ENGINEERING DESIGN MANUAL

FOR

STONETERRA™
WALL SYSTEM

1st EDITION

PREPARED BY

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LIST OF FIGURES

LIST OF TABLES

INTRODUCTION

The purpose of this manual is to provide general guidelines for designing StoneTerra^{TM} (Prefabricated Modular Block) mechanically stabilized earth (MSE) walls that are safe as well as cost-effective. The scope of this manual includes discussions on wall components, design theories, design methodology, special design considerations, and design examples.

Design methodology presented in this manual generally meets the requirements of The American Association of State Highway and Transportation Officials (AASHTO) retaining wall design guidelines presented in AASHTO LRFD Bridge Design Specifications, 4th Edition, 2007. Because AASHTO guidance documents do not apply in all situations and do not include comprehensive guidelines for designing StoneTerra^{TM} MSE walls, the National Concrete Masonry Association (NCMA) design manual and the Federal Highway Administration (FHWA) design guidelines were also reviewed for engineering guidance while developing this manual. The recently developed K-stiffness method was also reviewed for consistency and guidance. The author wishes to thank Stan Boyle and Kathleen Noonan for their review, and commentary on drafts of this document.

Design theory and methodology presented in this manual are based on conventional geotechnical and structural engineering principles and on experience with the design and construction of numerous MSE structures in the USA over the past few decades. The user of this manual should comprehend basic geotechnical and structural engineering theory and design concepts so that the presented design procedures can be properly understood. Provided the user of this manual understands the design procedures presented, the systematic step-by-step approach presented in this manual can be effectively followed.

This manual should be used only as a guidance document. All retaining wall designers should judge the appropriateness and accuracy of the design guidelines and equations presented in this manual for the intended application. The use of this manual to design walls constructed of other similar appearing blocks is not recommended. The authors of this manual and/or the block manufacturer are not responsible for the short-term or the long-term performance of a constructed wall designed per the guidelines presented in this manual. This manual provides only general information and the authors and block manufacturer do not assume any liability caused by its use. Even though significant efforts have been undertaken to ensure accuracy, any and all responsibility and liability lies with the designer. StoneTerra, Inc., the owner of this manual, and the authors disclaim any expressed or implied warranties of any kind and freedom from infringement of any patent, trademark, or copyright in regard to the information contained or referred to herein. Any and all the information contained in this manual should not be construed as providing a license under any patents. StoneTerra^{TM} is a registered trademark of StoneTerra, Inc.

StoneTerra™ MSE WALL COMPONENTS

A StoneTerra™ MSE wall consists of many components. These components are illustrated in Figure 1. Mechanical properties and engineering significance of these components are discussed below.

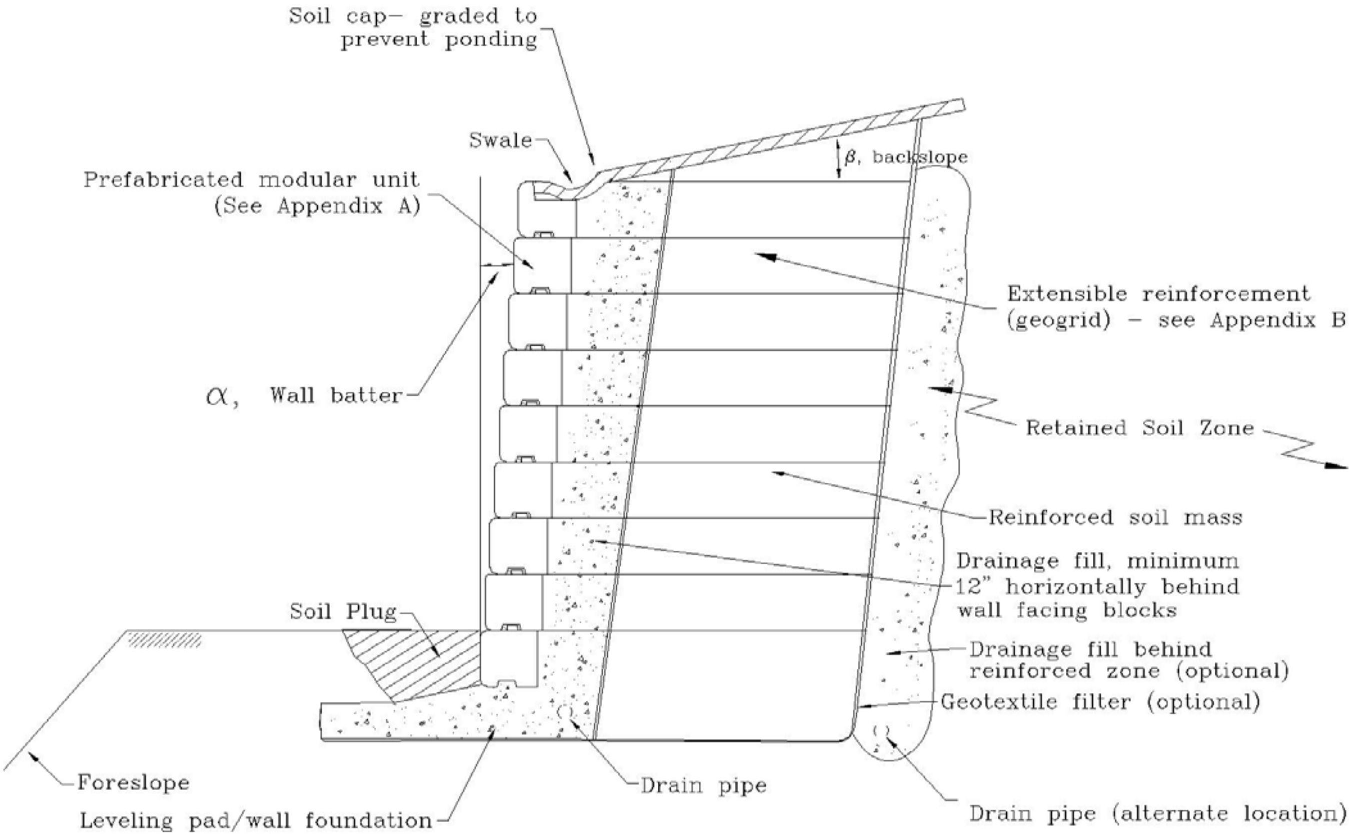


FIGURE 1 Typical MSE Wall Cross Section

PREFABRICATED MODULAR UNITS: Prefabricated modular units are dry-stacked concrete units that create the face of the MSE structure. There are various shapes and sizes of StoneTerra™ units that can be used to fit the required longitudinal wall profiles and architectural requirements. These shapes and sizes can be found in Appendix A. Also, a variety of fascia treatments are available. Colors are various shades of gray and can be modified by staining.

StoneTerra™ units are generally made of "surplus" or "return" concrete, thus making them one of the most environmentally friendly products in the retaining wall industry. Based on years of manufacturing experience in the Western USA and Canada, it is known that the compressive strength in these blocks constantly exceeds 2,200 pounds per square inch (psi), with significantly higher strengths regularly achieved. The compressive strength requirements of these blocks are governed by durability requirements of a particular application. The compressive strength and water absorption properties of the block can be modified for projects situated in colder climates or at high altitudes to provide adequate freeze-thaw resistance (durability) and resistance to de-icing salts on highways. In such cases, modifying the material properties of blocks can adequately address the freeze-thaw issue.

Due to the internal and external drainage around StoneTerra™ units, concrete durability is enhanced. Space between adjacent blocks and clearance of roughly 1/2 inch around keys provides positive drainage through the wall face. A drainage system placed immediately behind the wall also enhances drainage.

StoneTerra™ units are placed in a running bond so that they interlock at locking groove and shear key locations to provide structural integrity and wall batter. Wall batter is achieved by stacking blocks on a level base – an offset in the casting of the base and top of each block imports 1:24 batter to the face. It is possible to achieve additional batter using an inclined foundation. The center of gravity of wall and wall batter are sensitive physical dimensions of the wall structure with respect to the wall stability. StoneTerra™ units provide formwork for compacting soils behind the wall, facial stability, including erosion protection, and an architectural fascia for the structure.

The weight of the StoneTerra™ and shearing resistance due to locking grooves and shear keys developed at the top and bottom surface of each block provide resistance to destabilizing forces generated by retained soil mass and surcharge loading. Based on laboratory testing performed by StoneTerra, Inc., the friction angle for the inter-unit shear resistance (XY graph of Normal Force vs. Shear Force) is as high as 45 degrees, thus, providing excellent internal shear capacity for the wall facing. Also, due to the high inter-unit shear resistance, StoneTerra™ MSE walls do not shear during construction.

GEOSYNTHETIC REINFORCEMENT (GEOGRID): These are high tensile strength polymeric sheet materials. Geogrids are placed in horizontal layers to unify the mass of the composite MSE wall structure to increase the resistance of the wall to the destabilizing forces generated by the retained soils and surcharge loads.

To achieve a composite MSE wall structure, geogrids must possess adequate tensile strength, be placed in sufficient layers, and develop sufficient connection and anchorage capacity to hold the composite MSE structure together. The StoneTerra™ blocks may include an additional groove that allows for better connection of geogrid to the wall face. To estimate the strength, length, and number of layers of a particular geogrid required to form an adequately stable MSE wall structure, a limit equilibrium design procedure is widely used. Various design parameters used in the limit equilibrium design procedure are defined somewhat differently in various reference documents on MSE structures. The definitions used in this design manual are generally consistent with AASHTO reference documents.

DRAINAGE FILL OR DRAINAGE COMPOSITE: Drainage fill consists of free-draining, well-graded, coarse-grained aggregates placed immediately behind the blocks to reduce the hydrostatic pressures or seepage forces and to prevent clogging of aggregate drainage medium if a geotextile fabric is not used. Drainage composite consists of a geosynthetic drainage net covered on each side with a geotextile filter fabric. Drainage composite is sometimes used instead of drainage fill.

REINFORCED SOIL MASS: Compacted structural fill reinforced with layers of geogrids placed behind the wall facing blocks or drainage fill. According to AASHTO guidelines, reinforced soil mass should extend a distance behind the wall equal to a minimum of 70% of the height of the wall (0.7H).

RETAINED SOIL: Native soils or compacted structural fill situated immediately behind the reinforced soil mass. The primary function of the MSE wall is to retain this soil mass without failure. The properties of retained soil mass (unit weight, angle of internal friction, cohesion, etc.) have the greatest influence on the final design of the wall structure. The retaining wall designer's challenge is to fully understand soil properties, StoneTerra™ properties, geogrid properties, and drainage requirements to optimize the design.

LEVELING PAD / WALL FOUNDATION: Compacted and free draining crushed rock pad that distributes the weight of block wall over a wider area and provides a working surface during construction. The leveling pad provides a stiff, but flexible, layer to assist in stress distribution and attenuation of differential settlements. As an alternative, a thin concrete leveling course can be used, but crushed rock is generally preferred from a constructability standpoint.

FOUNDATION SUBGRADE: Competent native soil or compacted structural fill that supports the MSE wall structure including the reinforced soil mass. Competent subgrade implies that the allowable bearing capacity of foundation subgrade is equal to or greater than the bearing pressure exerted by the stacked blocks at the wall face and the MSE wall structure. A qualified geotechnical engineer must observe and approve the foundation subgrade to check that there will be adequate support for the MSE wall structure.

DRAIN PIPE: Perforated pipe with adequate flow capacity placed typically at the base of the wall to discharge collected water into a suitable receptacle by gravity flow. Location of discharge pipe behind the wall depends upon the drainage requirements of the wall structure and the design of drainage system.

DRAINAGE SWALE: A small depression adjacent to the top of wall to collect surface water run-off and discharge by gravity flow. Typically a maximum 2% gradient is used for a positive gravity flow. The drainage swale also prevents surface water from cascading over the top of wall.

GEOTEXTILE FILTER: A filter fabric (with adequate porosity) placed against the retained soil mass or between the drainage media and the retained soil mass to minimize clogging of drainage media. The properties of the filter should be specified by an experienced geotechnical engineer.

BACKSLOPE (β): Retained soil slope behind the wall measured from the horizontal plane. Increasing backslope increases lateral earth pressures on the wall. Overall (global) wall stability becomes more important for designs with steeper backslopes. (A qualified geotechnical engineer should perform analysis to check these failure modes.)

FORESLOPE / TOESLOPE: Downslope in front of the toe of wall. If foreslope is present, bearing capacity and overall (global) wall stability become critical for design. A qualified geotechnical engineer should perform analyses to check these failure modes.

SURCHARGE: Additional loads placed on the wall, usually in the form of a gravity load applied to the surface of the soil compressing the reinforced soil or the retained soil.

DESIGN THEORY

StoneTerraTM MSE walls are reinforced soil structures with a large width created by the blocks and the reinforced soil mass. The MSE wall structure uses its weight to resist destabilizing forces induced by the retained soil mass and surcharge loading. MSE walls can be built higher and support greater surcharge loads than conventional gravity walls. Design assumptions and theories used to analyze these structures are discussed below.

DESIGN ASSUMPTIONS

StoneTerraTM walls are analyzed using standard geotechnical and structural engineering methods for retaining walls with some assumptions that reflect the segmental characteristics of the walls. The following fundamental assumptions are made for analyzing these walls.

Modes of Wall Failure

StoneTerraTM MSE walls are assumed to fail in an external, internal, local, and/or global failure mode(s). External failure modes include base sliding, overturning, and bearing capacity failure. Internal failure modes include geogrid pullout, tensile overstress of geogrids, and internal sliding (movement between courses). Local failure modes include the instability of the facing connection between the block and geogrid and bulging of the blocks due to inadequate geogrid layer spacing. Global failure mode includes overall slope failure of the entire MSE wall structure including the retained soil. These failure modes are illustrated in Figure 2. The MSE wall must be designed to have adequate factors of safety against all failure modes.

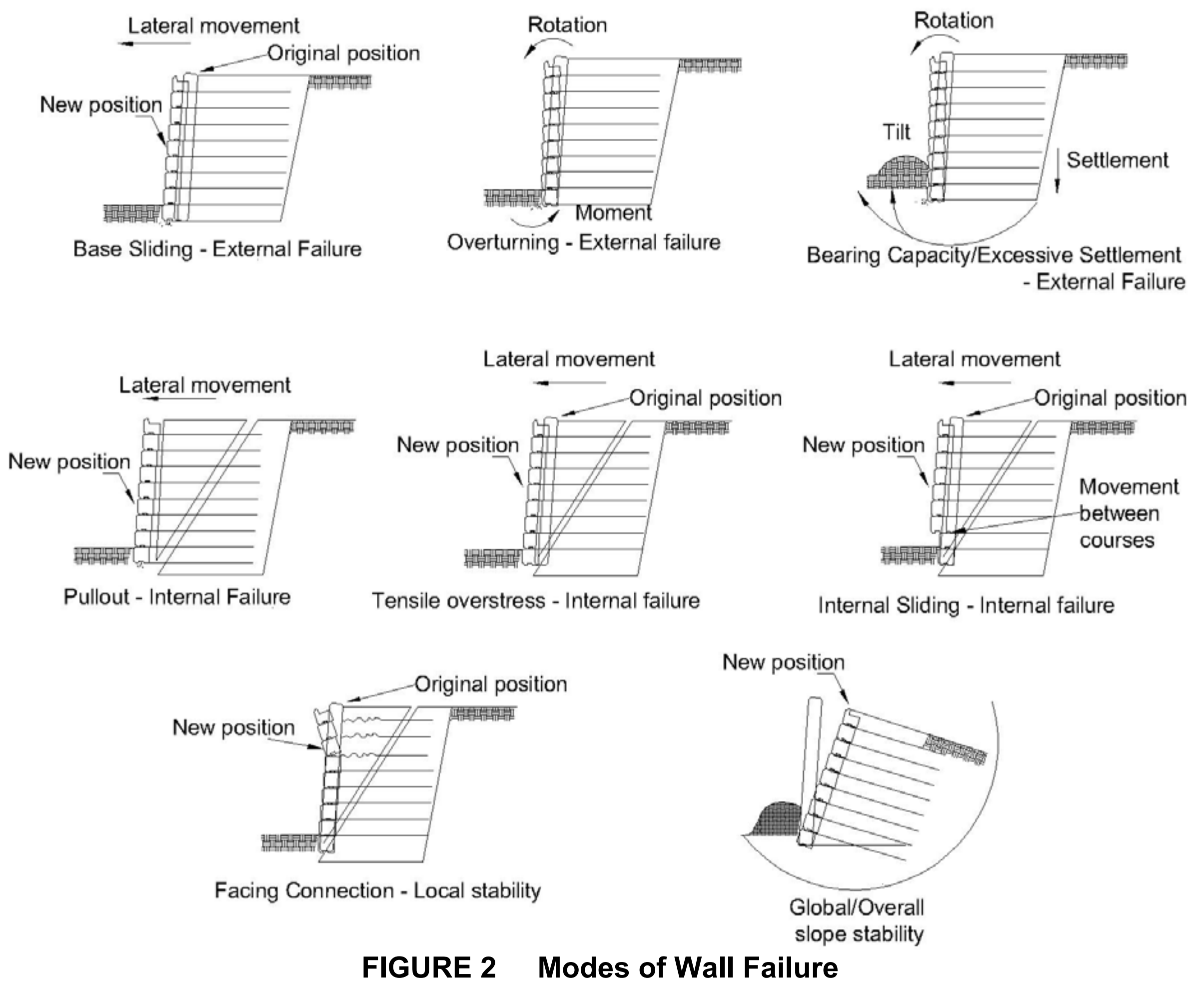


FIGURE 2 Modes of Wall Failure

Earth Pressure Theory

Coulomb's wedge theory using a plane surface of sliding is assumed to be applicable for estimating lateral earth pressures on the StoneTerra™ MSE wall structure. Coulomb's wedge theory approach is adopted primarily because it explicitly considers the influence of inclined wall facing (wall batter) and sloping backfill.

