
$$y = 7.94'$$

$\therefore\ s = (7.61', 7.94')$

Point ***u*** has the same x coordinate as point t = (9.22', 2.00'):

$$x = 9.22' \qquad (9.22')^2 + (y)^2 = (11.00')^2$$
$$(y)^2 = 36.00\ ft^2$$
$$y = 6.00'$$

$\therefore\ u = (9.22', 6.00')$

Summary of Coordinates for Critical Circle R = 11.00'

POINT	x FT	y FT	POINT	x FT	y FT
a	-5.54	9.50	*l*	4.00	2.00
b	-3.70	9.50	*m*	4.00	10.24
c	-3.70	10.36	*n*	4.59	2.00
d	-1.85	9.50	*o*	4.59	10.00
e	-1.85	10.84	*p*	6.00	2.00
f	0.00	9.50	*q*	6.00	9.22
g	0.00	11.00	*r*	7.61	2.00
h	1.34	9.50	*s*	7.61	7.94
i	1.34	10.92	*t*	9.22	2.00
j	2.68	9.50	*u*	9.22	6.00
k	2.68	10.67	*v*	10.82	2.00

Calculation of Driving and Resisting Forces

(1) SLICE	(2) Δx_i WIDTH FT	(3) Θ_i ANGLE DEG	(4) AVERAGE HEIGHT FT	(5) W_i WEIGHT KIPS	(6) DRIVING FORCE, W_i SIN Θ_i KIPS	(7)* $M_i(\Theta_i)$ F=1.44	(8) REISISTING FORCE, (5)TAN ϕ / (7) KIPS
1	1.84	-25.1	0.43	0.10	-0.04	0.707	0.10
2	1.85	-14.5	1.10	0.24	-0.06	0.851	0.19
3	1.85	-4.9	1.42	0.32	-0.03	0.956	0.23
4	1.34	3.4	1.46	0.23	0.01	1.026	0.15
5	1.34	10.6	1.30	0.21	0.04	1.069	0.13
6	1.32	18.0	4.70	0.74	0.23	1.096	0.46
7	0.59	22.1	8.12	0.57	0.21	1.103	0.35
8	1.41	29.0	7.61	1.29	0.62	1.102	0.79
9	1.61	38.5	6.58	1.27	0.79	1.074	0.80
10	1.61	50.3	4.97	0.96	0.74	0.999	0.65
11	1.60	68.2	2.00	0.38	0.35	0.806	0.32
				$\Sigma(6)$ =	2.86	$\Sigma(8)$ =	4.17

*To determine $M_i(\Theta_i)$ use the following equation (Ref. 17):

$$M_i = \cos\Theta_i\left(1 + \frac{\tan\Theta_i \tan\phi}{F}\right) \qquad FS_{gl} = \frac{\Sigma\,(8)}{\Sigma\,(6)} = \frac{4.17}{2.86} = 1.46$$

This factor of safety is within 1.5% of the value found using a commercial slope stability program to model this wall geometry and failure circle.

```
[illegible] PROGRAM - VERSION 2.1 (AUG. 9, 1989):
SLOPE STABILITY BY THE MODIFIED BISHOP METHOD.
TENSAR GEOGRID REINFORCEMENT CAN BE INCLUDED.
PROGRAM BY THE TENSAR CORPORATION, APRIL 1986.

DATE 10-24-1996        TIME 15:15

Global Stability    $NCMA Manual, 2nd Ed$8'Wall, Conv. SRW$10/24/96

*** UNITS - FEET AND KIPS

NO SEISMIC FORCES

SOIL INFORMATION:
NO.    GAMMA   CBAR  TAN(PHIBAR)  RU      MU
1    .120  .000  .675   PHREATIC1.00
2    .120  .250  .675   PHREATIC1.00

LINE ARRAY:
NO.   X-LEFT      Y-LEFT      X-RIGHT     Y-RIGHT     SLOPE    SOIL  TOPLINE
1     100.00      100.00      150.00      100.00      .0000     1      *
2     150.00      100.00       51.32      107.50     5.6818     2      *
3     151.32      107.50      153.32      107.50      .0000     2      *
4     153.32      107.50      200.00      107.50      .0000     1      *
5     149.90       99.50      150.00      107.50     5.6337     1
7     149.90       99.50      151.90       99.50      .0000     1
8     100.00       85.00      200.00       85.00      .0000     1

PHREATIC SURFACE COORDINATES:
 NO.     X-COORD.     Y-COORD.
  1       100.00        85.00
  2       200.00        85.00

UNIT WEIGHT OF FLUID=   .0624 KIP/FT3

NO STONE COLUMNS.

NO GEOGRIDS.

APPROXIMATELY   7. SLICES WILL BE USED AT RMAX.
THE MINIMUM TANGENT ELEVATION FOR ANY FAILURE CIRCLE IS   98.50
THE MAXIMUM TANGENT ELEVATION FOR ANY FAILURE CIRCLE IS   98.50

THERE ARE   2 INCREMENTS BETWEEN TANGENT LEVELS

FACTORS OF SAFETY WILL BE COMPUTED OVER PRESCRIBED GRID:
POINT    X-COORD.     Y-COORD.
  1       147.32       109.50
  2       147.32       109.50
  3       147.32       109.50

0 INCREMENTS FROM POINT 1 TO POINT 2.
0 INCREMENTS FROM POINT 2 TO POINT 3.

X =    147.32, Y =    109.50:
  RADIUS      FACTOR OF SAFETY   DRIVING MOM   NO SLICES
                                 KIP-FT/FT
  ------      ----------------   -----------   ---------
   11.00           1.441            32.688          11

SUMMARY OF RESULTS FOR CRITICAL CIRCLE:
  X =  147.32.
  Y =  109.50.
  RADIUS OF CRITICAL CIRCLE =  11.00.
  MINIMUM FACTOR OF SAFETY =  1.441.
CALCULATIONS HAVE BEEN COMPLETED
```

REINFORCED SOIL SRW
EXAMPLE CALCULATIONS

PROBLEM STATEMENT AND PARAMETER SELECTION:

This sample calculation demonstrates the procedure outlined in the *NCMA Design Manual for Segmental Retaining Walls, 2nd Ed.*, for analysis of reinforced segmental retaining walls. The analysis examines a wall's external stability as the resistance against sliding, overturning, and bearing capacity failure. The internal stability is examined for reinforcement stresses, shear resistance between the segmental units, facing connection strength and bulging. The following example considers an exposed wall height of eight feet (8.00') with an infinite backslope of 3H:1V. No dead or live load surcharges will be included in this non-critical application. The leveling pad is assumed to be compacted drainage aggregate that extends six inches laterally from the heel and toe of the lowermost SRW unit. The final grade in front of the wall will be level.

Soils: Infill Soil - $\phi_i = 28^o$, $\gamma_i = 120$ lb/ft^3, $c_i = 0$ lb/ft^2, $\delta_i = (2\phi_i/3) = 18.7^o$
Retained Soil - $\phi_r = 28^o$, $\gamma_r = 120$ lb/ft^3, $c_r = 0$ lb/ft^2, $\delta_e = \phi_e = 28.0^o$
Foundation Soil - $\phi_f = 28^o$, $\gamma_f = 120$ lb/ft^3, $c_f = 0$ lb/ft^2
Drainage Aggregate Leveling Pad - $\phi_d = 40^o$, $\gamma_d = 120$ lb/ft^3, $c_d = 0$ lb/ft^2

Segmental Concrete Unit:
$H_{cu} = 0.00'$ (no cap units), $H_u = 6" = 0.50'$
$W_u = 12" = 1.00'$, $\gamma_u = 120$ lb/ft^3, $G_u = 0.50'$
Segmental Unit Setback = 5/16" per course
$\therefore\ \omega = \tan^{-1}[(5/16")/6"] = 3^o$
Flat Interface Unit with shear capacity envelope $a_u = 400$ lb/ft, $\lambda_u = 30^o$ and connection strength envelope $a_{cs} = 200$ lb/ft, $\lambda_{cs} = 40^o$
Coefficient of Interaction (segmental units and drainage aggregate), $\mu_b = 0.70$

Geosynthetic Reinforcement:
Ultimate strength, T_{ult}, = 2625 lb/ft
From laboratory or field testing using silty sands ($\phi = 28^o$) the following parameters have been determined -
Durability Reduction Factor, $RF_D = 1.20$
Installation Damage Reduction Factor, $RF_{ID} = 1.25$
Creep Reduction Factor, $RF_{CR} = 1.66$
Factor of Safety against Uncertainties, $FS_{unc} = 1.5$
Coefficient of interaction (soil and geosynthetic reinforcement), $C_i = 0.7$
Coefficient of direct sliding (soil and geosynthetic reinforcement), $C_{ds} = 0.95$

Total Wall Height, H, Calculation:
Exposed height, $H' = 8.00'$
Minimum wall embedment depth, $H_{emb} = H'/20 = 0.4'$
Per Section 5 minimum allowable $H_{emb} = 0.50'$
Therefore, the total wall height, $H = 8.00' + 0.50' = 8.50'$

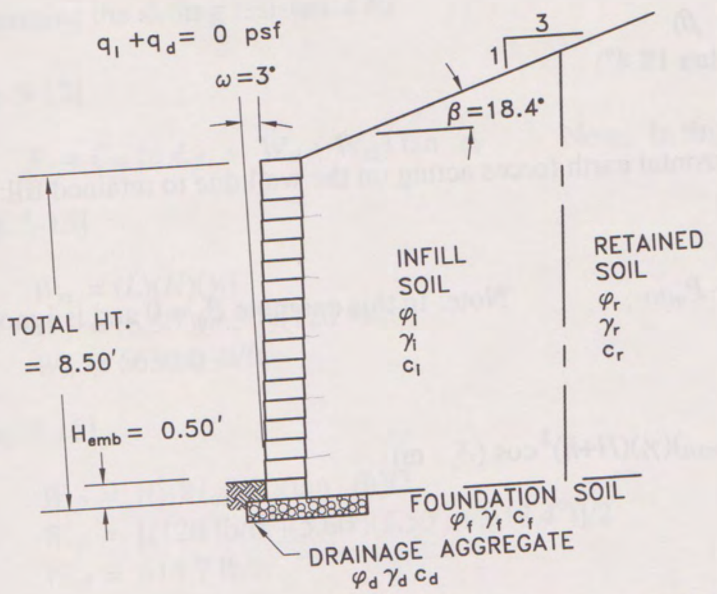


FIGURE B–1: REINFORCED SOIL SRW GEOMETRY

EXTERNAL STABILITY CALCULATIONS

1. Select trial reinforcement length, L, determine height of backslope

   Try $L = 6.50'$ (approximately ¾ H)

   [Eq. 5- 1]

   $$L' = L - W_u$$
   $$L' = 6.50' - 1.00' = 5.50'$$

   [Eq. 5- 2]

   $$L'' = \frac{L' \tan\beta \tan\omega}{1 - \tan\beta \tan\omega}$$

   $$L'' = \frac{(5.50')(\tan 18.4°)(\tan 3°)}{1 - (\tan 18.4°)(\tan 3°)}$$

   $$L'' = 0.0976'$$

   [Eq. 5- 3]

   $$L_\beta = L' + L'' = 5.50' + 0.0976'$$
   $$L_\beta = 5.5976'$$

[Eq. 5- 4]

$h = (L_\beta)\ (\tan\ \beta)$
$h = 5.5976'\ (\tan 18.4^o)$
$h = 1.8621'$

2. Calculate active horizontal earth forces acting on the wall due to retained fill:

[Eq. 5-11]

$P_{a(H)} = P_{s(H)} + P_{q(H)}$ Note: In this example $\boldsymbol{P_q} = 0$ and is ignored

[Eq. 5- 6]

$P_{s(H)} = 0.5(K_{a(ext)})(\gamma_r)(H{+}h)^2 \cos(\delta_e - \omega)$

Calculate $\boldsymbol{K_{a(ext)}}$:

[Eq. 3-16]

$\delta_e = \ \phi_r = 28^o$

[Eq. 3-11]

$$K_{a(ext)} = \frac{\cos^2(\phi_r + \omega)}{\cos^2\omega\ \cos(\omega - \delta_e)\left[1 + \sqrt{\frac{\sin(\phi_r + \delta_e)\sin(\phi_r - \beta)}{\cos(\omega - \delta_e)\cos(\omega + \beta)}}\right]^2}$$

$$K_{a(ext)} = \frac{\cos^2(28^o + 3^o)}{\cos^2(3^o)\cos(3^o - 28^o)\left[1 + \sqrt{\frac{\sin(28^o + 28^o)\sin(28^o - 18.4^o)}{\cos(3^o - 28^o)\cos(3^o + 18.4^o)}}\right]^2}$$