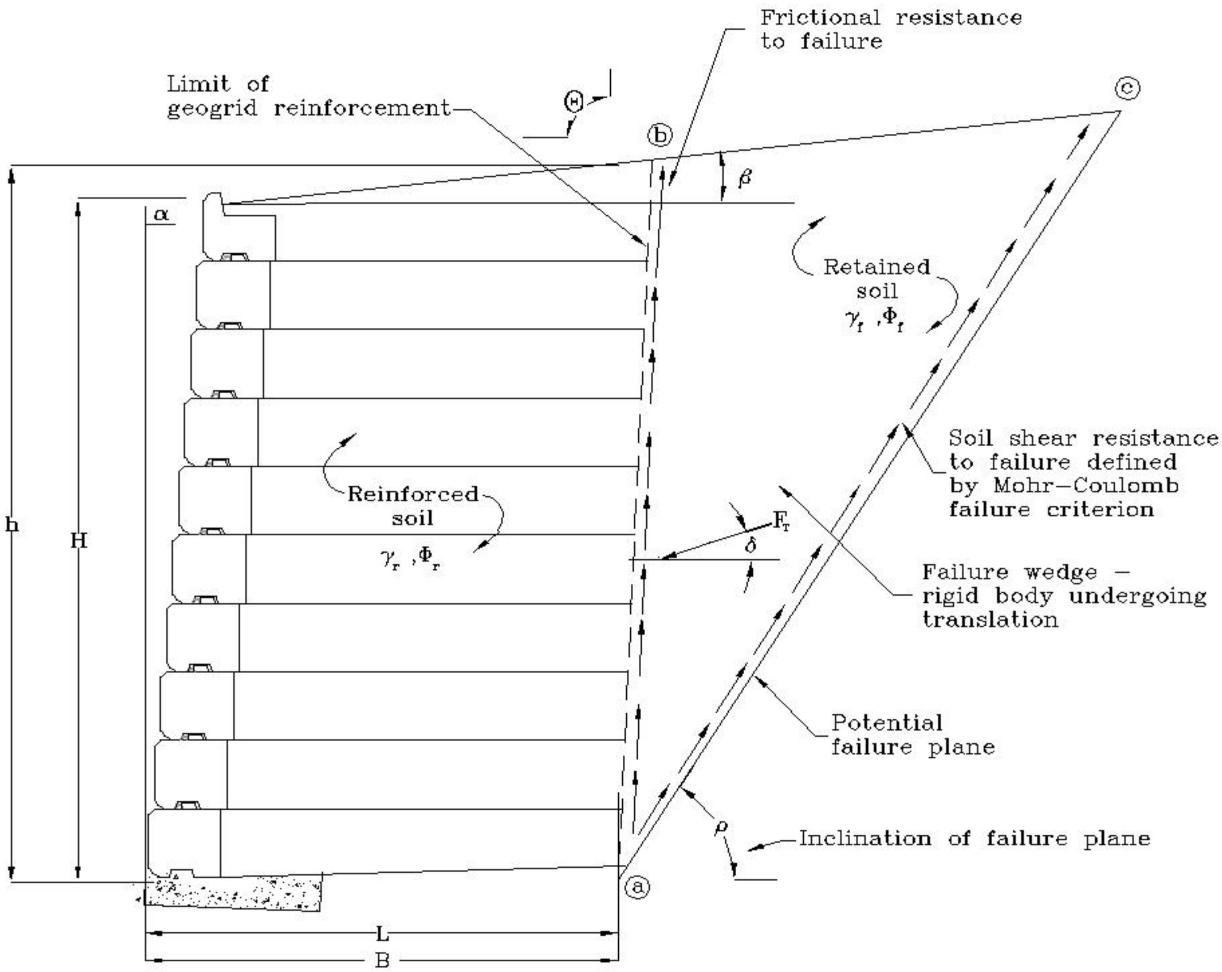


FIGURE 3 Coulomb's Wedge Theory

Coulomb theory assumes that the soil adjacent to the wall along a planar failure surface originating from the base of the reinforced soil at the limit of the geogrid into the retained soil mass is in a state of incipient failure (limiting equilibrium) as illustrated in Figure 3. The shear resistance at every location along the failure surface is described by a Mohr-Coulomb failure criterion using shear strength parameters of soil.

The state of limiting equilibrium for the wedge of soil "a, b, c" as shown in Figure 3 is reached due to yielding or outward movement of the retaining wall. The wedge of soil behind the wall is assumed to be in an "active state" exerting active earth pressures due to the outward movement of wall with respect to the retained soil mass.

The following equations and Figure 3 illustrate Coulomb's theory:

Active earth pressure coefficient (k_{af}):

$$k_{af} = \frac{\sin^2(\theta + \phi_f)}{\sin^2\theta \sin(\theta - \delta)\left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2}$$

Active Earth Pressure (F_T):

For cohesionless soils: $$F_T = \frac{k_{af}\gamma_f h^2}{2}$$

For cohesive soils*: $$F_T = \left(\frac{\gamma_f h^2}{2}\right)k_a - 2ch\sqrt{k_{af}} + \frac{2c^2}{\gamma_f}$$

*Use geotechnical engineering judgment and care when using this equation.

Where:

ϕ_f = internal friction angle of retained soil, degrees (°)
γ_f = wet unit weight of retained soil, pounds per cubic foot (pcf)
c = cohesion of retained soil, pounds per square foot (psf)
ϕ_r = internal friction angle of reinforced soil, degrees (°)
γ_r = wet unit weight of reinforced soil, pounds per cubic foot (pcf)
β = backslope angle, degrees (°)
θ = inclination of back of wall, degrees (°)
δ = wall backfill friction angle, degrees (°)
ρ = angle of inclination of failure plane degrees (°)
h = effective total height of soil at back of reinforced soil mass, feet (ft)

The soil mass in front of the wall near the toe of the wall is subjected to inward movement of the wall (lateral compression). Therefore, this soil mass is assumed to be in a "passive state" exerting passive earth pressures on the toe of the wall. Because it is impossible to ensure the presence of the soil mass in front of the toe of the wall during its entire design life, passive earth pressures are ignored in our design. Neglecting passive earth pressures introduces slight conservatism into the design process and also simplifies the design process.

Earth pressure theories such as the Coulomb theory and the Rankine theory have been used for estimating earth pressures on retaining structures. These theories are based on fundamentally different principles, and, therefore, will provide different results in both internal and external calculations. Rankine earth pressure theory assumes that friction does not occur between the wall structure and the retained soil mass. Rankine theory provides simpler formulae and failure plane definition. The designer should understand and be comfortable with an earth pressure theory and its limitations, and then follow the theory in its entirety.

Groundwater and Hydrostatic Pressures

Design equations presented in this manual are based on the assumption that the groundwater table is well below (at least 0.66H where H is the height of wall) the StoneTerra™ MSE wall pad elevation. Also, it is assumed that the drainage system placed behind the wall is functional and that there are no hydrostatic pressures acting on the back of the reinforced zone or internal to the reinforced zone face.

For other groundwater conditions, a brief discussion is provided in the "Special Design Considerations" section. In general, a geotechnical engineer should be consulted to provide design parameters under these conditions.

Width of Reinforced Soil Mass

Since the StoneTerra™ blocks are relatively thick, the width of the block may be added to the length of the geogrid behind the block for external stability analyses. The minimum length of the reinforcement and reinforced soil mass should the greater of 0.7H or 8 feet, where H is the design wall height. Shorter geogrid lengths are possible on non-AASHTO projects.

Dead and Live Load Surcharge

Live load surcharges are considered only as destabilizing forces. This approach is usually adopted to ensure a safe design and to avoid the use of live loads as contributing forces in the stability analyses because the presence of these loads cannot be assured for the entire life period of the structure. Dead loads are considered to contribute to both destabilizing and stabilizing forces, depending on their locations.

Zone of Maximum Stress for Internal Stability

The zone of maximum stress in the geogrid reinforcement is assumed to be located at the boundary between the active zone and the resistant zone as illustrated in Figure 4. The zone of maximum stress is assumed to begin at the back of the facing elements at the heel of the wall. The tensile load in geogrids is determined at the zone of maximum stress and at the connection with wall units.

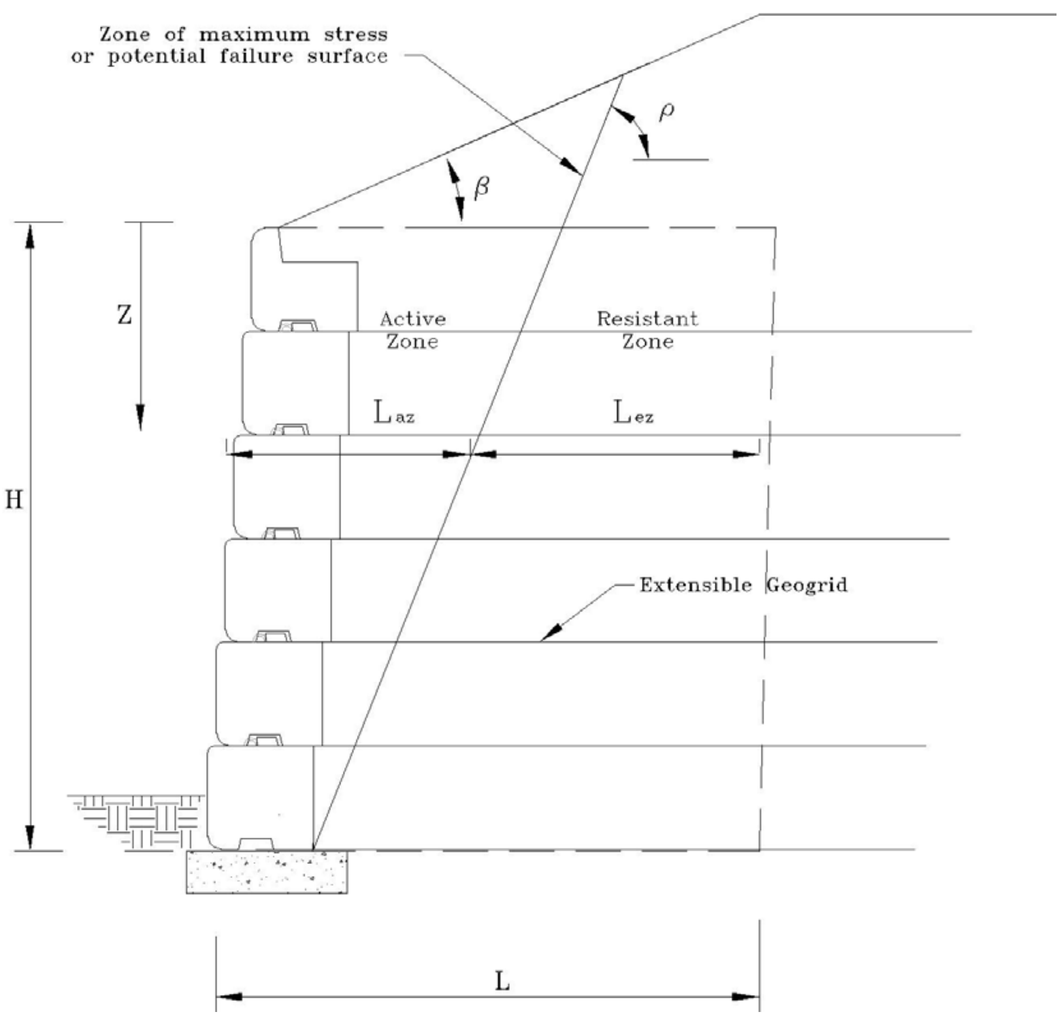


FIGURE 4 Zone of Maximum Stress

Connection Strength

The connection strength characteristics between geogrid and StoneTerra™ units are assumed to be represented by graphs presented in Appendix B for various geogrids. These graphs are derived from full-scale laboratory tests performed by industry professionals. Additional testing and/or engineering judgment may be required if a geogrid other than one of the tested products listed in Appendix B is used.

FACTOR OF SAFETY CRITERIA

The StoneTerra™ MSE wall should be dimensioned to ensure stability against possible failure modes discussed earlier using the Factors of Safety (FS) in Table 1.

Failure Mode	Minimum Required Factor of Safety
Sliding	1.5 75% of this value for Seismic Conditions
Overturning	2.0 - walls bearing on soil, 1.5 - walls bearing on rock, 75% of above values for Seismic Conditions
Bearing Capacity	3.0 2.0 - with the approval of a geotechnical engineer 1.5 - Seismic Conditions
Settlement	Differential Settlements: Critical Structures = 1 ft in 200 ft, Non-critical = 1 ft in 100 ft
Overall (Global) Stability	1.3 - Non critical Structures 1.5 - Critical Structures 1.1 - Seismic Conditions
Tensile Overstress	1.0
Facing shear capacity, connection, pullout	1.5 75% of this value for Seismic Conditions

TABLE 1. Minimum Required Factors of Safety

The designer should select an appropriate FS value based on the certainty with which design parameters and the consequences of failure are known. FS values given above are recommended minimum values for typical levels of uncertainty involved in the determination of structural dimensions of the wall and imposed loading.

The retaining wall designer should always review the local regulatory requirements for factors of safety prior to designing the wall. The wall should be designed, as a minimum, to satisfy the local regulatory requirements.

DESIGN METHODOLOGY

The components of the StoneTerra™ MSE wall and design assumptions, theory, and factor of safety criteria were previously discussed. In this section, the pieces are assembled in a design methodology which will determine the maximum height of a StoneTerra™ MSE wall for given soil and external loading conditions.

The design methodology consists of the following steps:

Determine:

1. The wall's structural dimensions,
2. Soil properties, and
3. Earth pressures and surcharge loading.

Evaluate:

1. External stability including the estimation of FSs against sliding, overturning, bearing capacity, and settlement failures.
2. Internal stability, and
3. Overall (global) stability.

Each step is discussed in detail below.

To simplify the design procedure, the retaining wall system is designed as a two-dimensional system (height and width) with length equal to one foot. Therefore, all the units for forces are given as pounds per foot (lbs/ft) and all the units for moments are given as pound-feet foot per foot (lb-ft/ft).

DETERMINATION OF STRUCTURAL DIMENSIONS

Structural dimensions of the StoneTerra™ MSE wall are shown in Figure 5. These dimensions include:

H	=	Design wall height (ft)
h	=	Effective total height of soil at back of reinforced soil mass (ft)
L	=	Length of geogrid beyond block face (ft)
B	=	Width of reinforced soil mass (ft)
β	=	Inclination of ground slope behind wall face blocks measured from horizontal plane (degrees, °)
α	=	Wall batter measured from vertical (degrees, °)
θ	=	Inclination of back of wall measured clockwise from horizontal (degrees, °)
	=	$90° + \alpha$ (assumes uniform reinforced length)

Wall Batter

Wall batter (α) is measured from the vertical to the face of the wall. A minimum batter of 1:24 or 2.4°is present due to the offset in the casting pattern of the base and top of the StoneTerra™ blocks. Generally, the wall should be designed as a vertical wall (i.e., ignoring the batter) to simplify the design and introduce conservatism. Based upon the experience of the authors and on past performance of these wall systems, the designer can economically design the wall as a vertical wall and recommend a wall batter of up to 10° (1H:6V) depending upon the desired wall appearance, right-of-way requirements, the degree of anticipated wall curvature, and the potential for excessive surcharge loading during construction.

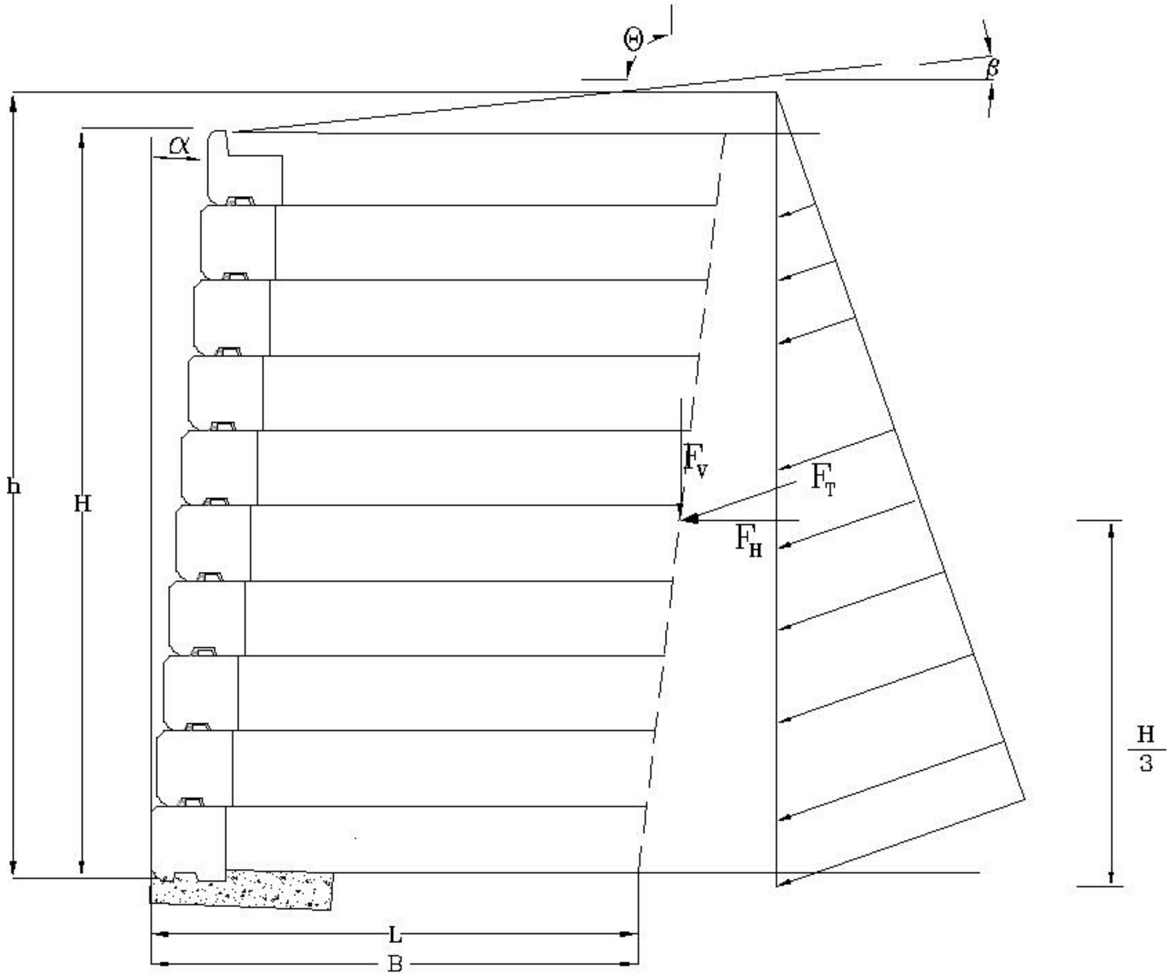


FIGURE 5 Wall Geometry and Earth Pressure Determination

Wall Heights

The MSE design wall height (H) is defined as the difference in elevation between the top of the wall at the wall face and the top of the leveling pad or foundation. The MSE effective total wall height of soil at the back of the reinforced soil mass (h) is the height of soil representing surcharge pressure.

$$h = H + L(\tan \beta)$$

Wall Embedment Depth

Wall embedment is generally required to provide frost protection and to provide safety against erosion and surficial instability near the MSE wall foundation. The following general guidelines may be used to estimate wall embedment depth:

- Minimum wall embedment depth = 12 inches or prevailing frost depth whichever is greater. An increase in embedment to 10 percent of the wall height is required if the wall height exceeds 8 to 10 feet.

- For critical structures, minimum required wall embedment is 24 inches. According to AASHTO, the minimum required embedment depth is 24 inches or frost depth, whichever is greater. It should be noted that the AASHTO recommendation is based on a rigid foundation system and may not be entirely applicable for flexible systems like the StoneTerra™ MSE wall which functions efficiently with less embedment depth. It is more important to have a qualified geotechnical engineer evaluate and inspect the foundation subgrade and confirm the embedment depth for bearing capacity and slope stability.

- Wall embedment depth for toe slope conditions:

Slope in Front of Wall	Minimum Embedment Depth (ft)
3H:1V	H/10
2H:1V	H/7
1.5H:1V	H/5
Fill slope	4 ft horizontal bench plus minimum depth using above criteria

  These values are rules of thumb that work under many circumstances. If toe slopes are necessary, a qualified geotechnical engineer should perform global wall (slope) stability and bearing capacity analyses to evaluate stability.

- The wall embedment depth should be increased by a geotechnical engineer when weak bearing soils, potential for scour of the wall toe, a submerged wall foundation, or potential for significant shrink/swell/frost of foundation soils exist.

It is important to note that the observation of foundation subgrade by a geotechnical engineer and subsequent determination or confirmation of wall embedment depth is critical for assuring external stability of the wall against bearing capacity and settlement modes of failure.

Soil Reinforcement Length

For StoneTerra™ concrete blocks, the minimum length of the reinforcement behind the wall face should be the greater of (0.7H – 2 feet) or 6 feet, where H is the design wall height. The reinforcement length should be uniform throughout the entire height of the structure unless unusually long reinforcement lengths are required near the top due to large surcharge loads.

Bottom Width of Wall

The bottom width of reinforced soil mass (B) for a StoneTerra™ MSE wall is the soil reinforcement length. The minimum bottom width of wall is,

B = 0.7H ft or 8 ft, whichever is greater

Sufficient wall bottom width is critical for providing external stability of the wall against sliding, overturning bearing capacity, and settlement modes of failure. The wall bottom width can be increased to reduce bearing pressures and to achieve the required minimum FS value. Generally, the bottom width can be increased by increasing the geogrid length. The bottom width includes the width of the block.

For cut slope conditions, it is important to optimally estimate bearing capacity of foundation soils and to reduce the bottom width of wall. The designer's goal should be to reduce the amount of excavation and backfilling required behind the wall face blocks for accommodating the wall footing. Note that compaction of backfill placed immediately behind the wall face blocks can be difficult to achieve.

DETERMINATION OF SOIL PROPERTIES

A StoneTerra™ wall is designed to resist a wedge of soil as illustrated in Figure 3. Properties of this soil mass including internal angle of friction (ϕ_f), wall friction angle (δ), base friction angle (δ_b), cohesion (c), and unit weight (γ_f) may be estimated using the project specific geotechnical information.

It is best to consult a qualified geotechnical engineer to obtain reasonable soil properties for wall design. Over reliance on a generalized geotechnical report may lead to a highly conservative or uneconomical wall design. In some cases, additional subsurface exploration and laboratory testing may become necessary to obtain reasonable soil properties for optimizing the wall design.

Soil strength parameters should be based on drained conditions for granular soils and fine-grained soils for long-term stability. In this manual, soil parameters are based on drained conditions, thus conforming to effective stress analysis.

The following guidelines may be used for preliminary wall design purposes only.

Soil Type	ϕ (degrees)	γ_f (pcf)	δ or δ_b (degrees)
Crushed rock	34	125-135	3/4ϕ
Sands	30-34	105-125	3/4ϕ
Silty sands	28-32	110-125	3/4ϕ
Sandy silts	28-30	110-125	1/2ϕ
Sandy clays	24-28	100-115	1/2ϕ

  Higher soil friction angle values may be used provided direct shear tests are performed and/or the design friction angle values are confirmed by a qualified geotechnical engineer.

- Base friction angle (δ_b) is based on the friction angle of base material, typically crushed rock. However, the base friction angle is equal to or less than the wall backfill friction angle ($\delta_b \leq \delta$). In any case, considerable geotechnical engineering judgment is necessary to choose optimal friction angle design values.

- A qualified geotechnical engineer must be contacted and additional laboratory testing may be necessary if highly plastic clays, weathered rock formations, glacial till, or very stiff residual soils are present. Soil cohesion must be considered in wall design if these soil conditions exist. Otherwise, unreasonable soil properties will result in an uneconomical wall design.

- Additional subsurface exploration and laboratory testing should be performed for taller walls and difficult soil conditions such as layered soils.

- Unless the designer has a comprehensive understanding of geotechnical principles, conservative soil parameters should be used.

DETERMINATION OF EARTH PRESSURES

Earth pressures are estimated using Coulomb's wedge theory as illustrated previously in Figure 3. Coulomb's wedge theory considers the wall friction angle (δ). Therefore, 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 calculations, it is necessary to have $\delta > \alpha$.

The following equations may be used to calculate resultant of active earth pressures.

$$k_{af} \quad = \quad \frac{\sin^2(\theta + \phi_f)}{\sin^2\theta \sin(\theta - \delta)\left(1 + \sqrt{\dfrac{\sin(\phi_f + \delta)\sin(\phi_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right)^2}$$

For horizontal backslopes, $\beta = 0$.
For broken backslopes, $\beta = I$ as shown in Figure 6.

Active Earth Pressure (F_T):

For cohesionless soils: $$F_T = \frac{k_{af}\gamma_f h^2}{2}$$

For cohesive soils*: $$F_T = \left(\frac{\gamma_f h^2}{2}\right)k_a - 2ch\sqrt{k_{af}} + \frac{2c^2}{\gamma_f}$$

*Use geotechnical engineering judgment and extreme care when using this equation

F_H = Horizontal component of F_T = $F_T \cos(90^o - \phi_f + \delta)$
F_V = Vertical component of F_T = $F_T \sin(90^o - \phi_f + \delta)$
δ = ½ ϕ_f to ¾ ϕ_f (typical)

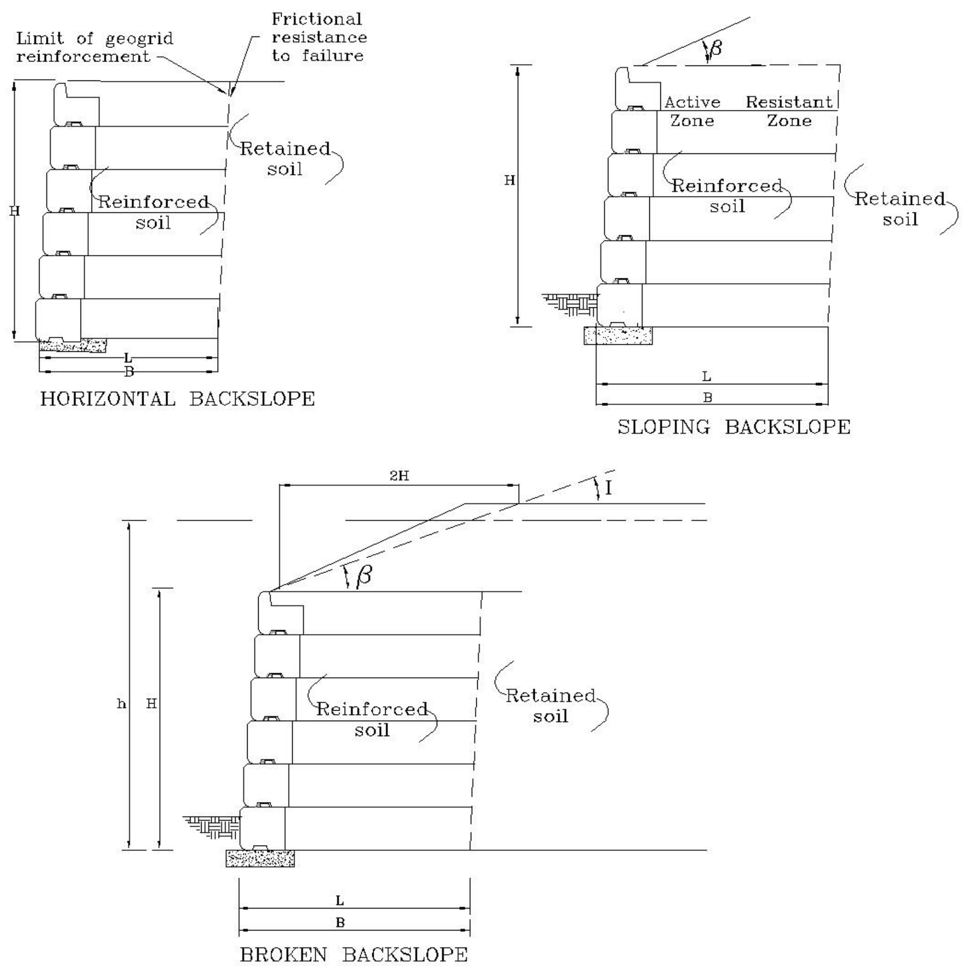


FIGURE 6 Backslope Configurations

The other major theory for estimating earth pressure is the Rankine earth pressure theory. Basically, both theories (Coulomb and Rankine) estimate earth pressures by modeling the weight of the sheared mass sliding along a theoretical plane of failure. The Coulomb equation considers wall friction angle (δ). Also, Coulomb's equation allows for the consideration of backslope or sloping backfill. Rankine equation does not consider wall friction angle or sloping backfill. Thus, for a vertical wall (no wall batter) with a flat backslope, both equations should provide similar earth pressure values.

It is important to consult with a qualified geotechnical engineer when estimating earth pressure values for cohesive soils. For broken backslope conditions, either an equivalent infinite backslope may be estimated or a 'trial wedge' method may be used for estimating lateral earth pressures as confirmed by a qualified geotechnical engineer. The choice of the earth pressure theory is best left to the designer. Whatever theory is used, it should follow through the entire design process.

DETERMINATION OF SURCHARGE LOADING

An external load applied to the retained soil mass behind the wall is called a 'surcharge load.' MSE walls may be subjected to a variety of loads, including concentrated vertical or horizontal surcharge loads. The impact of a surcharge load on the wall's stability depends upon its magnitude and its location from the face of the wall. The closer the location of the surcharge load to the wall, (i.e., back of the reinforced zone) the worse the impact on wall stability. If the surcharge load is located beyond a certain distance from the wall, then its impact is insignificant. This distance is referred to as the extent of active zone (LP). LP can be calculated as shown in the following equation:

LP = Extent of active zone (feet) = $B + (H)\tan\left(45^o - \frac{\phi_f}{2}\right)$

In general, if the retained soil mass is reasonably stable, then surcharge loads located beyond the distance equal to the design height of wall (H) would not significantly impact the wall stability. Thus, LP can also be roughly estimated as equal to H, the design wall height (LP≈H). Refer to "Applicability Conditions for Surcharge Loading" for additional information.