$K_{a(ext)} \ = 0.412$

Therefore,

$P_{a(H)} = P_{s(H)} = 0.5(0.412)(120\ \text{lb/ft}^3)(8.50'{+}1.862')^2 \cos(28.0^o - 3^o)$
$P_{a(H)} = 2405.5$ lb/ft

Determine the location of the resultant force $\boldsymbol{P_{a(H)} = P_{s(H)}}$:

[Eq. 5- 9]

$Y_{s(H)} = (\text{H+h})/3$
$Y_s = (8.50'{+}1.862')/3 = 3.45'$

3. Analyze Base Sliding

Determine the sliding resistance $\boldsymbol{R_s}$

[Eq. 5-12]

$R_s = C_{ds}\,(q_d L_\beta \;+\; W_{ri} + W_{r\beta})\tan\ \phi_i$ Note: In this example $q_d = 0.00$ lb/ft^2

[Eq. 5-15]

$W_{ri} = (L)(H)(\gamma_i)$
$W_{ri} = (6.50')(8.50')(120\text{ lb/ft}^3)$
$W_{ri} = 6630.0$ lb/ft

[Eq. 5-16]

$W_{r\beta} = [(\gamma_i)(L_\beta)(L')(\tan\ \beta)]/2$
$W_{r\beta} = [(120\text{ lb/ft}^3)(5.60')(5.50')(\tan 18.4^{o})]/2$
$W_{r\beta} = 614.7$ lb/ft

Per Section 5.5.2 where no reinforcement is placed at the base of the wall use C_{ds} = 1.00

$\therefore\quad R_s = 1.0(0\text{ lb/lf} + 6630.0\text{ lb/ft} + 614.7\text{ lb/ft})\tan 28^{o}$

$R_s = 3852.1$ lb/ft

Calculate the Factor of Safety against sliding, $\boldsymbol{FS_{sl}}$:

[Eq. 5-17]

$FS_{sl} = R_s / P_{a(H)}$
$FS_{sl} = (3852.1\text{ lb/ft}) / (2405.5\text{ lb/ft})$
$\underline{FS_{sl} = 1.60 > 1.50\ \therefore\ \text{O.K.}}$

4. Analyze overturning:

Check overturning for total wall height, $\boldsymbol{H}$, of 8.50':
Determine resisting moments, $\boldsymbol{M_r}$:

[Eq. 5-18]

$M_r = W_{ri}\,(X_{ri}) + (W_{r\beta})(X_{r\beta}) + (q_d)(L_\beta)(X_{q\beta})$
Note: In this example $\boldsymbol{q_d}$ = 0.00 lb/ft^2

Calculate the resisting moment arm, $\boldsymbol{X_{ri}}$:

[Eq. 5-19]

$X_{ri} = [L + (H\tan\omega)]/2$
$X_{ri} = [6.50' + (8.50'\tan 3^{o})]/2$
$X_w = 3.47'$

[Eq. 5-20]

$X_{r\beta} = [H \tan \omega + W_u + (2L'/3)]$
$X_{r\beta} = [0.45' + 1.00' + 3.67']$
$X_{r\beta} = 5.12'$

From sliding analyses: W_{ri} = 6630.0 lb/ft and $W_{r\beta}$ = 614.7 lb/ft

$\therefore \quad M_r = (6630.0 \text{ lb/ft})\,(3.47') + (614.7 \text{ lb/ft})(5.11')$
$M_r = 26170.3$ lb-ft/ft

Determine driving moments, M_o

[Eq. 5-12]

$M_o = P_{s(H)}(Y_s) + P_q(Y_q)$ Note: In this example $P_q = 0$ and is ignored

From previous analyses:

$P_{s(H)} = P_{a(H)} = 2405.5$ lb/ft and $Y_s = 3.45'$

$\therefore \quad M_o = (2405.5 \text{ lb/ft})\,(3.45')$
$M_o = 8299.0$ lb-ft/ft

Calculate the Factor of Safety against overturning, FS_{ot}:

[Eq. 5-23]

$FS_{ot} = M_r / M_o$
$FS_{ot} = (26170.3 \text{ lb-ft/ft}) / (8299.0 \text{ lb-ft/ft})$
$FS_{ot} = 3.15 > 2.00 \quad \therefore$ O. K.

5. Analyze bearing capacity

Check bearing capacity for a SRW with an exposed height of 8.00' and an embedment of 0.50':

[Eq. 5-28]

$FS_{bc} = Q_{ult} / Q_a$

Calculate the ultimate bearing capacity:

[Eq. 5-27]

$Q_{ult} = c_f N_c + 0.5\gamma_f(B)(N_\gamma) + \gamma_f(H_{emb})(N_q)$ Note: $c_f = 0$ lb/ft^2

From Figure 4-5 the bearing capacity factors for $\phi_f = 28^{o}$ are:

$N_\gamma = 16.72$ and $N_q = 14.72$

Compute the equivalent footing width, ***B***:

[Eq. 5-24]

$$B = L - 2e$$

Determine the eccentricity, ***e***, for an eccentric footing load:

[Eq. 5-25]

$$e = \frac{P_{s(H)} Y_s + P_{q(H)} Y_q - W_{ri}(X_{ri} - L/2) - W_{r\beta}(X_{r\beta} - L/2) - q_d L_\beta (X_{q\beta} - L/2)}{W_{ri} + W_{r\beta} + q_d L_\beta}$$

Note: In this example $P_{q(H)} = 0$ and is ignored

$$e = \frac{(2405.5\ \text{lb/ft})(3.45') - (6630.0\ \text{lb/ft})(3.47' - 6.5/2) - (614.7\ \text{lb/ft})(5.12' - 6.5/2)}{6630.0\ \text{lb/ft} + 614.7\ \text{lb/ft}}$$

$e = 0.785'$

$\therefore \quad B = 6.50' - 2(0.785')$
$B = 4.93'$

and
$Q_{ult} = 0.5(120\ \text{lb/ft}^3)(4.93')(16.72) + (120\ \text{lb/ft}^3)(0.50')(14.72)$
$Q_{ult} = 5828.9\ \text{lb/ft}^2$

Calculate the applied bearing stress, Q_a:

[Eq. 5-26]

$Q_a = [W_{ri} + W_{r\beta} + (q_l + q_d)(L_\beta)] / B$
$Q_a = (6630.0\ \text{lb/ft} + 614.7\ \text{lb/ft} + 0\ \text{lb/ft})/(4.93')$
$Q_a = 1469.5\ \text{lb/ft}$

Calculate the Factor of Safety against bearing capacity failure, FS_{bs}:

[Eq. 5-28]

$FS_{bc} = Q_{ult} / Q_a$
$FS_{bc} = (5828.9\ \text{lb/ft}^2) / (1469.5\ \text{lb/ft}^2)$
$FS_{bc} = 3.97 > 2.0 \ \therefore$ O.K.

INTERNAL STABILITY CALCULATIONS

1. Determine internal soil force due to reinforced fill:

   [Eq. 5-11]

   $$P_{a(H)} = P_{s(H)} + P_{q(H)}$$

   Note: In this example $P_q = 0$ and is ignored

   [Eq. 5-29]

   $$P'_{s(H)} = 0.5(K_{a(int)})(\gamma_i)(H)^2 \cos(\delta_i - \omega)$$

   Calculate $K_{a(int)}$:

   [Eq. 3-17]

   $$\delta_i = 0.667(\phi_i) = 0.667(28^\circ) = 18.7^\circ$$

   [Eq. 3-11]

   $$K_{a(int)} = \frac{\cos^2(\phi_i + \omega)}{\cos^2\omega \cos(\omega - \delta_i)\left[1 + \sqrt{\dfrac{\sin(\phi_i + \delta_i)\sin(\phi_i - \beta)}{\cos(\omega - \delta_i)\cos(\omega + \beta)}}\right]^2}$$

   $$K_{a(int)} = \frac{\cos^2(28^\circ + 3^\circ)}{\cos^2(3^\circ)\cos(3^\circ - 18.7^\circ)\left[1 + \sqrt{\dfrac{\sin(28^\circ + 18.7^\circ)\sin(28^\circ - 18.4^\circ)}{\cos(3^\circ - 18.7^\circ)\cos(3^\circ + 18.4^\circ)}}\right]^2}$$

   $$K_{a(int)} = 0.409$$

   Therefore,

   $$P'_{a(H)} = P'_{s(H)} = 0.5(0.409)(120\ \text{lb/ft}^3)(8.50')^2 \cos(18.7^\circ - 3^\circ)$$
   $$P'_{a(H)} = 1706.9\ \text{lb/ft}$$

2. Determine long-term design strength of reinforcement using NCMA Method "A" and the minimum number of reinforcement layers required:

   Select a geogrid with an ultimate tensile strength, T_{ult} = 2625 lb/ft

[Eq. 3-21]

$$T_a = \frac{T_{ult}}{RD_D \cdot RD_{ID} \cdot RF_{CR} \cdot FS_{UNC}}$$

$$T_a = \frac{2625 \text{ lb / ft}}{1.20 \cdot 1.25 \cdot 1.66 \cdot 1.5}$$

$$T_a = 700 \text{ lb/ft}$$

Determine the minimum number of layers of geogrid reinforcement using $\boldsymbol{T_a} = 700$ lb/ft:

[Eq. 5-33]

$$N_{min} = P'_{a(H)} / T_a$$
$$N_{min} = 1679.6 \text{ lb/ft} / 700 \text{ lb/ft}$$
$$N_{min} = 2.4$$

Note: Due to shear and connection strength capacity of some SRW units, spacing between layers of reinforcement may need to be limited to ensure successful construction. Consult the segmental unit manufacturer for recommended spacings. For this example a maximum spacing of 24" shall be assumed.

$$\therefore \quad N_{min} = 4$$

3. Select preliminary reinforcement elevations, $\boldsymbol{E_i}$, and determine factor of safety against tensile overstress. It is generally assumed the first layer of reinforcement is placed at one to three units from the base. For this example a 24" maximum spacing will be used thereafter. Reinforcement spacing generally increases with decreasing overburden.

Preliminary elevations: $E_{(1)} = 1.00'$, $E_{(2)} = 2.50'$, $E_{(3)} = 4.50'$, $E_{(4)} = 6.50'$

Determine the applied tensile load, $\boldsymbol{F_{g(n)}}$, of each layer:

[Eq. 5-36]

$$F_{g(n)} = (\gamma_i D_n + q_l + q_d)(K_{a(int)})(\cos \delta_i - \omega)(A_{c(n)})$$

Calculate the reinforcement contributory area, $A_{c(n)}$:

[Eq. 5-37]

$$A_{c(1)} = \frac{E_{(2)} + E_{(1)}}{2} = \frac{2.50' + 1.00'}{2} = 1.75'$$

[Eq. 5-38]

$$A_{c(2)} = \frac{E_{(3)} + E_{(2)}}{2} - \frac{E_{(2)} + E_{(1)}}{2} = \frac{4.50'+2.50'}{2} - \frac{2.50'+1.00'}{2} = 1.75'$$

$$A_{c(3)} = \frac{E_{(4)} + E_{(3)}}{2} - \frac{E_{(3)} + E_{(2)}}{2} = \frac{6.50'+4.50'}{2} - \frac{4.50'+2.50'}{2} = 2.00'$$

[Eq. 5-40]

$$A_{c(4)} = H - \frac{E_{(4)} + E_{(3)}}{2} = 8.50' - \frac{6.50' + 4.50'}{2} = 3.00'$$

Calculate the depth to the midpoint of $\boldsymbol{A_{c(n)}}$:

$$D_1 = H - (A_{c(1)}/2) = 8.50' - (1.75/2) = 7.62'$$
$$D_2 = H - A_{c(1)} - (A_{c(2)}/2) = 8.50' - 1.75' - (1.75'/2) = 5.87'$$
$$D_3 = H - A_{c(1)} - A_{c(2)} - (A_{c(3)}/2) = 8.50' - 1.75' - 1.75' - (2.00'/2) = 4.00'$$
$$D_4 = (A_{c(4)} / 2) = (3.00' / 2) = 1.50'$$

Calculate the applied tensile loads, $\boldsymbol{F_{g(n)}}$:

[Eq. 5-36]

$$F_{g(n)} = (\gamma_i)(D_n)(K_{a(int)})(\cos \delta_i - \omega)(A_{c(n)})$$
$$F_{g(1)} = (120 \text{ lb/ft}^3)(7.62')(0.409)(\cos 18.7^\circ - 3^\circ)(1.75') = 630.1 \text{ lb/ft}$$
$$F_{g(2)} = (120 \text{ lb/ft}^3)(5.87')(0.409)(\cos 18.7^\circ - 3^\circ)(1.75') = 485.4 \text{ lb/ft}$$
$$F_{g(3)} = (120 \text{ lb/ft}^3)(4.00')(0.409)(\cos 18.7^\circ - 3^\circ)(2.00') = 378.1 \text{ lb/ft}$$
$$F_{g(4)} = (120 \text{ lb/ft}^3)(1.50')(0.409)(\cos 18.7^\circ - 3^\circ)(3.00') = 212.6 \text{ lb/ft}$$

Compare the applied tensile load, $\boldsymbol{F_{g(n)}}$, to long-term design strength, $\boldsymbol{T_a}$:

[Eq. 5-32]

$$F_{g(n)} \leq T_{a(n)}$$
$F_{g(1)} = 630.1 \text{ lb/ft} < T_{a(1)} = 700 \text{ lb/ft} \therefore$ O.K.
$F_{g(2)} = 485.4 \text{ lb/ft} < T_{a(2)} = 700 \text{ lb/ft} \therefore$ O.K.
$F_{g(3)} = 378.1 \text{ lb/ft} < T_{a(2)} = 700 \text{ lb/ft} \therefore$ O.K
$F_{g(4)} = 212.6 \text{ lb/ft} < T_{a(2)} = 700 \text{ lb/ft} \therefore$ O.K

4. Analyze the pullout resistance of the reinforcement.

Calculate each reinforcement layer's anchorage length past the failure plane, $\boldsymbol{L_{a(n)}}$:

[Eq. 5-46]

$$L_{a(n)} = L_{(n)} - W_u - E_{(n)} \tan(90^\circ - \alpha_i) + E_{(n)} \tan \omega$$ Note: $\alpha_i = 46.5^\circ$ using Eq. 3-14

$L_{a(1)} = 6.50' - 1.00' - 1.00' \tan(90^\circ - 46.5^\circ) - 1.00' \tan 3^\circ = 4.60'$
$L_{a(2)} = 6.50' - 1.00' - 2.50' \tan(90^\circ - 46.5^\circ) - 2.50' \tan 3^\circ = 3.26'$
$L_{a(3)} = 6.50' - 1.00' - 4.50' \tan(90^\circ - 46.5^\circ) - 4.50' \tan 3^\circ = 1.47'$
$L_{a(4)} = 6.50' - 1.00' - 6.50' \tan(90^\circ - 46.5^\circ) - 6.50' \tan 3^\circ = -0.33'$

Note: $\boldsymbol{L_{a(4)}}$ must be increased to a minimum of 1.00'.

$\therefore \quad L_{(4)} = 6.50' + 0.33' + 1.00' = 7.83'$ for $L_{a(4)} = 1.00'$

Determine the average depth of overburden, $\boldsymbol{d_{(n)}}$:

[Eq. 5-47]

$d_{(n)} = (H - E_{(n)}) + [(E_{(n)} / \tan \alpha_i) - (H \tan \omega) + (L_{a(n)}/2)] \tan \beta$
$d_{(1)} = (8.50' - 1.00') + [(1.00'/\tan 46.5^\circ) - (8.50' \tan 3^\circ) + (4.60'/2)] \tan 18.4^\circ = 8.43'$
$d_{(2)} = (8.50' - 2.50') + [(2.50'/\tan 46.5^\circ) - (8.50' \tan 3^\circ) + (3.26'/2)] \tan 18.4^\circ = 7.18'$
$d_{(3)} = (8.50' - 4.50') + [(4.50'/\tan 46.5^\circ) - (8.50' \tan 3^\circ) + (1.47'/2)] \tan 18.4^\circ = 5.52'$
$d_{(4)} = (8.50' - 6.50') + [(6.50'/\tan 46.5^\circ) - (8.50' \tan 3^\circ) + (1.00'/2)] \tan 18.4^\circ = 4.07'$

Calculate the anchorage capacity, $\boldsymbol{AC_{(n)}}$, of each reinforcement layer:

[Eq. 5-45]

$AC_{(n)} = 2\,(L_{a(n)})(C_i)(d_{(n)}\gamma_i + q_d) \tan \phi_i$ Note: In this example $q_d = 0$
$AC_{(1)} = 2\,(4.60')(0.7)(8.43')(120\ \text{lb/ft}^3) \tan 28^\circ = 3463.9$ lb/ft
$AC_{(2)} = 2\,(3.26')(0.7)(7.18')(120\ \text{lb/ft}^3) \tan 28^\circ = 2090.9$ lb/ft
$AC_{(3)} = 2\,(1.47')(0.7)(5.52')(120\ \text{lb/ft}^3) \tan 28^\circ = 724.8$ lb/ft
$AC_{(4)} = 2\,(1.00')(0.7)(4.07')(120\ \text{lb/ft}^3) \tan 28^\circ = 363.6$ lb/ft

Determine the factor of safety against pullout, $\boldsymbol{FS_{po(n)}}$:

[Eq. 5-44]

$FS_{po(n)} = AC_{(n)} / F_{g(n)}$
$FS_{po(1)} = AC_{(1)} / F_{g(1)} = (3463.9\ \text{lb/ft}) / (630.1\ \text{lb/ft}) = 5.49 > 1.5 \ \therefore$ O.K.
$FS_{po(2)} = AC_{(2)} / F_{g(2)} = (2090.9\ \text{lb/ft}) / (485.4\ \text{lb/ft}) = 4.31 > 1.5 \ \therefore$ O.K.
$FS_{po(3)} = AC_{(3)} / F_{g(3)} = (724.8\ \text{lb/ft}) / (378.1\ \text{lb/ft}) = 1.92 > 1.5 \ \therefore$ O.K.
$FS_{po(4)} = AC_{(4)} / F_{g(4)} = (363.6\ \text{lb/ft}) / (212.6\ \text{lb/ft}) = 1.71 > 1.5 \ \therefore$ O.K.

5. Analyze internal sliding along lowest reinforcement layer. The driving force, $\boldsymbol{P_{a(H)}}$, decreases proportionately with $\boldsymbol{H_s}^2$ and the resisting forces, $\boldsymbol{R'_{s(n)}}$ and $\boldsymbol{V_u}$, decrease proportionately with $\boldsymbol{H}$ for this example problem. Therefore, if the lowest geogrid reinforcement layer has an adequate $\boldsymbol{FS_{sl(n)}}$ then all other geogrid layers will adequately resist the associated active earth forces.

Determine internal sliding reinforcement lengths:

[Eq. 5-51]

$$\Delta L = (E_{(n+1)} - E_{(n)}) / \tan \alpha_e$$

Note: $\alpha_e = 45.2^o$ using Eq. 3-14

$$\Delta L = (2.50' - 1.00') / \tan 45.2^o$$

$$\Delta L = 1.49'$$

[Eq. 5-50]

$$L'_{s(n)} = L - W_u - \Delta L$$

$$L'_{s(1)} = 6.50' - 1.00' - 1.49'$$

$$L'_{s(1)} = 4.01'$$

[Eq. 5-52]

$$L''_{s(n)} = \frac{L'_{s(n)} \tan\beta \ \tan\omega}{1 - \tan\beta \ \tan\omega}$$

$$L''_{s(1)} = \frac{(4.01')\tan 18.4^o \tan 3^o}{1 - \tan 18.4^o \tan 3^o}$$

$$L''_{s(1)} = 0.072'$$

[Eq. 5-53]

$$L_{\beta(n)} = L'_{s(n)} + L''_{s(n)}$$

$$L_{\beta(n)} = 4.01' + 0.072'$$

$$L_{\beta(n)} = 4.08'$$

[Eq. 5-54]

$$h_{(n)} = L_{\beta(n)} \tan \beta$$

$$h_{(n)} = 4.08' \tan 18.4^o$$

$$h_{(n)} = 1.36'$$

[Eq. 5-55]

$$W'_{r(i,n)} = (L'_{s(n)})(H - E_{(n)})(\gamma_i)$$

$$W'_{r(1)} = (4.01')(8.50'-1.00')(120 \text{ lb/ft}^3)$$

$$W'_{r(1)} = 3609.0 \text{ lb/ft}$$

[Eq. 5-56]

$$W'_{r(\beta,n)} = [(\gamma_i)(L_{\beta(n)})(L'_{s(n)})(\tan\beta)]/2$$

$$W'_{r\beta(1)} = [(120 \text{ lb/ft}^3)(4.08')(4.01')(\tan 18.4^o)]/2$$

$$W'_{r\beta(1)} = 326.6 \text{ lb/ft}$$

Determine the sliding resistance at lowest geogrid reinforcement layer, $\boldsymbol{R'_{s(1)}}$:

[Eq. 5-49]

$R'_{s(n)} = C_{ds}\,(q_d L_{\beta(n)} + W'_{r(i,n)} + W'_{r(\beta,n)}) \tan\phi_i$ Note: In this example $\boldsymbol{q_d} = 0$

$R'_{s(1)} = 0.95(3609.0 \text{ lb/ft} + 326.6 \text{ lb/ft}) \tan 28^{o}$

$R'_{s(1)} = 1988.0 \text{ lb/ft}$

Calculate the shear capacity of the segmental concrete units at the lowest geogrid reinforcement layer using the shear capacity envelope determined by laboratory testing for flat interface units, i.e. $\boldsymbol{a_u} = 400$ lb/ft and $\boldsymbol{\lambda_u} = 30^{o}$.

[Eq. 4-25]

$V_{u(n)} = a_{u(n)} + W_{w(n)} \tan \lambda_u$

$V_{u(n)} = 400 \text{ lb/ft} + W_{w(n)} \tan 30^{o}$

[Eq. 4-9] (Modified to determine the weight of the column of SRW units on a reinforcement layer.)

$W_{w(n)} = (H_h - E_{(n)})\,(\gamma_u)(W_u)$

Confirm whether the hinge height, $\boldsymbol{H_h}$, or total wall height, $\boldsymbol{H}$, is appropriate:

since

$H = 8.50'$

and

[Eq. 4-1]

$H_h = 2(W_u - G_u) / \tan \omega$

$H_h = 2(1.00' - 0.50')/\tan 3^{o}$

$H_h = 19.08'$

$H_h > H$ use $H = 8.50'$

$\therefore\ W_{w(1)} = (8.50'-1.00')(120 \text{ lb/ft}^3)(1.00')$

$W_{w(1)} = 900.0 \text{ lb/ft}$

and

$V_{u(1)} = 400 \text{ lb/ft} + (900.0 \text{ lb/ft}) \tan 30^{o}$

$V_{u(1)} = 919.6 \text{ lb/ft}$

Determine the active earth force at the lowest geogrid reinforcement layer, $\boldsymbol{P_{a(H,1)}}$:

[Eq. 5-31]

$P'_{a(H,n)} = P'_{s(H,n)} + P'_{q(H,n)}$ Note: In this example $\boldsymbol{P'_{q(H)}} = 0$ and is ignored

$\therefore\ P_{a(H,n)} = P_{s(H,n)}$

[Eq. 5-6]

$$P_{s(H,n)} = 0.5(K_{a(ext)})(\gamma_r)(H - E_{(n)} + h_{(n)})^2 \cos(\delta_e - \mapsto)$$

from previous calculations $K_{a(ext)} = 0.412$, $\delta_e = 28^o$

$$\therefore \quad P_{a(H,1)} = 0.5(0.412)(120 \text{ lb/ft}^3)(8.50'\text{-}1.00'\text{+}1.36')^2 \cos(28^o\text{-}3^o)$$
$$P_{a(H,1)} = 1758.7 \text{ lb/ft}$$

Calculate the Factor of Safety against internal sliding at lowest geogrid reinforcement layer failure, $FS_{sl(n)}$:

[Eq. 5-48]

$$FS_{sl(n)} = (R'_{s(n)} + V_{u(n)}) \,/\, P_{a(H,n)}$$
$$FS_{sl(1)} = (1988.0 \text{ lb/ft} + 919.6 \text{ lb/ft}) \,/\, (1758.7 \text{ lb/ft})$$
$$FS_{sl(1)} = 1.65 > 1.5 \quad \therefore \text{O.K.}$$

Since internal sliding at the lowest geogrid reinforcement layer has an adequate FS_{sl} then all other geogrid reinforcement layers will adequately resist the associated active external earth forces.