Strip loads, point loads, and/or heavy loading due to equipment, railroads, footings, closely spaced tiers, etc., are discussed within the scope of this manual. In general, a Boussinesq stress distribution is used for estimating this surcharge loading. Due to the complexity involved in the determination of surcharge due to strip and point loading, it is advisable to contact a qualified geotechnical engineer to confirm the calculated values per the procedures given above and to obtain optimal and/or reasonable surcharge load values.

Concentrated Vertical Surcharge Loads

MSE walls are routinely subjected to the following concentrated vertical surcharge loads:

- Uniform vehicular traffic live load (q_1, psf)
- Uniform dead load from traffic barrier or slab (q_2 , psf)
- Adjacent strip footing or traffic barrier load in terms of applied bearing pressure (Q_1 , lbs/ft)
- Isolated column/footing load in terms of applied bearing pressure (Q_2, lbs/ft)
- Concentrated column/point load (Q_3, lbs)

The following typical live load surcharges (q_1) may be used in preliminary wall design:

- Light traffic, light storage, automobile parking – 50 psf.
- Light-traffic, commercial automobile parking lots – 100 psf.
- Highway traffic loading, heavy truck traffic – 250 psf.

The effect of a vertical surcharge load on the wall stability can be evaluated by determining how the increased stresses within the retained soil mass vary with depth. The concentrated surcharge loads can be generally incorporated into the design by using a simplified uniform vertical distribution of 2V:1H to determine the vertical component of stress with depth. Vertical surcharge loads can be combined with earth pressures using the principle of superposition so that the wall's stability can be evaluated under the influence of surcharge loading as shown in Figures 7 and 9.

The types of surcharge loadings are defined as follows:

Uniform Loads

q_1	=	Uniform vehicular traffic live load (psf)
q_2	=	Uniform dead load (psf)

Strip Footing Load

Q_1	=	Applied bearing pressure for strip surcharge footing load (lbs/ft)
x_1	=	Horizontal distance from wall face to Q_1, (ft)
B_{r1}	=	Footing width for Q_1, (ft)
P_{v1}	=	Concentrated strip footing load (lbs) = $(Q_1)(B_{r1})$

Isolated Footing Load

Q_2	=	Applied bearing pressure for isolated surcharge footing load (psf)
x_2	=	Horizontal distance from wall face to Q_2 (ft)
B_{r2}	=	Footing width for Q_2 (ft)
L_{r2}	=	Footing length for Q_2 (ft)
P_{v2}	=	Concentrated isolated footing/column load (lbs) = $(Q_2)(B_{r2})(L_{r2})$

Point Load

Q_3	=	Load (lbs)
x_3	=	Horizontal distance from wall face to Q_3 (ft)
P_{v3}	=	Concentrated point load (lbs) = Q_3

Surcharge loading calculations should be performed as follows:

F_{q1} = Calculation of lateral earth pressure force due to q_1 (lbs/ft) = $q_1(H)(k_{af})$

F_{q2} = Calculation of lateral earth pressure force due to q_2 (lbs/ft) = $q_2(H)(k_{af})$

F_{pv1} = Resultant active lateral earth pressure force due to P_{v1} (lbs/ft)

$$= 0.5k_{af}\left(H\cos\alpha - 2(x_1 - B)\right)\left[\frac{P_{v1}}{B_{r1} + 2(x_1 - B)} + \frac{P_{v1}}{\left(x_1 - B + B_{r1} + \frac{H\cos\alpha}{2}\right)}\right]$$

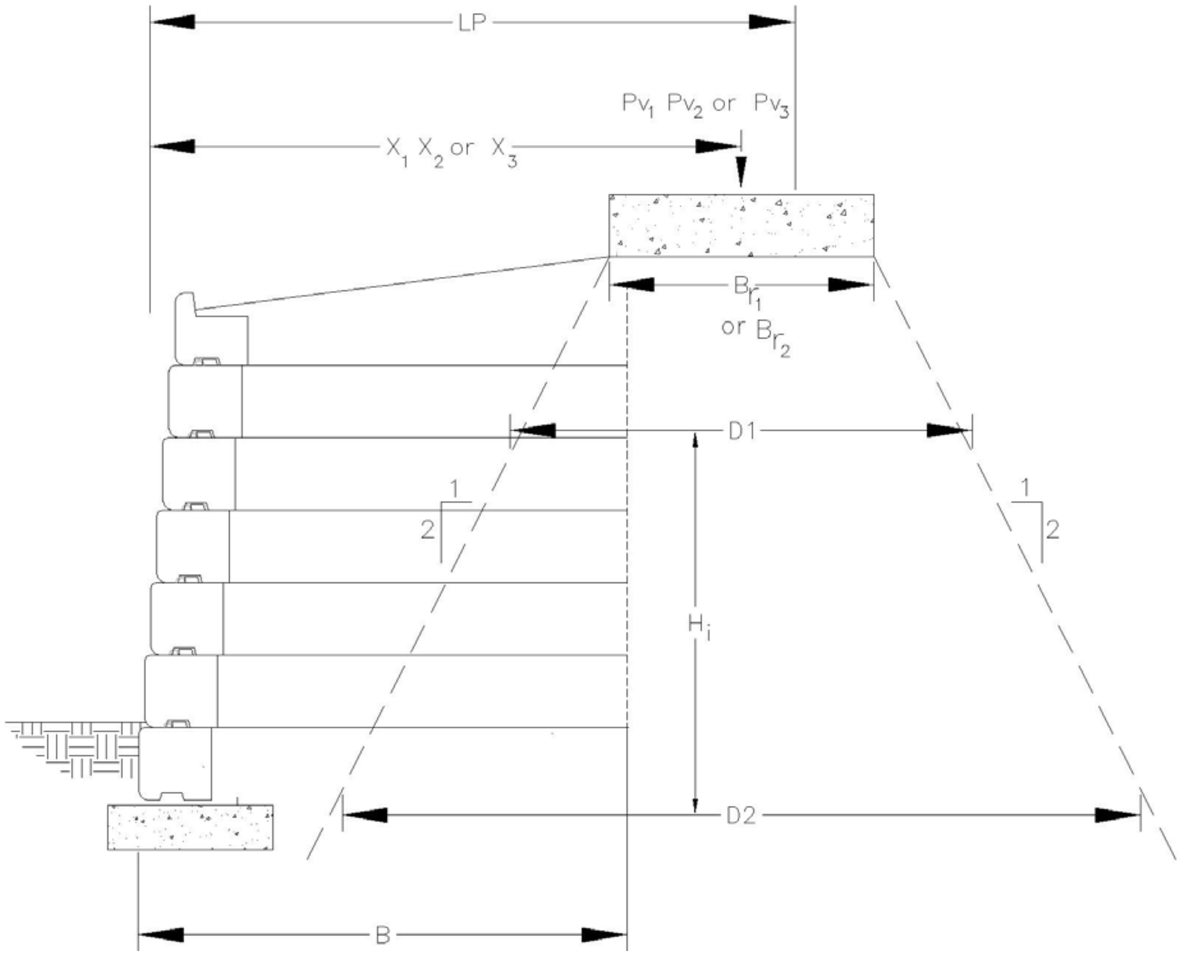


FIGURE 7 External Stability - Vertical Load Stress Distribution

h_{pv1} = Distance to F_{pv1} from bottom of wall (ft)

$$= \left(\frac{H\cos\alpha - 2(x_1 - B)}{3}\right)\left[\frac{2\left(\frac{P_{v1}}{\left(B_{r1}+2(x_1-B)\right)}\right)+\frac{P_{v1}}{\left(x_1 - B + B_{r1} + \frac{H\cos\alpha}{2}\right)}}{\left(\frac{P_{v1}}{B_{r1}+2(x_1-B)}\right)+\left(\frac{P_{v1}}{x_1 - B + B_{r1} + \frac{H\cos\alpha}{2}}\right)}\right]$$

F_{pv2} = Resultant active lateral earth pressure force due to P_{v2} (lbs/ft)

$$= 0.5k_{af}\left(H\cos\alpha - 2(x_2 - B)\right) *$$

$$\left[\frac{P_{v2}}{\left(B_{r2}+2(x_2-B)(L_{r2}+2(x_2-B)\right)}+\frac{P_{v2}}{\left(x_2 - B + B_{r2} + \frac{H\cos\alpha}{2}\left(L_{r2}+H\cos\alpha\right)\right)}\right]$$

h_{pv2} = Distance to F_{pv2} from bottom of wall (ft)

$$= \frac{H\cos\alpha(x_2 - B)}{3} *$$

$$\left[\frac{2\left(\frac{P_{v2}}{(B_{r2}+2(x_2-B)(L_{r2}+2(x_2-B))}\right)+\frac{P_{v2}}{\left(x_2-B+B_{r2}+\frac{H\cos\alpha}{2}(L_{r2}+H\cos\alpha)\right)}}{\left(\frac{P_{v2}}{((B_{r2}+2(x_2-B)(L_{r2}+2(x_2-B))}\right)+\frac{P_{v2}}{\left(x_2-B+B_{r2}+\frac{H\cos\alpha}{2}(L_{r2}+H\cos\alpha)\right)}}\right]$$

F_{pv3} = Resultant active lateral earth pressure force due to P_{v3} (lbs/ft)

$$= 0.5k_{af}(H\cos\alpha - 2(x_3 - B))\left[\frac{P_{v3}}{(2(x_3-B))^2}+\frac{P_{v3}}{\left(x_3-B+\frac{H\cos\alpha}{2}\right)^2}\right]$$

h_{pv3} = Distance to F_{pv3} from bottom of wall (ft)

$$= \frac{(H_w\cos\alpha - 2(x_3 - B))}{3}\left[\frac{2\left(\frac{P_{v3}}{(2(x_3-B))^2}\right)+\frac{P_{v3}}{\left(x_3-B+\frac{H\cos\alpha}{2}\right)^2}}{\left(\frac{P_{v2}}{(2(x_3-B))^2}\right)+\frac{P_{v3}}{\left(x_3-B+\frac{H\cos\alpha}{2}\right)^2}}\right]$$

Concentrated Horizontal Surcharge Loads

MSE walls may be subjected to concentrated horizontal or lateral loads. These surcharge loads can be combined with earth pressures using the principle of superposition so that the wall stability can be evaluated under the influence of horizontal surcharge loading. These concentrated surcharge loads should be generally incorporated into the design by using methodology illustrated in Figures 8 and 9. If the footing is located outside the active zone (i.e., x_{h1}, x_{h2}, x_{h3} > LP), the footing load is not considered (i.e., P_{h1}, P_{h2}, P_{h3} = 0).

Surcharge loading calculations should be performed as follows. Up to three horizontal surcharge loads (P_{h1}, P_{h2}, P_{h3}) are considered in the following example calculations:

Horizontal Load 1

P_{h1} = Horizontal surcharge load (lbs/ft)
x_{h1} = Horizontal distance from wall face to P_{h1}(ft)
B_{rh1} = Footing width for P_{h1}(ft)
e_1 = Load eccentricity for P_{h1}(ft)
L_1 = Stress distribution parameter for estimating additional stresses due to P_{h1} (ft)

$$= (x_{h1} + B_{rh1} - 2e_1)\left(\tan\left(45 + \frac{\phi_r}{2}\right)\right)$$

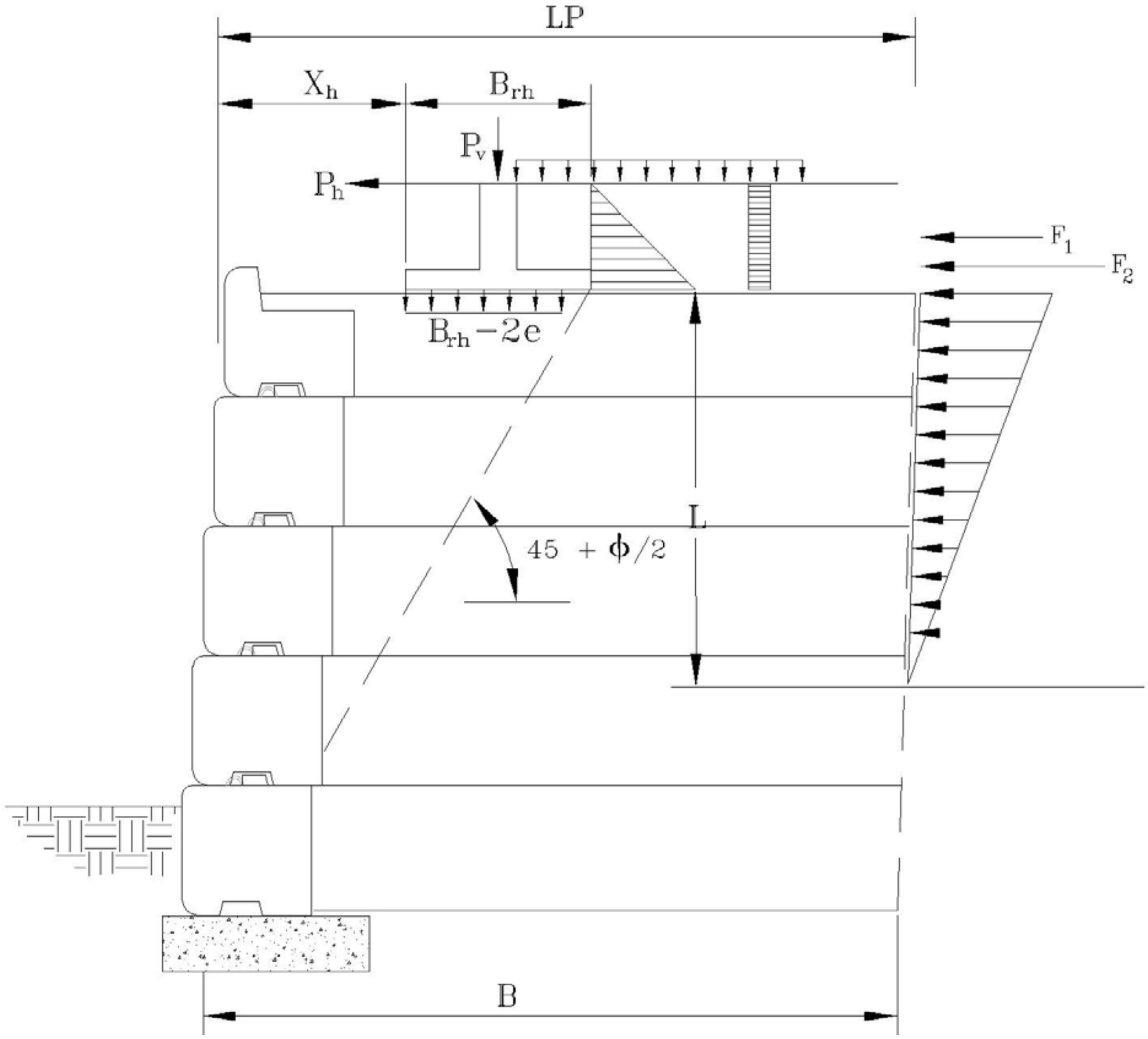


FIGURE 8 External Stability - Horizontal Load Stress Distribution

<u>Horizontal Load 2</u>

P_{h2} = Horizontal load (ft)
x_{h2} = Horizontal distance from face wall to P_{h2} (ft)
B_{rh2} = Footing width for P_{h2} (ft)
e_2 = Load eccentricity for P_{h2} (ft)

L_2 = Stress distribution parameter for estimating additional stresses due to P_{h2} (ft)

$$= (x_{h2} + B_{rh2} - 2e_2)\left(\tan\left(45 + \frac{\phi_r}{2}\right)\right)$$

<u>Horizontal Load 3</u>

P_{h3} = Horizontal load (lbs/ft)
x_{h3} = Horizontal distance from wall face to P_{h3} (ft)
B_{rh3} = Footing width for P_{h3} (ft)
e_3 = Load eccentricity for P_{h3} (ft)

L_3 = Stress distribution parameter for estimating additional stresses due to P_{h3} (ft)

$$= (x_{h3} + B_{rh3} - 2e_3)\left(\tan\left(45 + \frac{\phi_r}{2}\right)\right)$$

If L_1, L_2, $L_3 \leq 0$, then P_{h1}, P_{h2}, P_{h3} are not included in overturning moment calculations.

Hydrostatic Loading Conditions

One of the design assumptions behind design methodology discussed is that the groundwater level is situated at least $0.66H_e$ feet below the bottom of wall. If groundwater level is present at or above the leveling pad elevation, it may generate a significant loss in available soil shear strength leading to instability. Therefore, when such groundwater conditions exist, effective stress soil parameters including buoyant unit weight may be used for foundation soils, drainage soils, and/or retained soils depending upon the groundwater level and the recommended drainage system.

In general, the following guidelines should be followed for estimating surcharging loading due to groundwater.

h_w = Estimated differential head or height of water level above bottom of wall (ft). Use 3.0 ft for walls along rivers & canals or flowing water bodies, or the maximum water elevation, whichever to be greater.

γ_w = Unit weight of water (pcf)

F_w = Estimated horizontal hydrostatic load (lbs/ft) = $0.5\ (\gamma_w)\ H_w^2$

Moment Arm = Hydrostatic load acts at $\frac{h_w}{3}$ ft above the wall base.

Applicability Conditions for Surcharge Loading

Condition I:
If x_1, x_2, x_3 > LP, surcharge loads are considered to be zero.

Condition II:
If B < x_1, x_2, x_3 < LP, consider F_{pv1}, F_{pv2}, F_{pv3} as driving forces to perform reasonable wall design and to avoid infinite loops.

Condition III:
If x_1, x_2, x_3 < B, then ONLY consider P_{v1}, P_{v2}, P_{v3} in 'resisting' moments and 'resisting' forces to perform reasonable wall design and to avoid infinite loops.

Surcharge loads are generally used in external stability analyses as driving forces only. Sometimes they are not included as resisting forces because their permanence cannot be guaranteed. However, this judgment should be made by the project designer and the owner.

Condition IV:
Uniform live surcharge load, q_1, is not considered in resisting moments and resisting forces. This condition is necessary because live loads are not permanent, thus, not providing resistance at all times.

Surcharge loading on backslope may be modeled as horizontal surcharge at the top of the slope on the horizontal surface. If backslope does not exist and the surcharge is located a short distance back from the wall face, the designer may model the surcharge by introducing a small slope angle and slope height to match the location of surcharge.

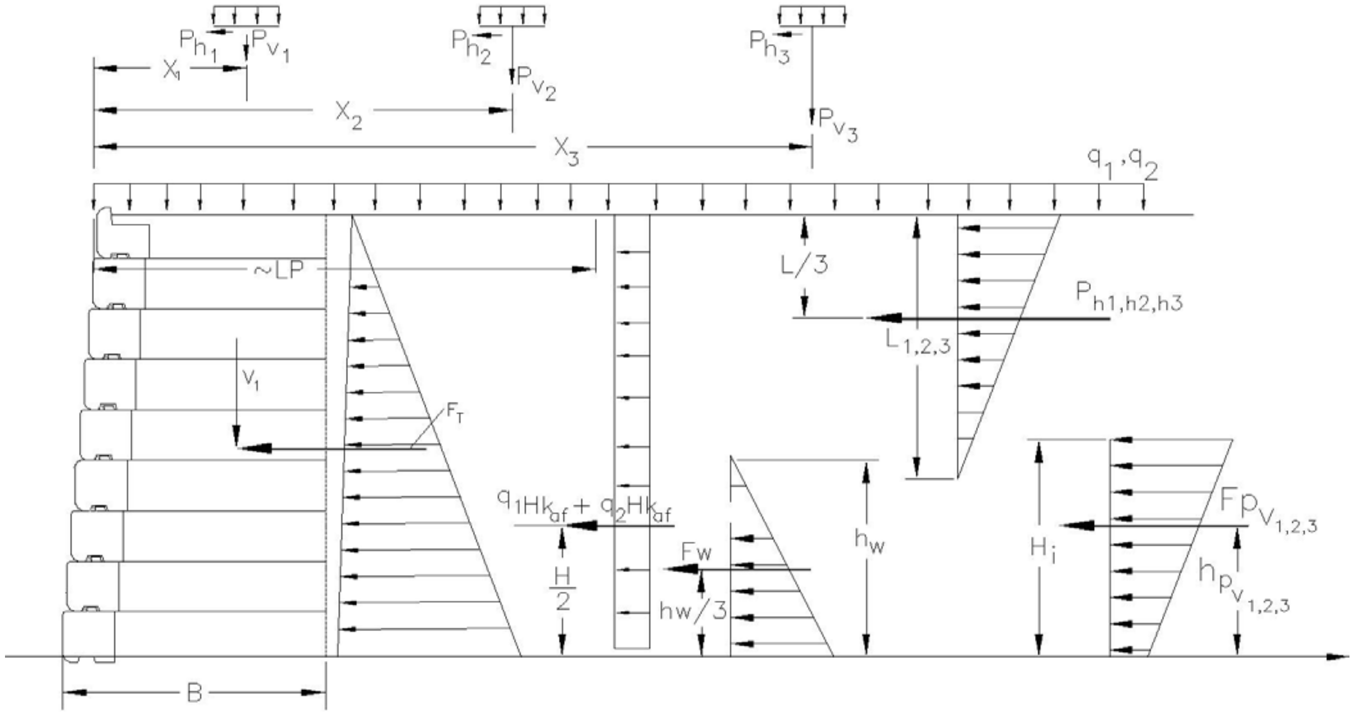


FIGURE 9 Superposition of Loads

EVALUATION OF EXTERNAL STABILITY

Evaluation of external stability includes determining the center of gravity of the wall and estimating the FSs against overturning, sliding, bearing capacity, and settlement modes of failure. External stability calculations are made by assuming that the StoneTerra™ MSE wall acts as a rigid body. External stability calculations are performed on a wall section of unit length. All forces are expressed as force per unit length of wall and moments as force-length per unit length of wall.

Evaluation of Overturning and Sliding Stability

The overturning stability of a StoneTerra™ MSE wall is evaluated by calculating a factor of safety (FS) that is the ratio of the sum of resisting moments (M_R) to the sum of the driving moments (M_D) taken about the toe of the wall as indicated below. The sliding stability of a StoneTerra™ MSE wall is evaluated by calculating a factor of safety (FS) that is the ratio of the external forces acting to destabilize the wall to the resisting forces acting against the destabilizing forces as indicated below. The overturning and sliding evaluations with different backslope configurations are illustrated in Figures 10 through 12.

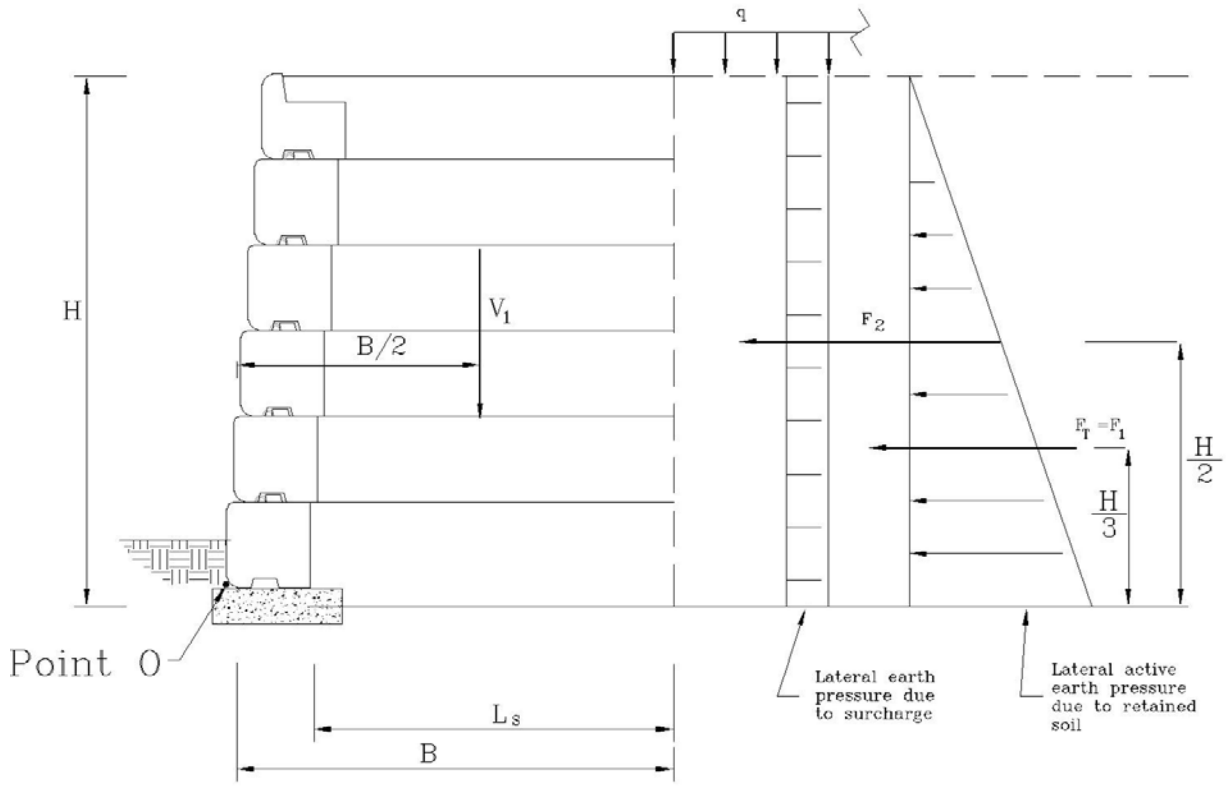


FIGURE 10 Overturning and Sliding– Horizontal Backslope with Traffic Surcharge

Case I: Horizontal Backslope (β = 0) with Traffic Surcharge

For overturning, take moments about Point "O" shown in Figure 10,

M_R = Resisting moment (lb-ft/ft of wall) = $V_1\left(\frac{B}{2}\right) = \left((\gamma_f)(H)(Ls) + (\gamma_{block})(H)(B_{block})\right)\left(\frac{B}{2}\right)$

M_D = Driving moment (lb-ft/ft of wall) = $F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)$

Factor of safety against overturning,

$$FS_{OT} = \frac{M_R}{M_D} \geq 2.0$$

For sliding,

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall) = $V_1\tan(\rho)$

Where $\rho = \tan^{-1}\left(\frac{2}{3}\tan(\phi_r)\right)$

Note: Use ϕ_r if $\phi_r < \rho$

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall) = $F_1 + F_2$

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} \geq 1.5$$

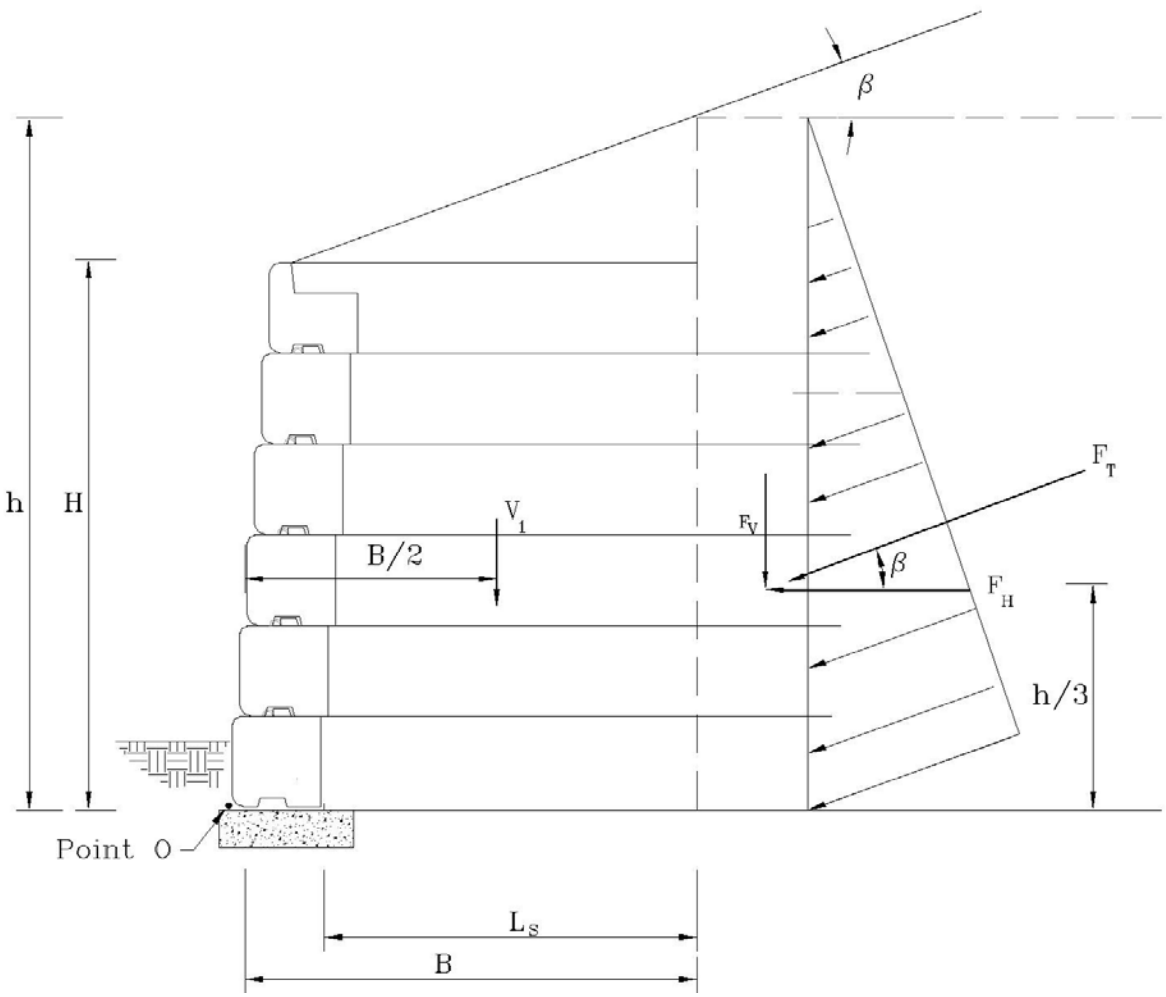


FIGURE 11 Overturning and Sliding– Sloping Backslope

Case II: Sloping Backslope

For overturning, take moments about Point "O" shown in Figure 11,

M_R = Resisting moment (lb-ft/ft of wall) = $V_1\left(\frac{B}{2}\right) + V_2\left(\frac{2Ls}{3}\right) + F_V B$

= $\left((\gamma_f)(H)(L) + (\gamma_{block})(H)(B_{block})\right)\left(\frac{B}{2}\right) + \frac{\gamma_r Ls(h-H)}{2}\left(\frac{2Ls}{3}\right) + F_T(\sin\beta)(B)$

M_D = Driving moment (lb-ft/ft of wall) = $F_H\left(\frac{H}{3}\right)$ = $F_T(\cos\beta)\left(\frac{H}{3}\right)$

Factor of safety against overturning,

$$FS_{OT} = \frac{M_R}{M_D} \geq 2.0$$

For sliding,

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall) = $(V_1 + V_2 + F_V)\tan(\rho)$

= $(V_1 + V_2 + F_T(\sin\beta))\tan(\rho)$
Note: Use ϕ_r if $\phi_r < \rho$

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall) = F_H = $F_T(\cos\beta)$

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} \geq 1.5$$

Case III: Broken Backslope ($\beta = I$)

For overturning, take moments about Point "O" shown in Figure 12,

M_R = Resisting moment (lb-ft/ft of wall) = $V_1\left(\frac{B}{2}\right) + V_2\left(\frac{2Ls}{3}\right) + F_V B$

$$= \left((\gamma_f)(H)(L) + (\gamma_{block})(H)(B_{block})\right)\left(\frac{B}{2}\right) + \frac{\gamma_r Ls(h-H)}{2}\left(\frac{2Ls}{3}\right) + F_T(\sin I)(B)$$

With,

F_T = $0.5\gamma_f h^2 k_{af}$

$$k_{af} = \frac{\sin^2(\theta + \phi_f)}{\sin^2\theta \sin(\theta - \delta)\left(1 + \sqrt{\frac{\sin(\phi_f + \delta)\sin(\phi_f - I)}{\sin(\theta - \delta)\sin(\theta + I)}}\right)^2}$$