6. Analyze the connection strength between each geogrid reinforcement layer and the segmental concrete units:

[Eq. 5-59]

$$T_{ultconn} = a_{cs} + W_{w(n)} \tan \lambda_{cs}$$

Calculate the connection strength of the segmental concrete units and geogrid reinforcement layers using the connection strength envelope determined by laboratory testing for flat interface units, i.e. a_{cs} = 200 lb/ft and $\lambda_{cs} = 40^o$.

$$T_{ultconn(n)} = 200 \text{ lb/ft} + W_{w(n)} \tan 40^o$$

From previous internal sliding resistance calculations the hinge height (H_h = 19.08') is greater than the total wall height (H = 8.50'). Therefore, use H in the calculation of $W_{w(n)}$:

[Eq. 4- 9]

$$W_{w(n)} = (H - E_{(n)})(\gamma_u)(W_u)$$

@ $E_{(1)}$ $W_{w(1)} = (H - E_{(1)})(\gamma_u)(W_u)$

$W_{w(1)} = (8.50'-1.00')(120\ lb/ft^3)(1.00')$

$W_{w(1)} = 900.0\ lb/ft$

$T_{ultconn(1)} = 200\ lb/ft + (900.0\ lb/ft)\ \tan 40^o$

$T_{ultconn(1)} = 955.2\ lb/ft$

@ $E_{(2)}$ $W_{w(2)} = (H - E_{(2)})(\gamma_u)(W_u)$

$W_{w(2)} = (8.50'-2.50')(120\ lb/ft^3)(1.00')$

$W_{w(2)} = 720.0\ lb/ft$

$T_{ultconn(2)} = 200\ lb/ft + (720.0\ lb/ft)\ \tan 40^o$

$T_{ultconn(2)} = 804.1\ lb/ft$

@ $E_{(3)}$ $W_{w(3)} = (H - E_{(3)})(\gamma_u)(W_u)$

$W_{w(3)} = (8.50'-4.50')(120\ lb/ft^3)(1.00')$

$W_{w(3)} = 480.0\ lb/ft$

$T_{ultconn(3)} = 200\ lb/ft + (480.0\ lb/ft)\ \tan 40^o$

$T_{ultconn(3)} = 602.8\ lb/ft$

@ $E_{(4)}$ $W_{w(4)} = (H - E_{(4)})(\gamma_u)(W_u)$

$W_{w(4)} = (8.50'-6.50')(120\ lb/ft^3)(1.00')$

$W_{w(4)} = 240.0\ lb/ft$

$T_{ultconn(4)} = 200\ lb/ft + (240.0\ lb/ft)\ \tan 40^o$

$T_{ultconn(4)} = 401.4\ lb/ft$

Compare the applied tensile force, $F_{g(n)}$, to the long-term allowable connection strength, $T_{cl(n)}$:

[Eq. 5-57]

$$T_{cl(n)} = T_{ultconn(n)}/FS_{cs} \leq T_{a(n)}$$

From previous calculations:

$F_{g(1)} = 630.1\ lb/ft$

$F_{g(2)} = 485.4\ lb/ft$

$F_{g(3)} = 378.1\ lb/ft$

$F_{g(4)} = 212.6\ lb/ft$

@ $E_{(1)}$ $T_{cl(1)} = (955.2\ lb/ft) / 1.5 = 636.8\ lb/ft$ <700 lb/ft

$\therefore$ $T_{cl(1)} = 636.8\ lb/ft > F_{g(1)} = 630.1\ lb/ft$ O.K.

@ $E_{(2)}$ $T_{cl(2)} = (804.1\ lb/ft) / 1.5 = 536.1\ lb/ft$ <700 lb/ft

$\therefore$ $T_{cl(2)} = 536.1\ lb/ft > F_{g(2)} = 485.4\ lb/ft$ O.K.

@ $E_{(3)}$ $T_{cl(3)} = (602.8\ lb/ft) / 1.5 = 401.9\ lb/ft$ <700 lb/ft

$\therefore$ $T_{cl(3)} = 401.9\ lb/ft > F_{g(3)} = 378.1\ lb/ft$ O.K.

@ $E_{(4)}$ $T_{cl(4)} = (401.4\ lb/ft) / 1.5 = 267.6\ lb/ft$ <700 lb/ft

$\therefore$ $T_{cl(4)} = 267.6\ lb/ft > F_{g(4)} = 212.6\ lb/ft$ O.K.

7. Analyze the resistance to bulging at each geogrid reinforcement layer elevation, $\boldsymbol{E_{(n)}}$:

Calculate the horizontal active earth force at $\boldsymbol{E_{(n)}}$:

[Eq. 5-31]

$$P'_{a(H,n)} = P'_{s(H,n)} + P'_{q(H,n)} \qquad \text{Note: } \boldsymbol{P'_{q(H)}} = 0 \text{ and is ignored}$$
$$\therefore \quad P'_{a(H,n)} = P'_{s(H,n)}$$

[Eq. 5-29]

$$P'_{s(H,n)} = 0.5(K_{a(int)})(\gamma_i)(H - E_{(n)})^2 \cos(\delta_i - \omega)$$

From previous calculations $\boldsymbol{K_{a(int)}} = 0.409$ and $\delta_i = 18.7^{\circ}$

$$@\ E_{(1)} \quad P'_{a(H,1)} = 0.5(0.409)(120 \text{ lb/ft}^3)(8.50'-1.00')^2 \cos(18.7^{\circ} - 3^{\circ}) = 1328.9 \text{ lb/ft}$$
$$@\ E_{(2)} \quad P'_{a(H,2)} = 0.5(0.409)(120 \text{ lb/ft}^3)(8.50'-2.50')^2 \cos(18.7^{\circ} - 3^{\circ}) = 850.5 \text{ lb/ft}$$
$$@\ E_{(3)} \quad P'_{a(H,3)} = 0.5(0.409)(120 \text{ lb/ft}^3)(8.50'-4.50')^2 \cos(18.7^{\circ} - 3^{\circ}) = 378.0 \text{ lb/ft}$$

Calculate the available segmental concrete unit shear capacity at $\boldsymbol{E_{(n)}}$ using the shear capacity envelope determined by laboratory testing for flat interface units, i.e. $\boldsymbol{a_u} = 400$ lb/ft and $\lambda_u = 30^{\circ}$:

[Eq. 4-25]

$$V_{u(n)} = a_{u(n)} + W_{w(n)} \tan \lambda_u$$
$$V_{u(n)} = 400 \text{ lb/ft} + W_{w(n)} \tan 30^{\circ}$$

[Eq. 4- 9]

$$W_{w(n)} = (H - E_{(n)})\ (\gamma_u)(W_u) \qquad \text{Note: Use } \boldsymbol{H} \text{ since } \boldsymbol{H_h > H}$$

$$@\ E_{(1)} \quad V_{u(1)} = 400 \text{ lb/ft} + (8.50'-1.00')(120 \text{ lb/ft}^3)(1.00') \tan 30^{\circ} = 919.6 \text{ lb/ft}$$
$$@\ E_{(2)} \quad V_{u(2)} = 400 \text{ lb/ft} + (8.50'-2.50')(120 \text{ lb/ft}^3)(1.00') \tan 30^{\circ} = 815.7 \text{ lb/ft}$$
$$@\ E_{(3)} \quad V_{u(3)} = 400 \text{ lb/ft} + (8.50'-4.50')(120 \text{ lb/ft}^3)(1.00') \tan 30^{\circ} = 677.1 \text{ lb/ft}$$

Determine the factor of safety against shear failure, $\boldsymbol{FS_{sc(n)}}$:

[Eq. 5-61]

$$FS_{sc(n)} = V_{u(n)} / [P_{a(H,n)} - (F_{g(i)} + \ldots + F_{g(n)})]$$

From previous calculations:

$$F_{g(1)} = 630.1 \text{ lb/ft}$$
$$F_{g(2)} = 485.4 \text{ lb/ft}$$
$$F_{g(3)} = 378.1 \text{ lb/ft}$$
$$F_{g(4)} = 212.6 \text{ lb/ft}$$

@ $E_{(1)}$ $FS_{sc(1)} = V_{u(1)} / [P_{a(H,1)} - (F_{g(2)} + F_{g(3)} + F_{g(4)})]$

$FS_{sc(1)} = (919.6 \text{ lb/ft}) / [1328.9 \text{ lb/ft} - (485.4 \text{ lb/ft} + 378.1 \text{ lb/ft} + 212.6 \text{ lb/ft})]$

$FS_{sc(1)} = 3.64 > 1.5$ O.K.

@ $E_{(2)}$ $FS_{sc(2)} = V_{u(2)} / [P_{a(H,2)} - (F_{g(3)} + F_{g(4)})]$

$FS_{sc(2)} = (815.7 \text{ lb/ft}) / [850.5 \text{ lb/ft} - (378.1 \text{ lb/ft} + 212.6 \text{ lb/ft})]$

$FS_{sc(2)} = 3.14 > 1.5$ O.K.

@ $E_{(3)}$ $FS_{sc(3)} = V_{u(3)} / [P_{a(H,3)} - (F_{g(4)})]$

$FS_{sc(2)} = (677.1 \text{ lb/ft}) / [378.0 \text{ lb/ft} - (212.6 \text{ lb/ft})]$

$FS_{sc(2)} = 4.09 > 1.5$ O.K.

8. Analyze the unreinforced SRW height at the top of the wall for factor of safety against overturning, $\boldsymbol{FS_{ot}}$ in a similar manner as for the analysis of a conventional SRW:

[Eq. 4- 5]

$$P'_{s(H,4)} = 0.5(K_{a(int)})(\gamma_i)(H - E_{(4)})^2 \cos(\delta_i - \omega)$$

since

$$P'_{q(H)} = 0, \; P'_{a(H,4)} = P'_{s(H,4)}$$
$$P'_{a(H,4)} = 0.5(0.409)(120 \text{ lb/ft}^3)(8.50'-6.50')^2 \cos(18.7^o - 3^o)$$
$$P'_{a(H,4)} = 94.5 \text{ lb/ft}$$

[Eq. 4- 7]

$$Y'_{s(4)} = (H - E_{(4)})/3 = (8.50'-6.50')/3 = 0.67'$$

Determine the overturning moments:

[Eq. 4-17]

$$M_{o(4)} = P_{s(H,4)}Y_{s(4)} + P_{q(H,4)}Y_{q(4)}$$ Note: $\boldsymbol{P_{q(4)}} = 0$ lb/ft

$$M_{o(4)} = (94.5 \text{ lb/ft})(0.67')$$
$$M_{o(4)} = 63.3 \text{ lb-ft/ft}$$

Determine the resisting moment arm, $X_{w(4)}$, and moment $M_{r(4)}$:

[Eq. 4-15]

$$M_{r(4)} = W_{w(4)}(X_{w(4)})$$

[Eq. 4-9]

$$W_{w(4)} = (H - E_{(n)})(\gamma_u)(W_u)$$ Note: Use $\boldsymbol{H}$ since $\boldsymbol{H_h > H}$

$$W_{w(4)} = (8.50'-6.50')(120 \text{ lb/ft}^3)(1.00')$$
$$W_{w(4)} = 240 \text{ lb/ft}$$

[Eq. 4-16]

$X_{w(4)} = G_u + 0.5[(H - E_{(n)} - H_u)\tan \omega]$
$X_{w(4)} = 0.5 + 0.5[(8.50' - 6.50' - 0.50') \tan 3^o]$
$X_{w(4)} = 0.54'$

$\therefore$ $M_{r(4)} = (240 \text{ lb/ft})(0.54')$
$M_{r(4)} = 129.4 \text{ lb-ft/ft}$

Calculate the Factor of Safety against overturning, $\boldsymbol{FS_{ot(4)}}$:

[Eq. 4-14]

$FS_{ot(4)} = M_{r(4)}/M_{o(4)}$
$FS_{ot(4)} = (129.4 \text{ lb-ft/ft}) / (63.3 \text{ lb-ft/ft})$
$FS_{ot(4)} = 2.04 > 2.00$ O.K.

9. Analyze the Factor of Safety against shear capacity failure at the uppermost geogrid reinforcement layer, $\boldsymbol{FS_{sc(4)}}$ in a manner similar to that of a conventional SRW:

[Eq. 4-25]

$V_{u(4)} = a_{u(4)} + W_{w(4)} \tan \lambda_u$
$V_{u(4)} = 400 \text{ lb/ft} + W_{w(4)} \tan 30^o$

[Eq. 4- 9]

$W_{w(4)} = (H - E_{(4)})(\gamma_u)(W_u)$ Note: Use $\boldsymbol{H}$ since $\boldsymbol{H_h > H}$

@ $E_{(4)}$ $V_{u(4)} = 400 \text{ lb/ft} + (8.50'-6.50')(120 \text{ lb/ft}^3)(1.00') \tan 30^o = 538.6 \text{ lb/ft}$

[Eq. 4-27]

$FS_{sc(4)} = V_{u(4)}/P_{a(H,4)}$
$FS_{sc(4)} = (538.6 \text{ lb/ft}) / (94.5 \text{ lb/ft})$
$FS_{sc(4)} = 5.70 > 1.5$ O.K.

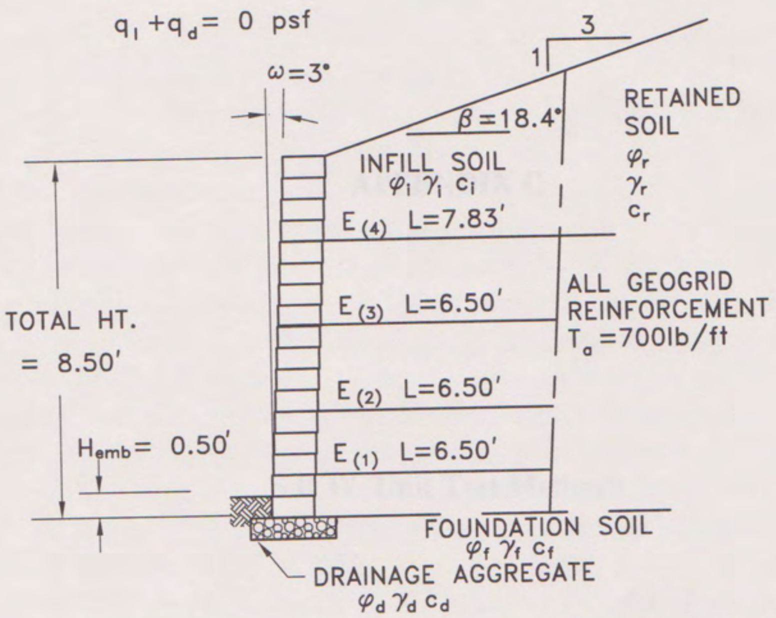


FIGURE B–2: REINFORCED SOIL SRW FINAL DESIGN

<u>External:</u>

FS_{sl} = 1.60 > 1.5

FS_{ot} = 3.15 > 2.0

FS_{bc} = 3.97 > 2.0

<u>Top of Wall:</u>

$FS_{ot(4)} = 2.04 > 2.00$

$FS_{sc(4)} = 5.70 > 1.5$

<u>Internal:</u>

Tensile Load: All $F_{g(n)} < T_{a(n)}$

Pullout: All $FS_{po(n)} > 1.5$

Internal Sliding at Geogrids: All internal FS_{sl} >1.5

Connection Strength: All $T_{cl(n)} > F_{g(n)}$

Shear Capacity: All $FS_{sc(n)} > 1.5$

11.3 Report the ultimate (peak) and service state shear strength of each test and the applied normal surcharge load used in the test. In addition, report the average results of each test series at the same applied normal surcharge.

11.4 For each test, show a plot of shear force versus average deformation recorded at the top concrete unit (see **Figure C.2-2**).

11.5 Provide a summary table of ultimate (peak) and service state shear strengths at each surcharge load and the average of any repeat tests (see **Table C.2-1**). Identify the geosynthetic sample on the table if applicable.

11.6 Summarize the results of connection tests on a plot of:

a) Shear strength based on ultimate (peak) load criterion versus surcharge load, and
b) Shear strength based on deformation criterion versus surcharge load (if applicable)

as shown in **Figure C.2-3**. The trend in the data may be approximated by one or more line segments using linear regression methods.

11.7 Report on the type of failure(s), its location and description.

11.8 Provide a sketch and photograph (optional) of the test setup and test specimen stacking configuration used.

11.9 Provide a grain size distribution curve of the granular infill used for placement in and between segmental concrete units (see **Figure C.2-4**).

12. Precision and Bias

12.1 Precision - The precision of this test method has not been established.

12.2 Bias - The true value of this test method can only be defined in terms of a specific test method. Within this limitation, the procedure described herein has no known bias.

Appendix C

SRW Unit Test Methods

Within this manual, the SRW unit properties required to design conventional and reinforced soil segmental walls were defined. Two performance properties, shear strength and connection strength, are critical in the analyses of wall sections. In order to establish the shear and connection strength capacities of SRW units test methods had to be developed. Included in Appendix C are NCMA connection and shear strength test methods **SRWU-1** - Determination of Connection Strength between Geosynthetics and Segmental Concrete Units and **SRWU-2** - Determination of Shear Strength between Segmental Concrete Units. When the tests are conducted on SRW systems, the results will indicate the performance differences between various segmental units. These results are used to determine the specific SRW system and the performance properties incorporated into the standard NCMA design methodology.

C.1 NCMA Test Method SRWU-1

"Determination of Connection Strength between Geosynthetics and Segmental Concrete Units"

1. Scope

This test method is used to determine the connection strength between a layer of geosynthetic reinforcement and segmental concrete block units used in construction of reinforced soil retaining walls. Each test is carried out under conditions that reproduce the connection system in the field at full-scale. The connection strength is defined based on ultimate strength (ultimate connection strength criterion) or tensile load measured after a prescribed amount of deformation recorded at the back of the segmental units (service state connection strength criterion). The results of a series of tests are used to define a relationship between connection strength for a segmental unit-geosynthetic connection system and surcharge load (or height of stacked units above the connection elevation).

2. Referenced Documents

2.1 NCMA TEK 2-4

2.2 ASTM Standards:
D 4595 - Test Method for Tensile Properties of Geotextiles by the Wide-Width Strip Method
D4354 - Practice for Sampling of Geotextiles for Testing

2.3 Bathurst, R.J., and Simac, M.R., "Laboratory Testing of Modular Concrete Block-Geogrid Facing Connections," ASTM Symposium on Geosynthetic Soil Reinforcement Testing, San Antonio, Texas, January 19, 1993

3. Definitions

3.1 **Ultimate Connection Strength** - The maximum tensile capacity of the connection as tested in accordance with the procedures of NCMA **SRWU-1**.

3.2 **Service State Connection Strength** - The tensile capacity of the connection system based on a service state deformation criterion and tested in accordance with NCMA **SRWU-1**. The deformation criterion is the maximum permissible deformation of the geosynthetic reinforcement at the back of the segmental units that will not affect the performance of the wall in service. The deformation criterion is typically prescribed in advance of testing to satisfy retaining wall performance criteria. If a deformation criterion is not specified a value of 0.75 inches (19 mm) should be used.

3.3 **Geosynthetic Reinforcement** - A planar structure consisting of a network of connected polymeric tensile elements used as a horizontal reinforcement material in geotechnical and civil engineering applications (geotextile or geogrid).

3.4 **Segmental Concrete Unit** - Modular concrete block manufactured specifically for mortarless retaining wall construction. The concrete units may be dry-cast machine molded or wet-cast and are made from portland cement, water, and mineral aggregates with or without the inclusion of other materials.

3.5 **Wall Facing** - Stacked segmental concrete block units placed in front of the reinforced soil zone. The units are positively connected to geosynthetic reinforcement layers by a built-in concrete shear key or by interface friction developed between concrete block layers with or without the aid of mechanical connectors.

3.6 **Granular Infill** - Coarse grained soil aggregate used to fill the voids in and between segmental concrete units as recommended by the segmental concrete unit manufacturer. An example is a crushed stone conforming to AASHTO No. 57 gradation.

4. Summary of Test Method

In this test method, a wide width geosynthetic reinforcement test specimen is attached to the stacked segmental block units as recommended for field construction. The top of the segmental block units is then loaded vertically to a constant surcharge load and the geosynthetic reinforcement tensioned under constant rate of displacement until failure of the connection system occurs. Failure of the system is defined as sustained loss of connection capacity or deformation of the reinforcement. Ultimate connection capacity and tensile capacity after a prescribed deformation has been recorded at the back of the segmental units, are used to define connection strength based on ultimate strength and service state criteria respectively. Tensile loads and strengths are reported in lb/ft width of geosynthetic sample (or kN/m).

5. Significance

5.1 The connection strength between geosynthetics and segmental concrete block units is used in design of reinforced soil retaining walls.

5.2 This test is used to determine connection strengths for the design of the connection system formed by segmental concrete block units and geosynthetic reinforcement layers in reinforced soil retaining walls.

5.3 This connection strength test is meant to be a performance test (laboratory or field); therefore, it should be conducted using full scale system components.

6. Apparatus

6.1 The Testing System - An example of a test apparatus and setup is illustrated in **Figure C.1-1**. The principal components of the test apparatus are:

- loading frame
- surcharge piston/actuator
- vertical loading platen with stiff rubber mat or airbag to apply uniform vertical pressure to top of concrete blocks
- vertical load cell to measure surcharge load
- geosynthetic loading clamp
- horizontal piston/actuator to load geosynthetic reinforcement in tension
- horizontal load cell to measure geosynthetic tensile force
- two horizontal displacement measurement devices to record displacement of the geosynthetic at the back of the segmental concrete blocks

6.2 Loading Frame - The loading frame must have sufficient capacity to act as a reaction to forces developed by the horizontal and vertical loading pistons/actuators.

6.3 The Geosynthetic Loading Clamp and Loading Assemblies - The geosynthetic is gripped at its free end with a clamp extending the full width of the sample. The clamp must be capable of applying a uniform force across the full width of the test specimen. For some geosynthetics, it may be necessary to epoxy bond the geosynthetic material to, or within, the clamp in order to obtain a uniform stress distribution across the entire width of the test specimen. A roller grip assembly may be used to apply the tensile load.

6.3.1 The tensile loading unit will generally be a constant rate of extension screw jack or hydraulic actuator that can be deformation rate controlled. The loading equipment should have a capacity that is at least equal to 100% of the wide strip tensile strength of the geosynthetic material (ASTM D 4595) multiplied by the sample width. It is recommended that the piston be capable of at least 6 inches of movement in order to facilitate setting up and to ensure that there is adequate stroke to achieve failure of extensible geosynthetic reinforcement samples.

6.3.2 The orientation of the tensioning force shall be horizontal and perpendicular to the back of the segmental units and shall be applied at the elevation where the geosynthetic exits the back of the segmental units.

6.4 Load Cells - A calibrated load cell shall be used to measure the tensile connection force and normal load during the test. The load cell used for measuring tension must have a capacity that is greater than or equal to 100% of the wide strip tensile strength of the geosynthetic material (ASTM D 4595) multiplied by the sample width. The load cell used for measuring the normal surcharge load should have a capacity that is greater than or equal to 100 % of the maximum anticipated normal load. The load cells should be accurate within $\pm$0.5% of its full scale range.

6.5 Displacement Measuring Devices - A minimum of two Linear Variable Displacement Transducers (LVDTs) or similar electronic displacement measuring devices are recommended to continuously monitor the displacement of the geosynthetic material at the back of the concrete units. Alternatively, dial gauges may be read and recorded manually at regular intervals. LVDTs, gauges or similar measuring devices must be accurate to $\pm$ 0.005 inches (0.1 mm).

7. Test Specimens

7.1 Segmental Concrete Units

7.1.1 Segmental concrete units must be full-size blocks and meet the manufacturers material and dimensional specifications.

7.1.2 The wall for connection testing must be constructed with full-size segmental units selected from a standard production lot. Model or prototype units should not be used unless it can be demonstrated that they are equivalent to production units.

7.1.3 Segmental concrete units may be reused in testing if there is no cracking, abrasion or wearing of the concrete surfaces between tests.

7.2 Geosynthetic Reinforcement

7.2.1 Sampling Requirements - The latest version of ASTM sampling protocol for geotextiles (ASTM D 4354) should be used for the geosynthetic material.

7.2.2 Conditioning - The test specimen should be brought to standard temperature and relative humidity conditions. The temperature is to be 21 $\pm$ 2°C (70 $\pm$ 4°F) and the relative humidity of 65 $\pm$ 5%.