M_D = Driving moment (lb-ft/ft of wall) = $F_H\left(\frac{H}{3}\right)$ = $F_T(\cos I)\left(\frac{H}{3}\right)$

Factor of safety against overturning,

$$FS_{OT} = \frac{M_R}{M_D} \geq 2.0$$

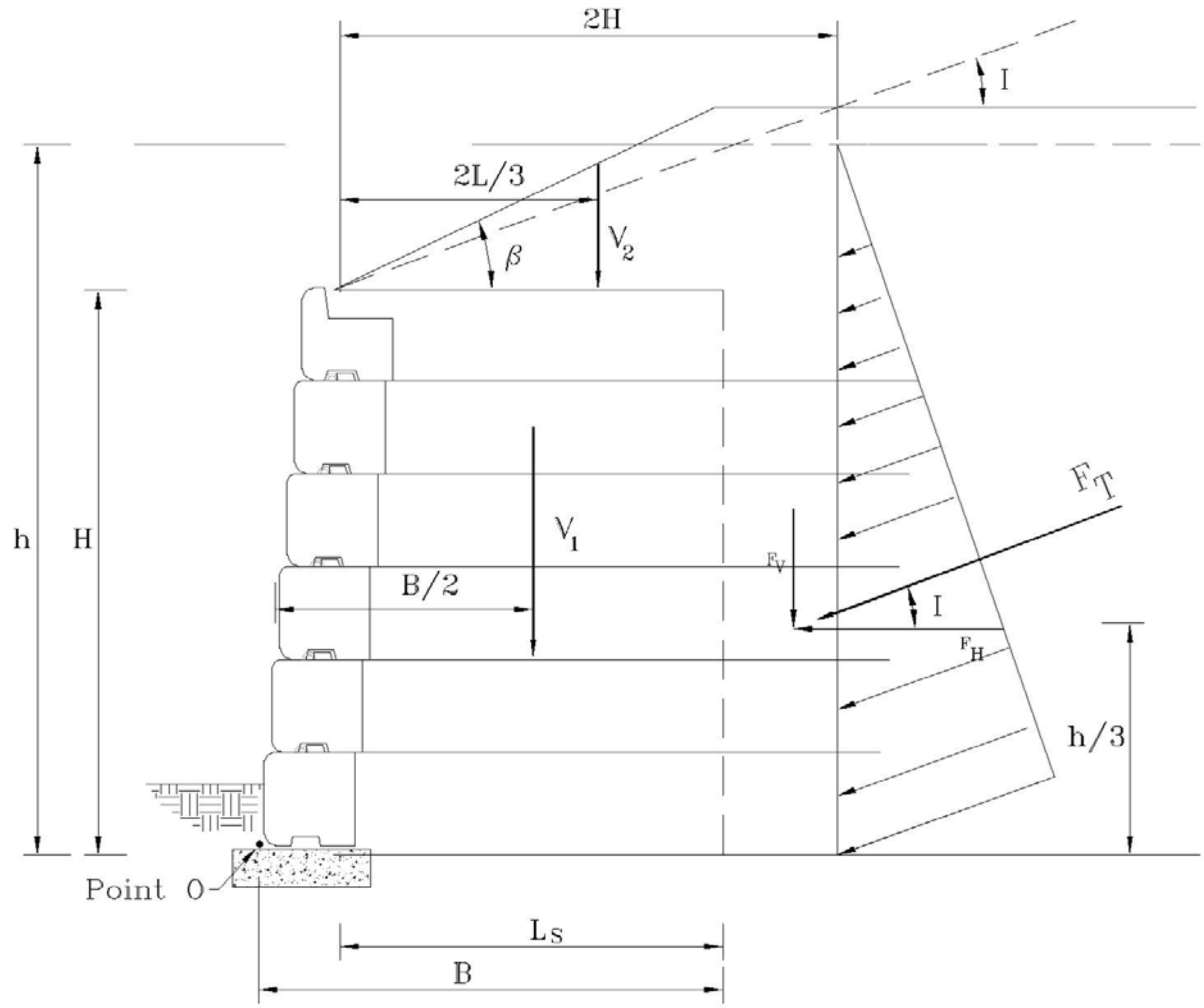


FIGURE 12 Overturning and Sliding – Broken Backslope

For sliding,

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall) = $(V_1 + V_2 + F_V)\tan(\rho)$

= $(V_1 + V_2 + F_T(\sin I))\tan(\rho)$

Note: Use ϕ_r if $\phi_r < \rho$

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall) = F_H = $F_T(\cos I)$

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} \geq 1.5$$

Case IV: Surcharge Loading

Assuming two uniform vertical surcharge loads (q_1 and q_2), three vertical surcharge footing loads (Q_1, Q_2, Q_3, or P_{v1}, P_{v2}, P_{v3}) and three horizontal surcharge loads (P_{h1}, P_{h2}, P_{h3}):

For overturning,

M_R = Resisting moment (lb-ft/ft of wall)

$$= V_1\left(\frac{B}{2}\right)+V_2\left(\frac{2Ls}{3}\right)+F_V B+P_{v1}(x_1)+P_{v2}(x_2)+P_{v3}(x_3)$$

=

$$\left((\gamma_f)(\mathrm{H})(\mathrm{L})+(\gamma_{\mathrm{block}})(H)(B_{block})\right)\left(\frac{B}{2}\right)+\frac{\gamma_r Ls(h-H)}{2}\left(\frac{2Ls}{3}\right)+F_T(\sin\beta)(B)+P_{v1}(x_1)+P_{v2}(x_2)+P_{v3}(x_3)$$

M_D = Driving moment (lb-ft/ft of wall

$$= \mathrm{F_H}\left(\frac{\mathrm{H}}{3}\right)+(\mathrm{F}_{q_1}+\mathrm{F}_{q_2})\left(\frac{\mathrm{H}}{2}\right)+\mathrm{P_{h1}}\left(\mathrm{H}-\frac{\mathrm{L_1}}{3}\right)+\mathrm{P_{h2}}\left(\mathrm{H}-\frac{\mathrm{L_2}}{3}\right)+\mathrm{P_{h3}}\left(\mathrm{H}-\frac{\mathrm{L_3}}{3}\right)+\mathrm{F_w}\left(\frac{\mathrm{H_w}}{3}\right)$$
$$+\mathrm{F_{pv1}}(\mathrm{h_{pv1}})+\mathrm{F_{pv2}}(\mathrm{h_{pv2}})+\mathrm{F_{pv3}}(\mathrm{h_{pv3}})$$

$$= \mathrm{F_T}(\cos\beta)\left(\frac{\mathrm{H}}{3}\right)+(\mathrm{F}_{q_1}+\mathrm{F}_{q_2})\left(\frac{\mathrm{H}}{2}\right)+\mathrm{P_{h1}}\left(\mathrm{H}-\frac{\mathrm{L_1}}{3}\right)+\mathrm{P_{h2}}\left(\mathrm{H}-\frac{\mathrm{L_2}}{3}\right)+\mathrm{P_{h3}}\left(\mathrm{H}-\frac{\mathrm{L_3}}{3}\right)+\mathrm{F_w}\left(\frac{\mathrm{H_w}}{3}\right)$$
$$+\mathrm{F_{pv1}}(\mathrm{h_{pv1}})+\mathrm{F_{pv2}}(\mathrm{h_{pv2}})+\mathrm{F_{pv3}}(\mathrm{h_{pv3}})$$

Factor of safety against overturning,

$$\mathrm{FS_{OT}}=\frac{\mathrm{M_R}}{\mathrm{M_D}}\geq 2.0$$

For sliding,

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall)

= $(V_1 + V_2 + F_V + P_{v1} + P_{v2} + P_{v3})\tan(\rho)$

= $(V_1 + V_2 + F_T(\sin\beta) + P_{v1} + P_{v2} + P_{v3})\tan(\rho)$

Note: Use ϕ_r if $\phi_r < \rho$

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall)

$$= \mathrm{F_H}+\mathrm{F}_{q_1}+\mathrm{F}_{q_2}+\mathrm{F_{pv1}}+\mathrm{F_{pv2}}+\mathrm{F_{pv3}}+\mathrm{P_{h1}}+\mathrm{P_{h2}}+\mathrm{P_{h3}}+\mathrm{F_w}$$

$$= \mathrm{F_T}\cos\beta+\mathrm{F}_{q_1}+\mathrm{F}_{q_2}+\mathrm{F_{pv1}}+\mathrm{F_{pv2}}+\mathrm{F_{pv3}}+\mathrm{P_{h1}}+\mathrm{P_{h2}}+\mathrm{P_{h3}}+\mathrm{F_w}$$

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} \geq 1.5$$

The calculated FS_{OT} value should not be excessively large or less than the minimum required FS shown in Table 1. The design wall height and/or wall batter should be adjusted as necessary to optimize the wall design.

The calculated FS_{SL} value should not be excessively large or less than the minimum required FS shown in Table 1. The wall base should be adjusted as necessary to optimize wall design. Also, a geotechnical engineer should be consulted to establish the wall base friction coefficient value.

As indicated earlier, the wall base friction coefficient shall be based upon the friction angle of base material, typically crushed rock. The wall base friction angle shall be equal to or less than the wall backfill friction angle ($\delta_b < \delta$). In any case, considerable geotechnical engineering judgment is necessary to choose optimal base friction coefficient design values.

Passive resistance at the base of wall is neglected in sliding stability calculations because the long-term existence and competence of passive soil wedge cannot be guaranteed. See Design Examples in Appendix C.

Evaluation of Bearing Capacity and Foundation Stability

Bearing capacity can be defined as the ability of the foundation soils to support the bearing pressures exerted by the wall system. Bearing capacity criteria generally consist of the shear capacity and the settlement (total and differential) of the supporting soil mass.

Bearing capacity failure or tilting of a StoneTerra™ MSE wall can occur due to overstressing of the foundation soils. Therefore, the bearing pressure exerted by a StoneTerra™ MSE wall structure must be less than the allowable bearing capacity of foundation soils.

Estimation of Bearing Pressure

The bearing capacity and foundation stability of a StoneTerra™ MSE wall is evaluated by calculating a factor of safety (FS_{BC}) that is the ratio of the ultimate bearing capacity (q_{ult}) to allowable bearing capacity (q_{all}). The calculated FS_{BC} value should not be excessively large or less than the allowable design value (typically 3.0). The wall base or thickness can be adjusted as necessary to optimize the wall design.

The important aspect of bearing capacity and foundation stability evaluation is the reasonable estimation of ultimate bearing capacity. Since the bearing capacity value is generally provided by a geotechnical engineer as the 'allowable bearing capacity' instead of the 'ultimate bearing capacity', it is more convenient to simply confirm that the calculated maximum vertical stress is not greater than the recommended 'allowable bearing capacity.'

If a geotechnical engineer simply recommends a bearing capacity value based on local experience assuming a minimal footing width, the value may be highly conservative and may not be applicable to given wall characteristics such as the wall size, the location of the wall foundation subgrade below exterior grades, or the reinforcement length. Geotechnical reports are often not prepared specifically for given wall characteristics. It is, therefore, important to discuss proposed wall characteristics with a geotechnical engineer before asking for an allowable bearing capacity value. The designer must confirm that a geotechnical engineer has reviewed proposed wall characteristics and actually calculated the ultimate bearing capacity.

In some cases, additional subsurface exploration and laboratory testing may become necessary to obtain reasonable soil properties for optimizing the wall design with respect to bearing capacity and foundation stability.

In general, the ultimate bearing capacity of foundation soils can be estimated as indicated below and as shown in Figures 13, 14, and 15. A geotechnical engineer must be consulted to review and confirm the calculated bearing capacity value for given foundations soils and for given wall characteristics.

q_{ult}	=	Ultimate bearing capacity (psf)
	=	$cN_c + 0.5\gamma_{found}\ B'\ N_\gamma + q\ N_q$, where,
N_c, N_q, N_γ	=	Bearing capacity factors (see Table D1 in Appendix D)
ϕ_{found}	=	Friction angle of foundation soil
c_{found}	=	Cohesion of foundation soil (psf)
γ_{found}	=	Wet unit weight of foundation soil (pcf)
H_{emb}	=	Depth of embedment of blocks (ft)
q	=	Surcharge = $\gamma\ (H_{emb})$
q_{all}	=	Allowable bearing capacity (psf) = $\dfrac{q_{ult}}{FS}$

The bearing capacity equation is the classical bearing capacity equation for an infinitely long continuous strip footing with shape and depth factors = 1.0. The quantities N_c, N_q and N_γ , are dimensionless bearing capacity coefficients (Vesic,1975) that can be obtained from Table D1 in Appendix D using the internal friction angle of the foundation soil, ϕ_{found}. There are several other bearing capacity equations and corresponding bearing capacity factors that can be referred to in any classical geotechnical text book. Note that the estimation of allowable bearing capacity is neither pure science nor a mathematical exercise, but a ‘scientific art’ requiring considerable geotechnical engineering judgment.

The following guidelines may be used for the preliminary estimation of allowable bearing capacity and to avoid the use of unreasonably low or excessively high allowable bearing capacity values.

• **Native Soil Type**	**ϕ (degrees)**	**γ(pcf)**	**q_{all} (psf)**
Weathered Rock	34-40	120-135	3500-6000
Sands	30-34	105-125	2500-3500
Silty sands	28-32	110-125	2000-2750
Sandy silts	28-30	110-125	2000-2500
Sandy clays	24-28	100-115	1750-2000
• **Fill Soil Type**			
Dense Well Graded Crushed Rock	34-36	125-135	3000-4000
Medium Well Graded Dense Crushed Rock	30-34	115-125	2500-3000
Medium Dense Sands or Sand-Silt Mixtures	28-32	100-115	1750-2000

A geotechnical engineer must be consulted to review and confirm the final design bearing capacity value for given foundations soils, settlement criteria, and wall characteristics.

Eccentricity and Bearing Pressures

Case I: Horizontal Backslope (β = 0) with Traffic Surcharge

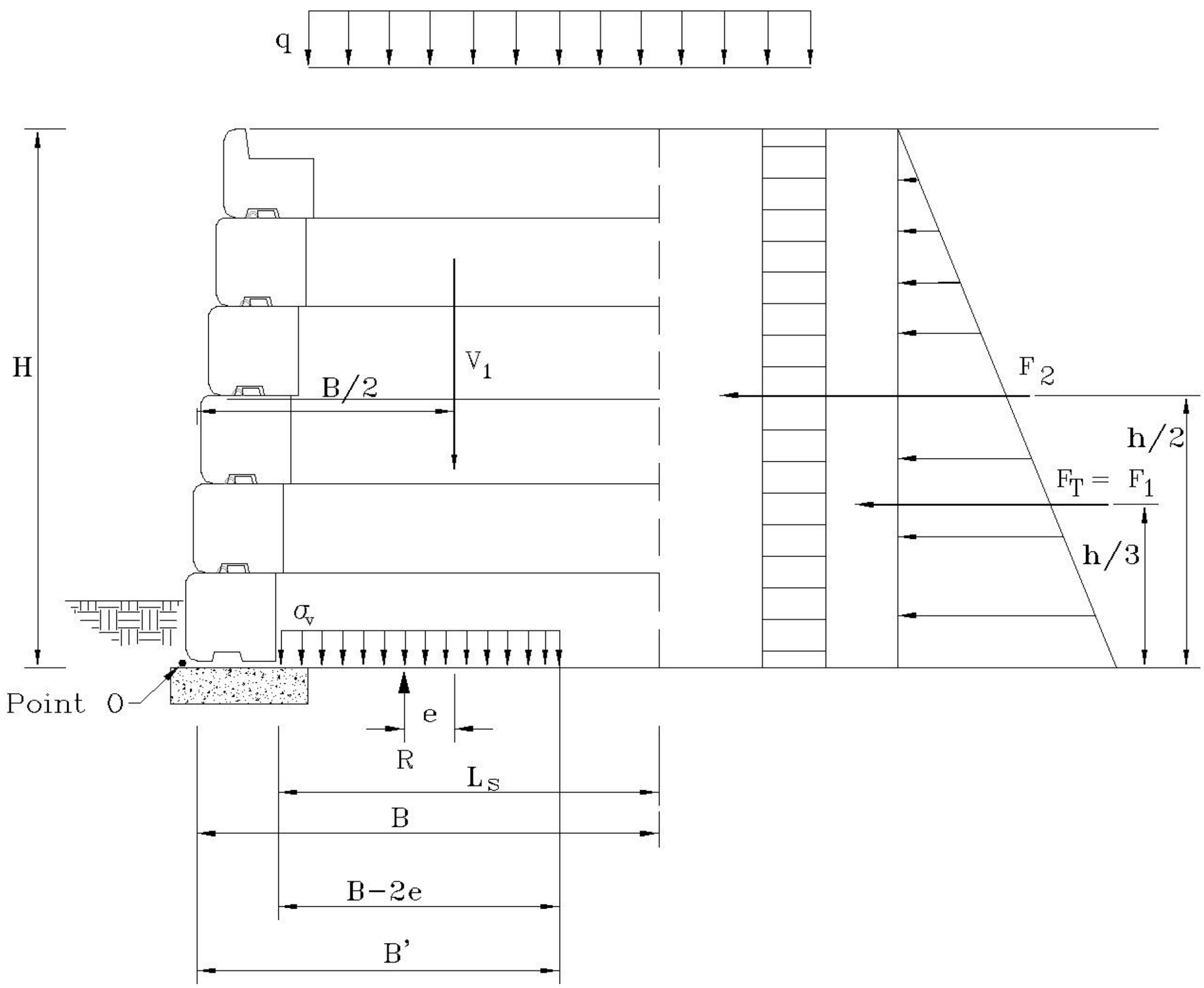


FIGURE 13 Bearing Capacity – Horizontal Backslope with Traffic Surcharge

Eccentricity of base loading can be calculated by summing moments about the center of the footing base. Eccentricity and bearing pressures are calculated as follows and as shown in Figure 13.