7.2.3 Specimen Width - The minimum width of geosynthetic test specimen shall be 36 inches (910 mm) unless it can be demonstrated that narrower samples will give connection performance that is equivalent to 36 inch wide samples. In no case shall the width of sample be less than the twice the distance between running joints for concrete units with a width of 18 inches (405 mm) or less. The geosynthetic sample width shall also cover at least two complete repeating patterns of any mechanical connectors used in the segmental concrete unit assembly.

7.2.4 Specimen Length - The geosynthetic specimen shall have sufficient length to cover the interface surface as recommended by the manufacturer. The sample must be trimmed to provide sufficient anchorage at the geosynthetic specimen loading clamp and at least 8 inches (200 mm) of free length between the back of the concrete blocks and loading clamp. In no case shall the reinforcement extend beyond the front face of the stacked concrete units.

7.2.5 A virgin sample of geosynthetic material shall be used for each test.

7.3 Test Assembly and Segmental Unit Construction

7.3.1 The connection shall be constructed as in the field using the same geosynthetic reinforcement, granular infill, concrete block units and connectors. The number, type and arrangement of connectors must be identical to that used in the field.

7.3.2 A single layer of segmental units shall be placed on a rigid base. A second layer of segmental units shall be placed over the bottom layer with the geosynthetic material located and placed between the layers as described in the construction specifications. Both layers of segmental units shall be rigidly braced to prevent lateral movement of the units during geosynthetic material tensioning.

7.3.3 The minimum width of the top layer of concrete units shall be 42 inch (1060 mm) wide and be fully supported by the bottom course. Smaller base widths are permissible if it can be demonstrated that the connection performance is the same as that for 42 inch wide test walls (see Section 7.2.3). The width of the top layer of concrete units in the direction perpendicular to the direction of tensioning shall extend at least 6 inches (150 mm) beyond each edge of the geosynthetic sample. Saw-cutting of the segmental units with a concrete/masonry saw is permissible provided that cut edges are located beyond the edge of the geosynthetic sample.

7.3.4 A minimum of one "as manufactured" running joint shall be located on the top or bottom layer of units at the center of pull of the geosynthetic sample. The running joints in the top row of units shall be staggered over the bottom course as recommended by the manufacturer.

7.4 Granular Infill

7.4.1 The granular infill selected for testing should meet the requirements of the retaining wall systems specification guidelines, or as agreed upon by parties involved. The fill should be compacted to the same density as in the field.

7.5 Surcharge Loading

7.5.1 The surcharge loading arrangement shall be selected to provide a uniform pressure distribution over the top layer of concrete block units. A rigid loading platen is required below the vertical piston/actuator. It must have sufficient area to cover the entire surface of the top layer of concrete units. One or more layers of stiff gum rubber mat placed between the rigid loading platen and concrete units is recommended to provide uniform pressure distribution. Alternatively, a pressurized air bag system may be used.

8. Test Procedure

8.1 Install and brace lower course of concrete segmental units. Place units with a running joint coincident with the center of pull of the geosynthetic reinforcement.

8.2 Place and compact granular infill (if required) to same density as in the field.

8.3 Center geosynthetic reinforcement with respect to center of horizontal tensioning piston/actuator. Locate reinforcement with respect to concrete keys and connectors as recommended by manufacturer. Record sample width, length and position on the concrete units.

8.4 Trim two samples of the same geosynthetic reinforcement to cover the interface between concrete units on either side of wide-width geosynthetic test specimen. These samples are required to ensure the top course of concrete units is level. Leave 0.5 inch (13 mm) between these samples and the edge of the wide-width test sample.

8.5 Place the top layer of concrete units over the geosynthetic sample ensuring the top layer is level.

8.6 Place and compact granular infill (if required) to same density as in the field. Ensure the top surface of the wall is level.

8.7 Place the surcharging arrangement and ensure there will be a uniform distribution of surcharge pressure over the top of the concrete units.

8.8 Position and secure vertical load frame. Position and secure the vertical loading actuator/piston over the center of the connection system.

8.9 Attach the tensile loading clamp to the geosynthetic sample leaving a minimum of 8 inches (200 mm) between the back of the concrete units and the loading clamp.

8.10 Attach two displacement recording devices to a bar clamp attached to the geosynthetic reinforcement immediately adjacent to the back of the concrete units. It is recommended the bar clamp be constructed from two light-weight aluminum angle bars that are lightly screw-clamped to the reinforcement and extend the full width of the geosynthetic sample. The displacement recording devices should be located equidistant from and on either side of the tensioning actuator. The devices should be approximately 10 to 12 inches, (250 to 300 mm) apart in order to calculate the average displacement of the geosynthetic sample during the test.

8.11 Apply a predetermined vertical load (surcharge pressure) to the top of the concrete units. Maintain this load at a constant value for the duration of testing.

8.12 Apply a tensile seating load to the geosynthetic reinforcement to remove slack from the sample. Zero load and displacement devices as required. The seating load shall not exceed 10 % of the peak connection strength or of 200 lb/ft (290 N/m), whichever is smaller.

8.13 Apply a constant rate of displacement of 10% tensile strain/minute at the loading clamp. The rate of displacement is calculated with respect to the initial free length of the reinforcement between the loading clamp and the point of attachment of the displacement measuring devices. Hence, an initial 10 inches (200 mm) free length of geosynthetic sample requires that the actuator move at a constant .08 in./min (20 mm/min).

8.14 Record tensile load, actuator displacement and geosynthetic reinforcement deformation at the back of the concrete units at regular intervals.

8.15 Continue the test until there is a sustained loss of tensile resistance recorded at the loading clamp due to failure of the reinforcement at or within the connection system and/or failure of the blocks. In some cases the failure will be defined as excessive deformation or slippage of the reinforcement in the connection without a sustained loss of tensile resistance. Failure or slippage of the geosynthetic reinforcement within the loading clamp constitutes an invalid test.

9. Number of Tests

9.1 The range of surcharge loads applied to a series of tests shall correspond to the range of surcharge loads anticipated for the connection system in the field.

9.2 The test procedure should be demonstrated to be repeatable by carrying out a minimum of three tests at one surcharge load level. The ultimate connection load from three nominally identical tests should not differ by more than ± 10% from the mean of the three tests.

10. Calculations

10.1 Plot the tensile load versus average geosynthetic reinforcement deformation recorded at the back of the concrete units as shown in Figure C.1-2.

10.1.1 Ultimate Connection Strength—Calculate the ultimate connection strength for each test using the Eq. C.1-1. This is the maximum force per unit width generated by the connection. Values are to be expressed in lb/ft (kN/m) using Eq. C.1-1 as follows:

[Eq. C.1-1]

$$T_{ultconn} = F_p / W_s$$

where:

$T_{ultconn}$ = Ultimate connection strength per width of geosynthetic sample, lb/ft (kN/m)
F_p = Ultimate (Peak) tensile connection load, lb (kN)
W_s = Width of geosynthetic specimen, ft (m)

10.1.2 Service State Connection Strength - Calculate the service state connection strength T_{cs} for each test using Eq. C.1-2:

[Eq. C.1-2]

$$T_{cs} = F_{ss} / W_s$$

where:

T_{cs} = Service State connection strength based upon a prescribed deformation criterion lb/ft (kN)
F_{ss} = Measured tensile connection load at prescribed deformation criterion, lb (kN)
W_s = Width of geosynthetic specimen, ft (m)

If the prescribed deformation criterion is not achieved before ultimate connection load is reached the service state connection load shall be taken as the peak load value.

11. Report

11.1 Indicate that the specimens were tested as directed in this Test Method or state any deviations from this method of test.

11.2 Describe in detail the segmental concrete units and mechanical connectors and the method of sampling used.

11.3 Describe the geosynthetic reinforcement and the method of sampling that was used.

11.4 For each test, show a plot of tensile load versus average geosynthetic reinforcement deformation recorded at the back of the concrete units, see Figure C.1-2.

11.5 Provide a summary table (see Table C.1-1) of ultimate and service state connection strengths at each surcharge load and the average of any repeat tests. On the same table, report the geosynthetic sample width for each test and the wide strip tensile strength of the geosynthetic material per ASTM D 4595.

11.6 Summarize the results of connection tests on a plot (see Figure C.1-3) of:

a) connection strength (based on ultimate (peak) load criterion) versus surcharge load

b) connection strength (based on deformation criterion) versus surcharge load.

The trend in the data may be approximated by one or more line segments using linear regression methods.

11.7 Report on the type of failure(s), its location and description.

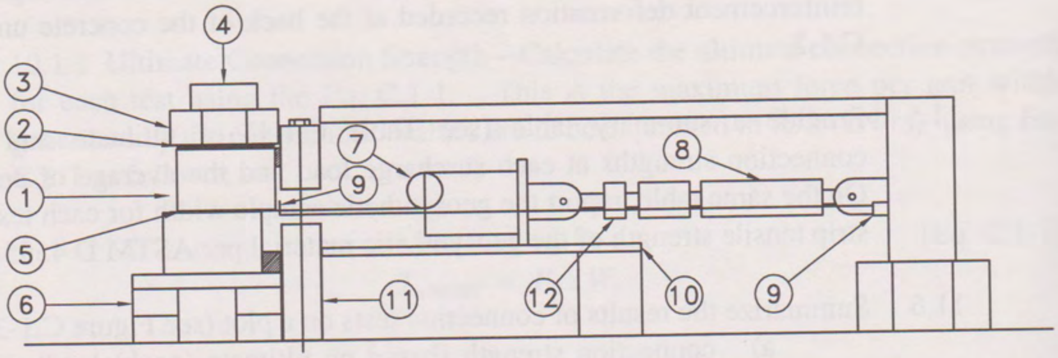
11.8 Provide a sketch and photograph (optional) of the test setup and stacking configuration used for the segmental units.

11.9 Provide a grain size distribution curve of the granular infill for placement in and between segmental concrete units (See Figure C.1-4).

12. Precision and Bias

12.1 Precision - The precision of this test method has not been established.

12.2 Bias - The true value of this test method can only be defined in terms of a specific test method. Within this limitation, the procedure described herein has no known bias.



1. STACKED SEGMENTAL CONCRETE UNITS
2. NORMAL LOAD DISTRIBUTION PAD
3. NORMAL SURCHARGE LOADING PLATE
4. NORMAL LOAD ACTUATOR/LOAD CELL
5. GEOSYNTHETIC REINFORCEMENT
6. PLATFORM
7. GEOSYNTHETIC CLAMPING DEVICE
8. HYDRAULIC ACTUATOR OR SCREW JACK
9. DEFORMATION MEASURING DEVICE (ATTACH TO GEOSYNTHETIC AT REAR OF BLOCK)
10. GUIDE RAIL
11. LOADING FRAME
12. LOAD CELL