Where:

B' = Effective base width (ft)

R = Resultant of vertical forces (lbs/ft of wall) = $V_1 + qB$

e = Eccentricity of R (ft) = $\dfrac{F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)}{R}$

Verify that:

$$\frac{V_1 + qB}{B - 2e} \le q_{all}$$

Case II: Sloping Backslope

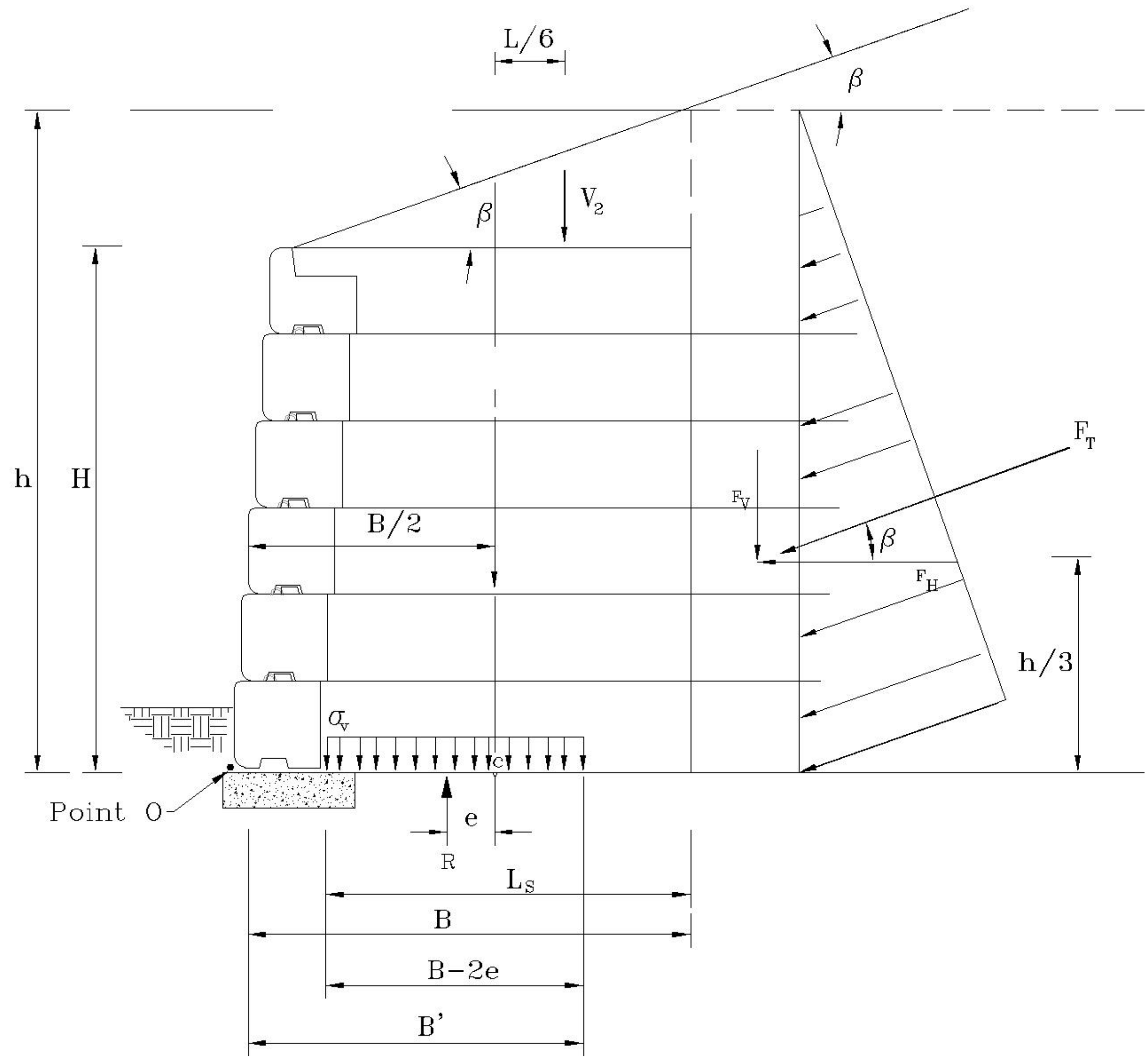


FIGURE 14 Bearing Capacity – Sloping Backslope

Eccentricity of base loading can be calculated by summing moments about the center of the footing base. Eccentricity and bearing pressures are calculated as follows and as shown in Figure 14.

Where:

R = $V_1 + V_2 + F_V = V_1 + V_2 + F_T \sin\beta$

$$e = \frac{F_H\left(\frac{H}{3}\right) - F_V\left(\frac{B}{2}\right) - V_2\left(\frac{Ls}{6}\right)}{R} = \frac{F_T(\cos\beta)\left(\frac{H}{3}\right) - F_T(\sin\beta)\left(\frac{B}{2}\right) - V_2\left(\frac{Ls}{6}\right)}{R}$$

Verify that:

$$\frac{V_1 + V_2 + F_V}{B - 2e} \leq q_{all}$$

Case III: Broken Backslope (β = I)

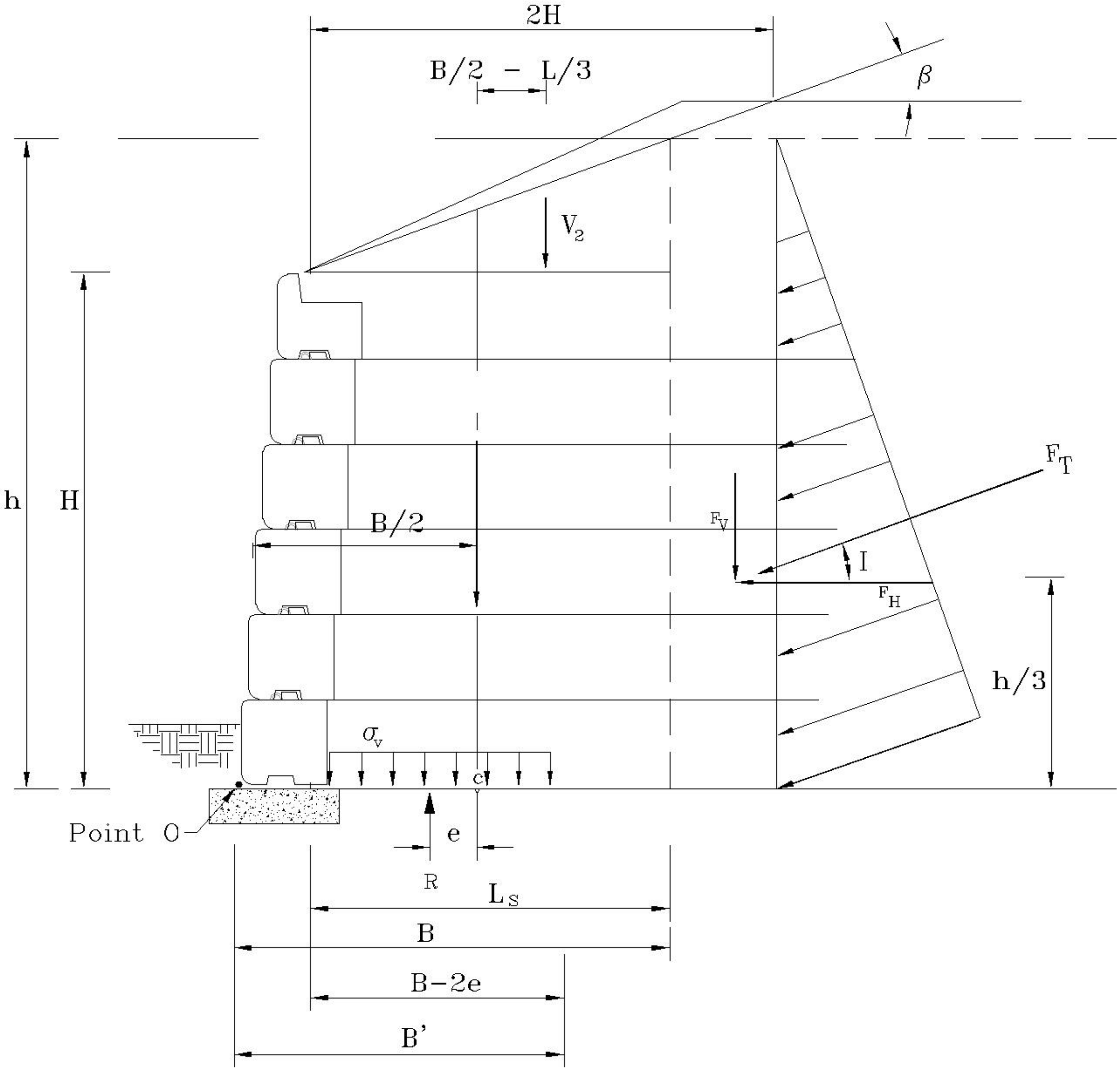


FIGURE 15 Bearing Capacity – Broken Backslope

Eccentricity of base loading can be calculated by summing moments about the center of the footing base. Eccentricity and bearing pressures are calculated as follows and as shown in Figure 15.

Where:

R = $V_1 + V_2 + F_V$

$$e = \frac{F_H\left(\frac{H}{3}\right) - F_V\left(\frac{B}{2}\right) - V_2\left(\frac{B}{2} - \frac{L}{3}\right)}{R} = \frac{F_T(\cos I)\left(\frac{H}{3}\right) - F_T(\sin I)\left(\frac{B}{2}\right) - V_2\left(\frac{B}{2} - \frac{L}{3}\right)}{R}$$

Verify that:

$$\frac{V_1 + V_2 + F_V}{B - 2e} \leq q_{all}$$

Case IV: Sloping Backslope (β = 0) with Surcharge Loading

Assuming two uniform vertical surcharge loads (q_1 and q_2), three vertical surcharge footing loads (Q_1, Q_2, Q_3, or P_{v1}, P_{v2}, P_{v3}) and three horizontal surcharge loads (P_{h1}, P_{h2}, P_{h3}). Eccentricity of base loading can be calculated by summing moments about the center of the footing base. Eccentricity and bearing pressures are calculated as follows.

Where:

$$R = V_1 + V_2 + F_V + (q_1+q_2)B + P_{v1} + P_{v2} + P_{v3}$$

$$e = \left[F_H\left(\frac{H}{3}\right) - F_V\left(\frac{B}{2}\right) - V_2\left(\frac{L}{6}\right) + (q_1Hk_{af} + q_2Hk_{af})\left(\frac{H}{2}\right) + P_{h1}\left(H - \frac{L_1}{3}\right) + P_{h2}\left(H - \frac{L_2}{3}\right) + P_{h3}\left(H - \frac{L_3}{3}\right) + F_w\left(\frac{H_w}{3}\right) + F_{pv1}(h_{pv1}) + F_{pv2}(h_{pv2}) + F_{pv3}(h_{pv3}) - P_{v1}\left(\frac{B}{2} - x_1\right) - P_{v2}\left(\frac{B}{2} - x_2\right) - P_{v3}\left(\frac{B}{2} - x_3\right)\right] \Big/ R$$

Verify that:

$$\frac{V_1 + V_2 + F_V}{B - 2e} + \frac{P_{v1}}{Br_1 + H} + \frac{P_{v2}}{(Br_2 + H)(Lr_2 + H)} + \frac{P_{v3}}{H^2} \leq q_{all}$$

See Appendix C for sample calculations.

Settlement

A StoneTerra™ MSE wall may impose pressures on foundation soils leading to compression of foundation materials and subsequent settlement of the StoneTerra™ MSE wall structure. This settlement may include elastic and/or consolidation and/or secondary components based upon the foundation soil conditions.

In cohesionless soils such as sands and gravels, settlements are typically small and occur mostly during construction. In saturated cohesive soils such as soft, silty clays or clayey silts, large time-dependent settlements may occur due to sustained wall loading. A qualified geotechnical engineer must be consulted to evaluate time-dependent settlements in saturated cohesive soils. In general, procedures given in the AASHTO LRFD Bridge Design Specifications may be used to evaluate foundation settlements.

Settlement criteria include limiting values for total settlement and differential settlement. Total settlement criterion is directly related to the allowable bearing capacity value and compliance of adjacent or supported structures, utilities, or other improvements. Differential settlements can result in flexural movement in the wall face, realignment of blocks, cracking of blocks, and damage to adjacent structure, utilities and other improvements. . Differential settlements should be limited to 1 foot in 100 linear feet of wall (1%) for non-critical structures and 0.5% for critical structures.

Differential settlements should be carefully evaluated where non-uniform subgrade conditions exist. This may include partial exposure of bedrock, sudden transition from elastic to rigid surfaces, crossing over utility trenches, box culverts, etc.

Generally, unsuitable soil conditions at the foundation level can be improved by using various methods. Some of these methods include:

- Preloading of foundation area to reduce post-construction settlements
- Overexcavation of incompetent material and replacing with structural backfill
- Expanding the aggregate leveling pad in thickness and width
- Reinforcing the aggregate leveling pad with geogrid
- Reducing foundation loads by tiering the wall
- Improving foundation soil conditions using dynamic compaction, geopier installation, etc.

A qualified geotechnical engineer should be consulted before using any type of soil improvement technique.

Tolerable settlement criteria should be project-specific and developed consistent with the function of proposed structure, anticipated service life, and consequence of unacceptable movements on structural performance and adjacent improvements.

EVALUATION OF OVERALL (GLOBAL) STABILITY

The global slopes (retained backslope and foreslope) adjacent to the StoneTerra™ MSE wall and foundation soils may fail as illustrated in Figure 2 due to changes in grades, soil strength, groundwater regime, and/or the additional gravitational forces imposed on the site soils by the StoneTerra™ MSE wall structure, etc. This general soil mass movement around the StoneTerra™ MSE wall structure is called an overall stability failure. The overall stability evaluation becomes critical when one or more combinations of the following conditions exist:

- Groundwater table near the wall foundation
- Steep retained backslopes and foreslopes
- Loose silty soils and soft cohesive soils are present below the wall foundation
- Tiered walls
- Walls adjacent to waterways
- Excessive and wide surcharge loading

The overall stability evaluation should be based upon the limiting equilibrium method of analyses, which employ the Modified Bishop, Simplified Janbu, or Spencer methods of analyses. These analyses use the "method of slices" approach in which vertical slices of soil above a trial failure surface are examined with respect to force and moment equilibrium as shown in Figure 16. The shape of the assumed failure surfaces is examined with respect to force and moment equilibrium. Several such failure surfaces are examined until the most critical failure surface (or minimum FS value) is found. Several geotechnical textbooks are available for a detailed discussion on global slope stability analyses.