FIGURE C.1-1: CONNECTION STRENGTH TEST SYSTEM

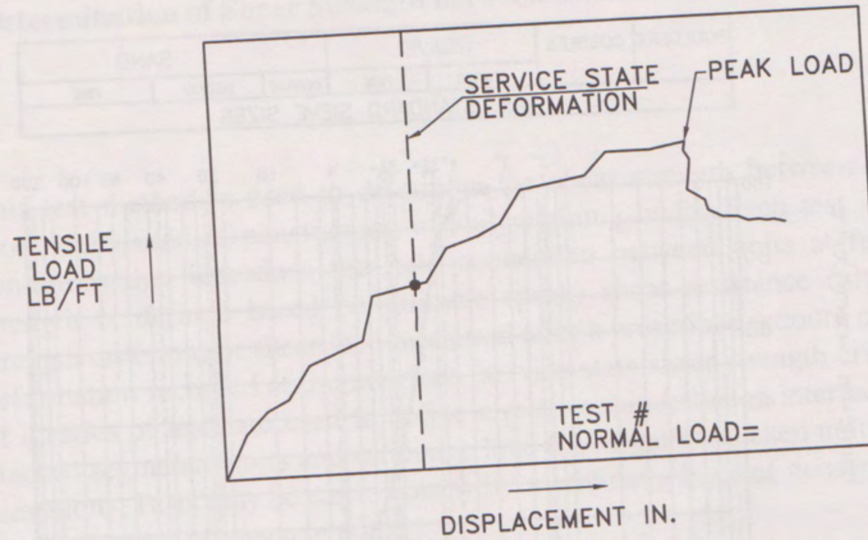


FIGURE C.1−2: TENSILE LOAD vs DISPLACEMENT

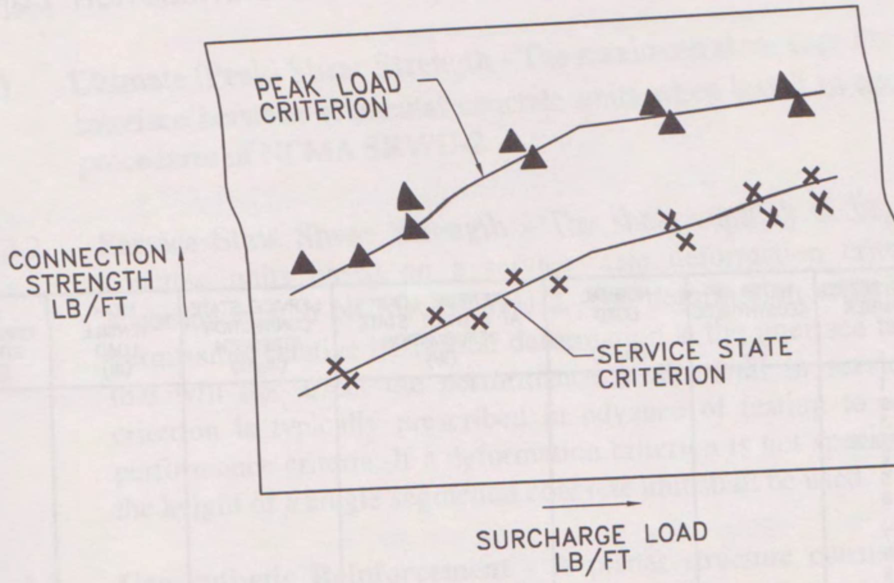


FIGURE C.1−3: CONNECTION STRENGTH vs SURCHARGE LOAD

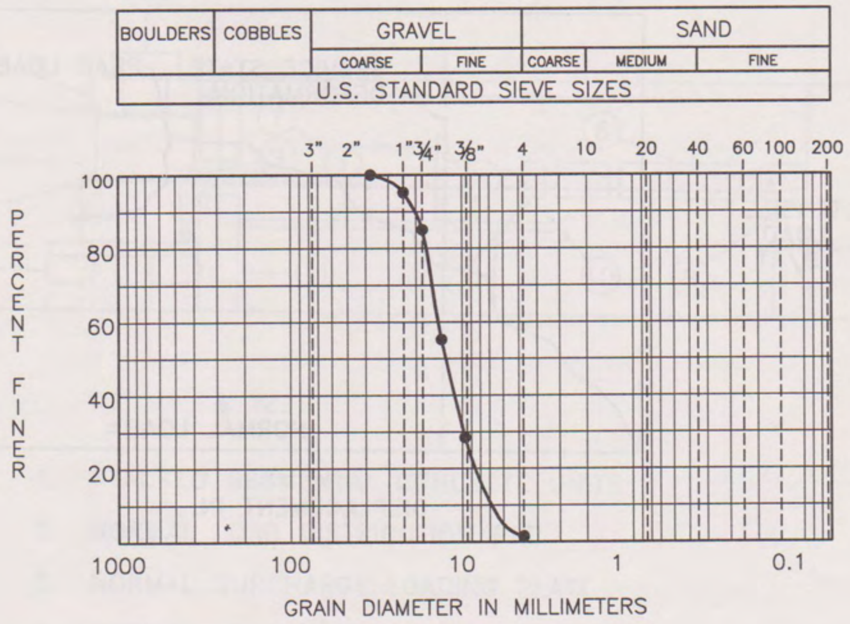


FIGURE C.1–4: GRAIN–SIZE DISTRIBUTION CURVE

TEST SERIES NUMBER	WIDTH OF GEOSYNTHETIC (ft)	NORMAL LOAD (lb/ft)	TENSILE LOAD AT SERVICE STATE DEFORMATION (lb)	SERVICE STATE CONNECTION STRENGTH (lb/ft)	PEAK TENSILE LOAD (lb)	PEAK CONNECTION STRENGTH (lb/ft)
1						
2						
3						
AVERAGE						
4						
5						
6						
7						
8						
9						

ULTIMATE TENSILE STRENGTH T_{indx} (ASTM D4595)= (lb/ft)

TABLE C.1–1: TEST RESULTS

C.2 NCMA Test Method SRWU-2

"Determination of Shear Strength between Segmental Concrete Units"

1. Scope

This test method is used to determine the shear strength between segmental concrete block units used in construction of soil retaining walls. Each test is carried out under conditions that reproduce the field connection between units at full-scale. The shear strength is defined based on ultimate (peak) shear resistance (ultimate [peak] shear strength criterion) or shear load measured after a prescribed amount of relative horizontal deformation recorded at the interface (service state shear strength criterion). The results of a series of tests are used to define a relationship between interface shear strength for stacked segmental units and surcharge load (or height of stacked units above the interface elevation). Tests may be carried out with or without a layer of geosynthetic reinforcement between layers of concrete units.

2. Referenced Documents

2.1 NCMA TEK 2-4

3. Definitions

3.1 **Ultimate (Peak) Shear Strength** - The maximum shear capacity developed at the interface between segmental concrete units when tested in accordance with the procedures of NCMA **SRWU-2**.

3.2 **Service State Shear Strength** - The shear capacity of the interface between concrete units based on a service state deformation criterion and tested in accordance with NCMA **SRWU-2**. The deformation criterion is the maximum permissible relative horizontal deformation at the interface between stacked units that will not affect the performance of the wall in service. The deformation criterion is typically prescribed in advance of testing to satisfy retaining wall performance criteria. If a deformation criterion is not specified, a value of 2% of the height of a single segmental concrete unit shall be used.

3.3 **Geosynthetic Reinforcement** - A planar structure consisting of a network of connected polymeric tensile elements used as a horizontal reinforcement material in geotechnical and civil engineering applications (geotextile or geogrid).

3.4 **Segmental Concrete Unit** - Modular concrete block manufactured specifically for mortarless retaining wall construction. The concrete units may be dry-cast machine molded or wet-cast and are made from portland cement, water, and mineral aggregates with or without the inclusion of other materials.

3.5 **Wall Facing** - Stacked segmental concrete block units placed in front of the backfill soil. The units are positively connected by a built-in concrete shear key or by interface friction developed between concrete block layers with or without the aid of mechanical connectors.

3.6 **Granular Infill** - Coarse grained soil aggregate used to fill the voids in and between segmental concrete units as recommended by the segmental concrete unit manufacturer. An example is a crushed stone conforming to AASHTO No. 57 gradation.

4. Summary of Test Method

In this test method, segmental units are stacked in two layers. The bottom layer is laterally restrained. The top layer is loaded vertically to a constant surcharge load and the interface is sheared at a constant rate of displacement until failure occurs. Failure of the system is defined as sustained loss of shear capacity or excessive relative shear deformation. Ultimate (peak) shear capacity and shear capacity after a prescribed deformation has been recorded at the interface are used to define shear strength based on ultimate (peak) load and service state deformation criteria, respectively. Shear loads and strengths are reported in lb/ft width of interface (or kN/m).

5. Significance

5.1 The shear strength between segmental concrete block units is used in design of reinforced and unreinforced soil retaining walls.

5.2 The results of a single test defines the relationship between interface shear strength and the applied normal surcharge load. The results of a series of tests can be used to establish the relationship between shear capacity and a range of normal loads (or variable height of wall above the interface).

5.3 The method of test is independent of connection type (mechanical and/or frictional).

5.4 This shear strength test is meant to be a performance test, therefore, it should be conducted using full scale system components.

6. Apparatus

6.1 The Testing System (See Figure C.2-1) - The test components to determine the shear strength between segmental concrete units are:

- a loading frame
- a shear loading plate and piston/actuator

- a restraining box/plate
- a load cell to measure the shearing force applied to the interface
- displacement measuring devices
- a surcharge loading device
- a load cell to measure surcharge load

6.2 Loading Frame - The loading frame must have sufficient capacity to act as a reaction to forces developed by the horizontal and vertical loading pistons/actuators.

6.3 Shear Loading Plate - The shear loading plate must be sufficiently rigid to apply a uniform force across the full width of the top course. For some segmental units it may be necessary to apply the load through a deformable material (such as a stiff foam rubber) which will conform to an irregular block surface, thereby allowing for a uniform load distribution.

6.4 Restraining Box/Plate - A rigid restraining box/plate is required to prevent horizontal movement of the lowermost concrete blocks during shear testing. The restraining box/plate area shall be of sufficient width and depth to accommodate the full scale "as manufactured" segmental concrete units being tested.

6.5 Shear Loading Assembly - The shear loading assembly will generally be a constant rate of extension screw jack or hydraulic actuator which can be deformation rate controlled. The loading equipment must have a capacity that is greater than the anticipated shearing loads. The orientation of the shearing force applied by the piston/actuator shall be horizontal and perpendicular to the back of segmental unit(s). The horizontal loading arrangement must not permit rotation of the top concrete unit during shear.

6.6 Load Cells - Calibrated load cells shall be used to measure the shear and normal loads applied during the test. The load cells should have a capacity that is greater than or equal to 100% of the anticipated maximum applied loads and be accurate within ± 0.5% of their full scale range.

6.7 Displacement Measuring Devices - Dial gages or electronic displacement devices shall be used to continuously monitor the horizontal displacement of the segmental concrete units. These devices shall have an accuracy of ± 0.005 inch (0.1 mm).

6.8 Surcharge Loading Assembly - A vertical surcharge is used to apply a normal stress across the stacked block interface. The load may be applied through a free moving, hanging load frame; a stationary vertical piston/actuator with steel rollers between plates to maintain constant normal stress; or an air pressure bag with steel rollers between loading plates.

7. Test Specimens

7.1 Segmental Concrete Units

7.1.1 Segmental concrete units must be full sized blocks and meet manufacturer's material and dimensional specifications.

7.1.2 The shear connection must be constructed with full-size segmental units selected from a standard production lot. Model or prototype units should not be used unless it can be demonstrated they are equivalent to production units.

7.1.3 For segmental units greater than 18 inches (450 mm) in width the top layer unit may be trimmed to a minimum test width of 18 inches (450mm) using a masonry/concrete saw. Bottom layer units must be placed with a running joint opposite the horizontal actuator and have a base layer width greater than the top layer.

7.1.4 Concrete segmental units may not be reused unless there is no cracking, abrasion or wearing of the concrete surfaces between tests or any other damage to the units.

7.2 Connections

7.2.1 Connection between the blocks must be made exactly as in the field. Mechanical connectors should consist of identical materials, size, and recommended spacing as suggested by the manufacturer.

7.3 Granular Infill

7.3.1 The granular infill selected for testing shall meet the requirements of the retaining wall system specification guidelines, or as agreed upon by parties involved. The fill shall be compacted to the same density as anticipated in field construction or as recommended by the manufacturer.

7.4 Geosynthetic Reinforcement

7.4.1 If the segmental concrete units are to be used for a geosynthetic reinforced soil wall, then a layer of a specified geosynthetic material shall be placed at the interface.

7.4.2 Specimen Width - The minimum width of geosynthetic test specimen shall be the full width of the test interface.

7.4.3 Specimen Length - The geosynthetic specimen shall have sufficient length to cover the interface surface as recommended by the manufacturer. The sample must be trimmed to provide sufficient anchorage at the actuator side of the test arrangement to prevent slippage of the front edge of the geosynthetic sample when the sample simulates a layer of reinforcement.

7.4.4 A virgin sample of geosynthetic material shall be used for each test.

7.5 Construction

7.5.1 The connection shall be constructed as in the field using the same geosynthetic reinforcement (if applicable), granular infill, concrete block units and connectors. The number, type and arrangement of connectors must be identical to that used in the field.

7.5.2 The layers shall be stacked such that the piston/actuator pushes against the back of the stacked units (i.e. the backfill side).

7.5.3 A single layer of segmental units shall be placed on a rigid base. A second layer comprising a single segmental unit shall be placed over the bottom layer with the geosynthetic (if applicable) located and placed between the layers as described in construction specifications. The bottom layer shall be rigidly braced to prevent horizontal movement.

7.6 Surcharge Loading

7.6.1 The surcharge loading arrangement shall be selected to provide a uniform pressure distribution over the top concrete block unit. The load may be applied through a free moving hanging, load frame; a stationary vertical piston/actuator with steel rollers between rigid plates; or an air pressure bag arrangement placed directly over the segmental unit. A stiff rubber mat or mortar may be required to ensure uniform pressure distribution to the top concrete unit when a roller plate assembly is used.

8. Test Procedure

8.1 Install and brace lower course of concrete segmental units. Place units with a running joint coincident with the axis of the horizontal actuator.

8.2 Place and compact granular infill (if required) to same density as in the field.

8.3 Center geosynthetic reinforcement with respect to center of horizontal piston/actuator (if required). Locate reinforcement with respect to concrete keys and connectors as recommended by manufacturer. Ensure the geosynthetic reinforcement is anchored on the piston/actuator side of the connection system to prevent slippage. Record sample width, length and position on the concrete units.

8.4 Place a single concrete unit over the bottom layer and center on running joint.

8.5 Place and compact granular infill (if required) to same density as in the field. Ensure the top concrete unit is level.

8.6 Place the surcharging/roller plate arrangement (or airbag arrangement) on the top concrete unit and ensure there will be a uniform distribution of surcharge pressure (see **Figure C.2-1**).

8.7 Position and secure the vertical load frame. Position and secure the vertical loading actuator/piston over the center of the connection system (if applicable).

8.8 Position the shear loading plate against the top course of test units. The shear load should be applied against the stacked units immediately above the shear interface to minimize moment loading.

8.9 Attach one displacement recording device at each front (or back) corner of the top concrete block.

8.10 Apply a predetermined vertical load (surcharge pressure) to the top of the concrete units. Maintain this load at a constant value for the duration of testing.

8.11 Apply a horizontal seating load to ensure that shear transfer keys, mechanical connectors or the like are snug. Zero load and displacement devices as required. The seating load should not exceed 10 % of the maximum shear strength for the test or 50 lb (220 N), whichever is less.

8.12 Apply shearing load at the constant rate of extension between the range of 0.01 to 0.03 in./min (0.25-0.75 mm/min).

8.14 Record measurements of shear load and displacement at regular intervals.

8.15 Record piston/actuator load, piston/actuator displacement, and horizontal deformation of the top concrete unit at regular intervals.

8.16 Continue the test until there is a sustained loss of shearing resistance due to failure of interface components and/or failure of the blocks. In some cases the failure will be defined as excessive deformation due to slippage of the reinforcement in the connection without a sustained loss of shear resistance.

9. Number of Tests

9.1 The range of surcharge loads applied to a series of tests shall correspond to the range of surcharge loads anticipated for the connection system in the field.

9.2 The test procedure should be demonstrated to be repeatable by carrying out a minimum of three tests at one surcharge load level. The peak connection load from three nominally identical tests should not differ by more than ± 10% from the mean of the three tests.

10. Calculations

10.1 Plot the shear force versus average horizontal deformation recorded of the top concrete unit (see example in **Figure C.2-2**).

10.2 Ultimate (Peak) Shear Strength - Calculate the ultimate (peak) shear strength S_p for each test using the Equation C.2-1. Values are to be expressed in lb/ft (kN/m) of width, using Equation C.2-1 as follows:

$$S_p = F_p / W_i \qquad \text{[Eq. C.2-1]}$$

where

S_p = Ultimate (Peak) shear strength per width of top block, lb/ft (kN/m)
F_p = Ultimate (Peak) shearing load, lb (kN)
W_i = Total width of top concrete unit over the interface surface, ft (m)

10.2 Service State Shear Strength - Calculate the service state shear strength S_{ss} for each test using Equation C.2-2:

$$S_{ss} = F_{ss} / W_i \qquad \text{[Eq. C.2-2]}$$

where:

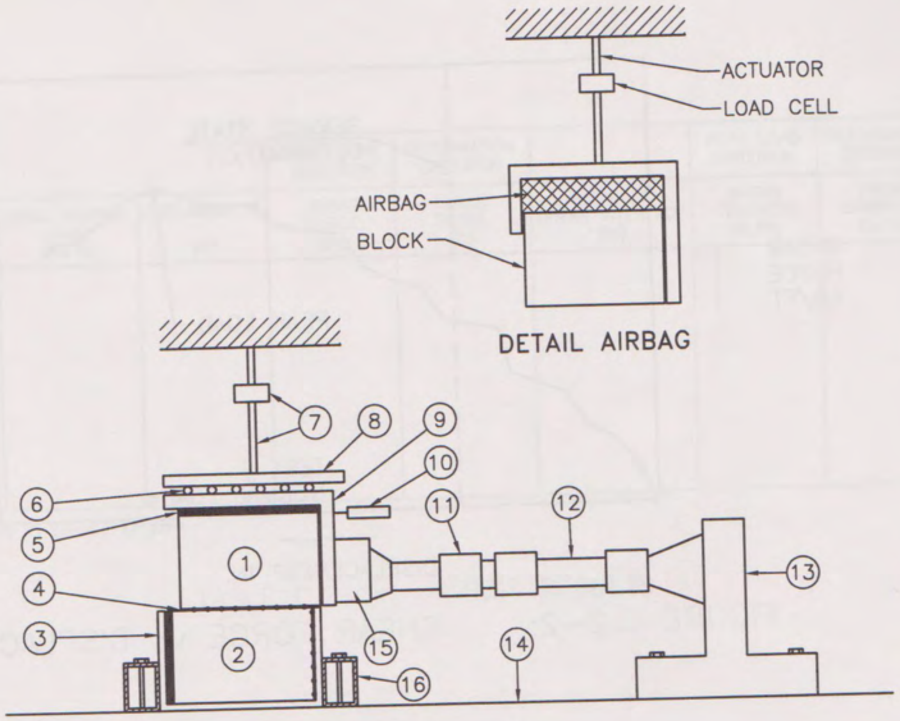
S_{ss} = Service state shear strength based on a prescribed deformation criterion, lb/ft (kN/m),
F_{ss} = Measured shear load at the prescribed deformation criterion, lb (kN)
W_i = Total width of top concrete unit over the interface surface, ft (m)

If the prescribed deformation criterion is not achieved before ultimate (peak) shear load is reached, the service state shear load shall be taken as the ultimate (peak) shear load value.

11. Report

11.1 Indicate the specimens were tested as directed in this Test Method or state any deviations from this method of test.

11.2 Describe in detail the segmental concrete units and mechanical connectors and the method of sampling used.



1. UPPER SEGMENTAL CONCRETE UNIT LAYER
2. LOWER SEGMENTAL CONCRETE UNIT LAYER
3. STEEL RESTRAINING BOX/PLATE
4. SEGMENTAL UNIT INTERFACE (ANCHORED GEOSYNTHETIC IF APPLICABLE)
5. NORMAL LOAD DISTRIBUTION LAYER (e.q. WOOD, STIFF RUBBER MAT, OR MORTAR LAYER)
6. ROLLERS (REQUIRED FOR STATIONARY SURCHARGE ACTUATOR)
7. NORMAL LOAD ACTUATOR/LOAD CELL
8. STEEL BEARING PLATE
9. STEEL LOADING PLATE
10. DISPLACEMENT MEASUREMENT DEVICE (2 MIN.)
11. LOAD CELL
12. HYDRAULIC ACTUATOR OR SCREW JACK
13. BEARING COLUMN
14. CONCRETE FLOOR
15. STEEL LOADING BULKHEAD
16. STRUCTURAL STEEL FLOOR BEAM

FIGURE C.2–1: INTERFACE SHEAR TEST SYSTEM

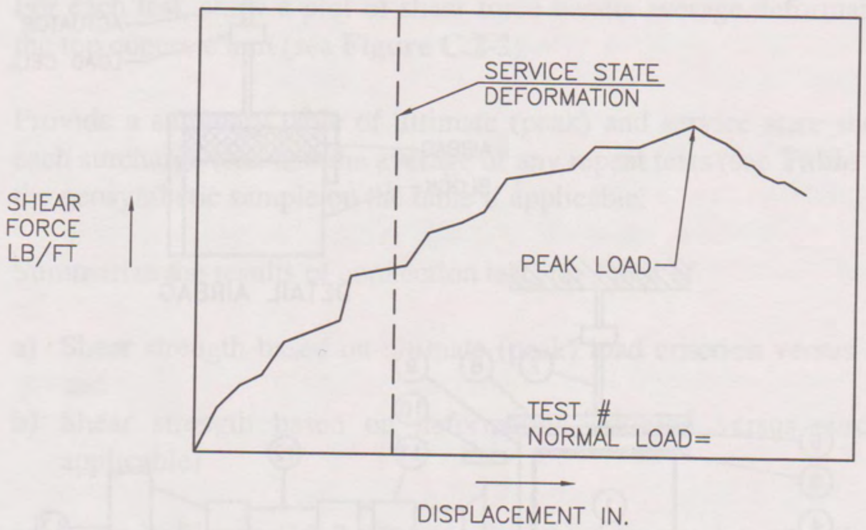


FIGURE C.2–2: SHEAR FORCE vs DISPLACEMENT

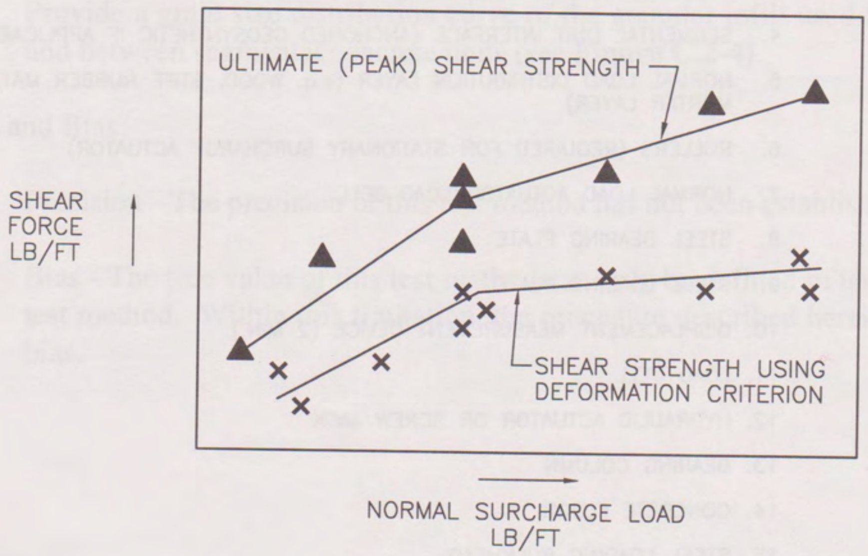


FIGURE C.2–3: SHEAR FORCE vs SURCHARGE LOAD

			PEAK LOAD CRITERION	DEFORMATION CRITERION		PEAK LOAD CRITERION	DEFORMATION CRITERION
TEST SERIES NUMBER	TOTAL NORMAL LOAD (lb/ft)	DISPLACEMENT (in)	SHEAR FORCE (lb)	SHEAR FORCE (lb)	WIDTH OF SHEAR INTERFACE (in)	SHEAR STRENGTH (lb/ft)	SHEAR STRENGTH (lb/ft)
1 2 3 AVERAGE 4 5 6 7 8 9							

GEOSYNTHETIC INTERFACE LAYER=

TABLE C.2–1: TEST RESULTS

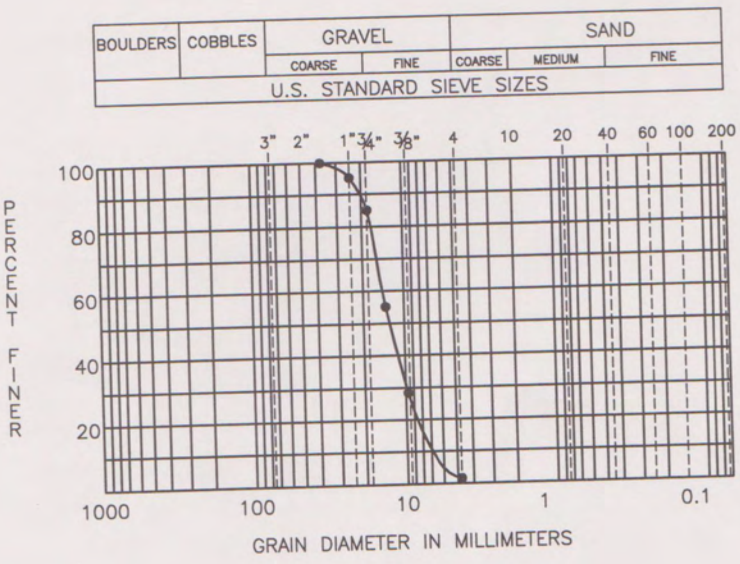


FIGURE C.2–4: GRAIN–SIZE DISTRIBUTION CURVE