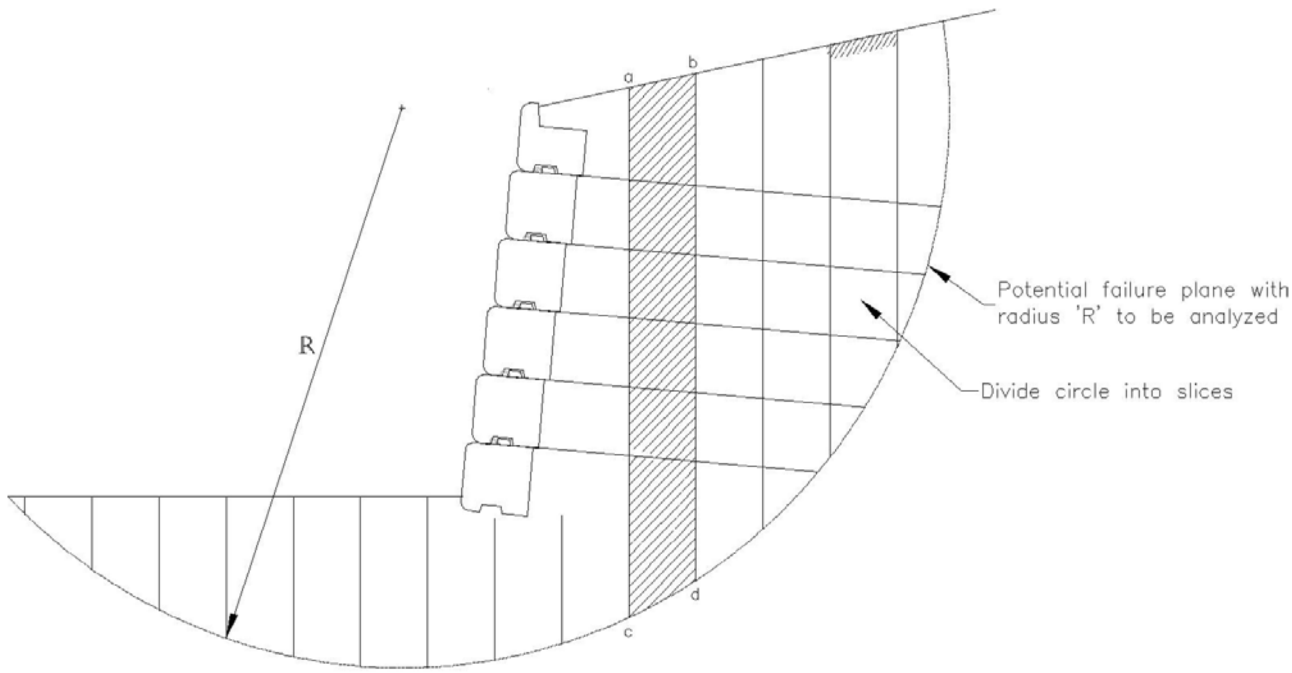


FIGURE 16 Global Stability Analysis – Method of Slices

The overall stability calculations for determining the most critical failure surface are complicated and often tedious because of the need for analyzing many potential failure surfaces. Computer software is generally used for overall (global slope) stability analyses. XSTABL™ and Slope/W are two examples of software packages available for global slope stability analyses. See References in Appendix E for other available software packages.

The calculated FS value should not be less than the allowable design value (typically 1.3 to 1.5). Otherwise, the design wall height, depth of embedment, and/or wall batter should be adjusted to optimize the wall design. Because of several complexities involved in the analytical procedures including the variety of variables (topographic and geometric conditions, soil strength parameters, subsurface soil strata modeling, and groundwater modeling), the overall stability should be performed by the geotechnical engineer. For modeling purposes, the wall should be modeled as a soil with high cohesion to prevent the occurrence of global failure circles through the wall.

EVALUATION OF INTERNAL AND LOCAL STABILITY

Evaluation of internal stability includes the determination of the FS against geogrid tensile overstress (rupture), geogrid pullout, and internal sliding modes of failure. These modes of failures are illustrated in Figure 2. Internal stability is evaluated by equating the tensile load applied to the geogrid to the allowable tension for the geogrid. The allowable tension for the geogrid is governed by geogrid rupture and pullout. The tensile load in the geogrid is determined at the zone of maximum stress and at the connection with the wall units. The zone of maximum stress is assumed to be located at the boundary between the active zone and the resistant zone as illustrated in Figure 17.

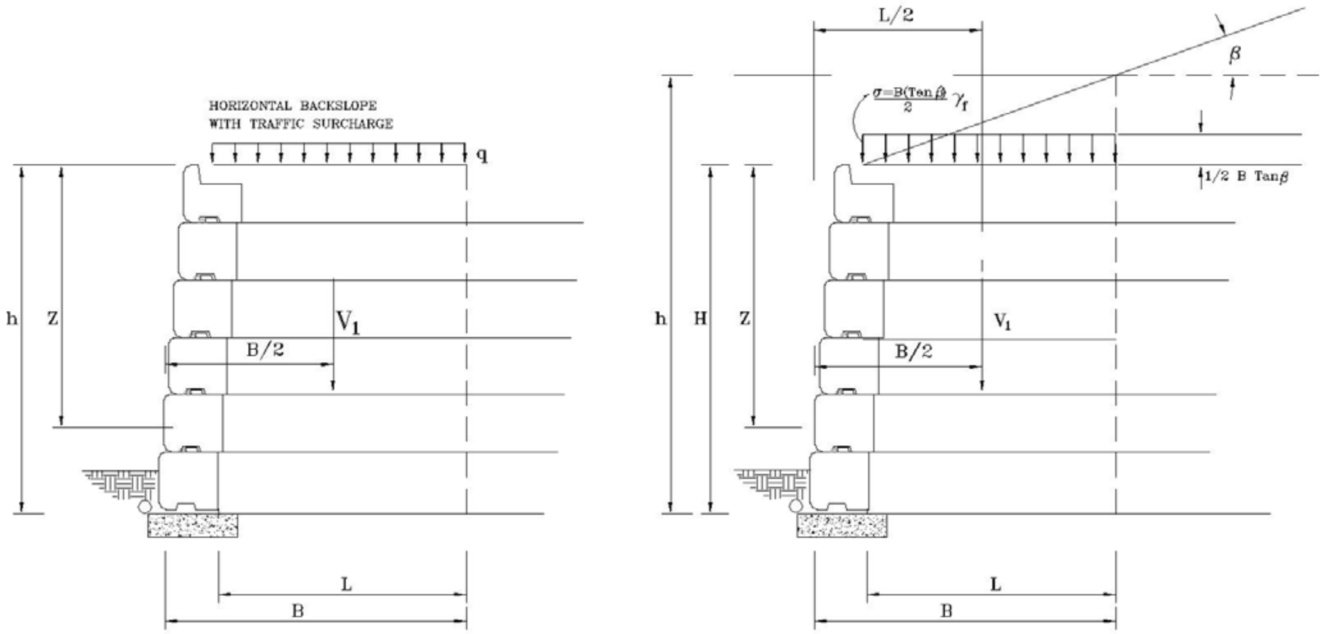


FIGURE 17 Internal Stability – Maximum Reinforcement Loads

Maximum Tensile Forces in Reinforcement Layers

Estimation of maximum geogrid tensile loads is estimated using a Simplified Coherent Gravity approach. The tensile load in the geogrid is obtained by multiplying the vertical stress at the geogrid layer by a lateral earth pressure coefficient, and applying the resulting lateral pressure to the tributary area for the geogrid layer. The vertical stress at the geogrid layer is calculated as overburden pressure from soil weight above the geogrid layer and from any surcharge loading.

The most important design parameters are the allowable tensile strength of geogrid (T_a) and the allowable connection strength (T_{ac}). T_a is defined as the Long Term Design Strength (LTDS) divided by an adequate factor of safety (FS) to account for uncertainties in the geometry of the structure, fill properties, reinforcement properties, and external surcharge loads. The majority of the reference documents recommend a FS of approximately 1.5. Thus, T_a = LTDS/1.5. The LTDS values are based on extensive testing and analysis and are generally provided by the geogrid manufacturer. Sample geogrid product data (with LTDS analysis) are included in Appendix B. LTDS values obtained from Machine Direction (MD) testing should be used and geogrids installed with the machine direction perpendicular to the wall face.

Calculation details are as shown below. Parameter are defined below and illustrated in Figure 17.

Where:

σ_v	=	Maximum vertical stress (lbs/ft^2)
σ_h	=	Maximum horizontal stress (lbs/ft^2)
$\Delta\sigma_v$	=	Any additional dead load surcharge determined by a geotechnical engineer (lbs/ft^2)
$\Delta\sigma_h$	=	Any additional horizontal stress at the reinforcement location due to dead load surcharge as determined by a geotechnical engineer (lbs/ft^2)
Z	=	Depth below top of wall (ft)
S_v	=	Vertical spacing of reinforcement layers (ft) = 2.0 ft (typically)
T_{max}	=	Maximum reinforcement load per unit width of wall applied to each layer reinforcement (lbs/ft)
	=	$\sigma_h(S_v)$

T_a = Allowable tensile load in soil reinforcement per geogrid manufacturer (lbs/ft), see Appendix B

$$k_r = \frac{\sin^2(\theta+\phi_r)}{\sin^2\theta\sin(\theta-\delta)\left(1+\sqrt{\frac{\sin(\phi_r+\delta)\sin(\phi_r-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right)^2}$$

Horizontal Stress at Each Reinforcement Level

Calculate horizontal stress, σ_h , due to the weight of the retained fill and any uniform surcharge loads along the potential failure line.

$$\sigma_h = \sigma_v k_r + \Delta\sigma_h$$

Where

$$\sigma_v = \gamma_r Z + q + \Delta\sigma_v \text{, for horizontal backslope}$$

$$\sigma_v = \gamma_r Z + \frac{L\tan\beta}{2}\gamma_f + \Delta\sigma_v \text{, for sloping backslope}$$

Maximum Tension in Each Reinforcement Layer

Calculate the maximum tension, T_{max}, at each reinforcement layer per unit width of wall based on the vertical geogrid spacing, S_V. For StoneTerra™ MSE walls, S_V = 2.0 feet.

$$T_{max} = \sigma_h S_V$$

Verify that,

$$T_a \geq T_{max}$$

Surcharge Loading

Vertical Surcharge Loading

To determine the additional vertical stress ($\Delta\sigma_v$) caused by vertical loading, refer to Figure 18 and the equations below.

Strip Load: $\Delta\sigma_v = \dfrac{P_v}{D_1}$

Isolated footing load: $\Delta\sigma_v = \dfrac{P_v}{D_1(L+Z_1)}$

Point load: $\Delta\sigma_v = \dfrac{P_v}{D_1^2}$

For $Z_1 \leq Z_2$, $D_1 = B_r + \dfrac{2Z_1}{2} = B_r + Z_1$

For $Z_1 > Z_2$, $D_1 = \dfrac{B_r + Z_1}{2} + x$

Where,

x	=	distance from face of wall to P_v (ft)
L	=	length of footing (ft)
B_r	=	footing width (ft)
D_1	=	effective width of applied load at any depth, Z_1 (ft)
Z_2	=	depth where effective width intersects back of wall (ft)
	=	$2x - B_r$

For eccentrically loaded footings, replace B_r with B', the equivalent footing width:

B' = $B_r - 2e$

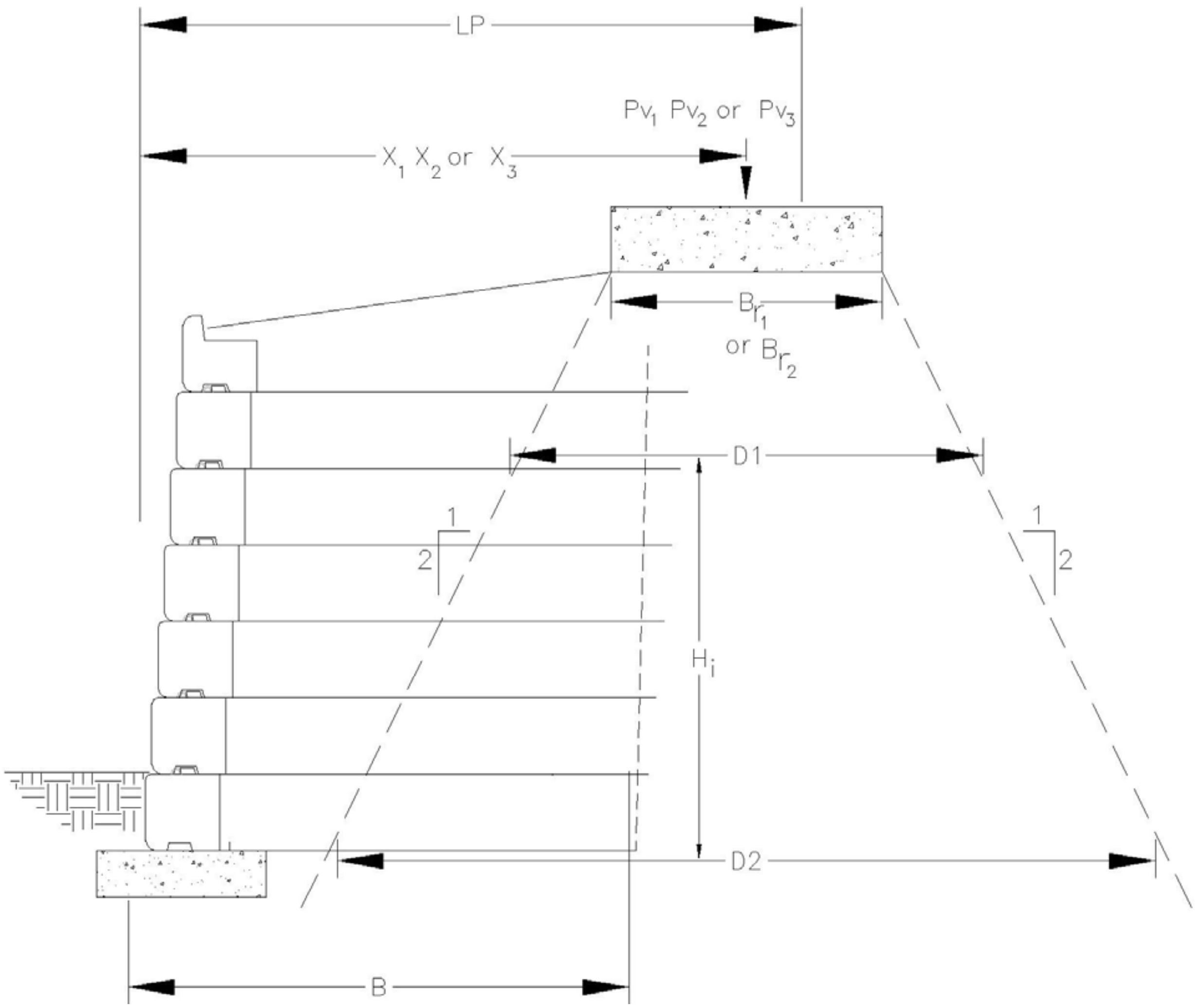


FIGURE 18 Internal Stability - Vertical Load Stress Distribution

Horizontal Surcharge Loading

To determine the additional vertical stress ($\Delta\sigma_h$) caused by horizontal loading, refer to Figure 19 and the equations below. Only consider horizontal loads that fall within the reinforced zone.

$$\Delta\,\sigma_{h\,max} = \frac{2(P_{h2} + F_{q1} + F_{q2})}{L_1} = \frac{2(P_{h1} + P_{h2} + P_{h3} + q_1Hk_r + q_2Hk_r)}{L_1}$$

Where,

$$L_1 = (x_h + B_{rh} - 2e)\tan\left(45 + \frac{\phi_r}{2}\right)$$

$$\Delta\sigma_{h_z} = \frac{Z(\Delta\sigma_{h_{max}})}{L_1}$$

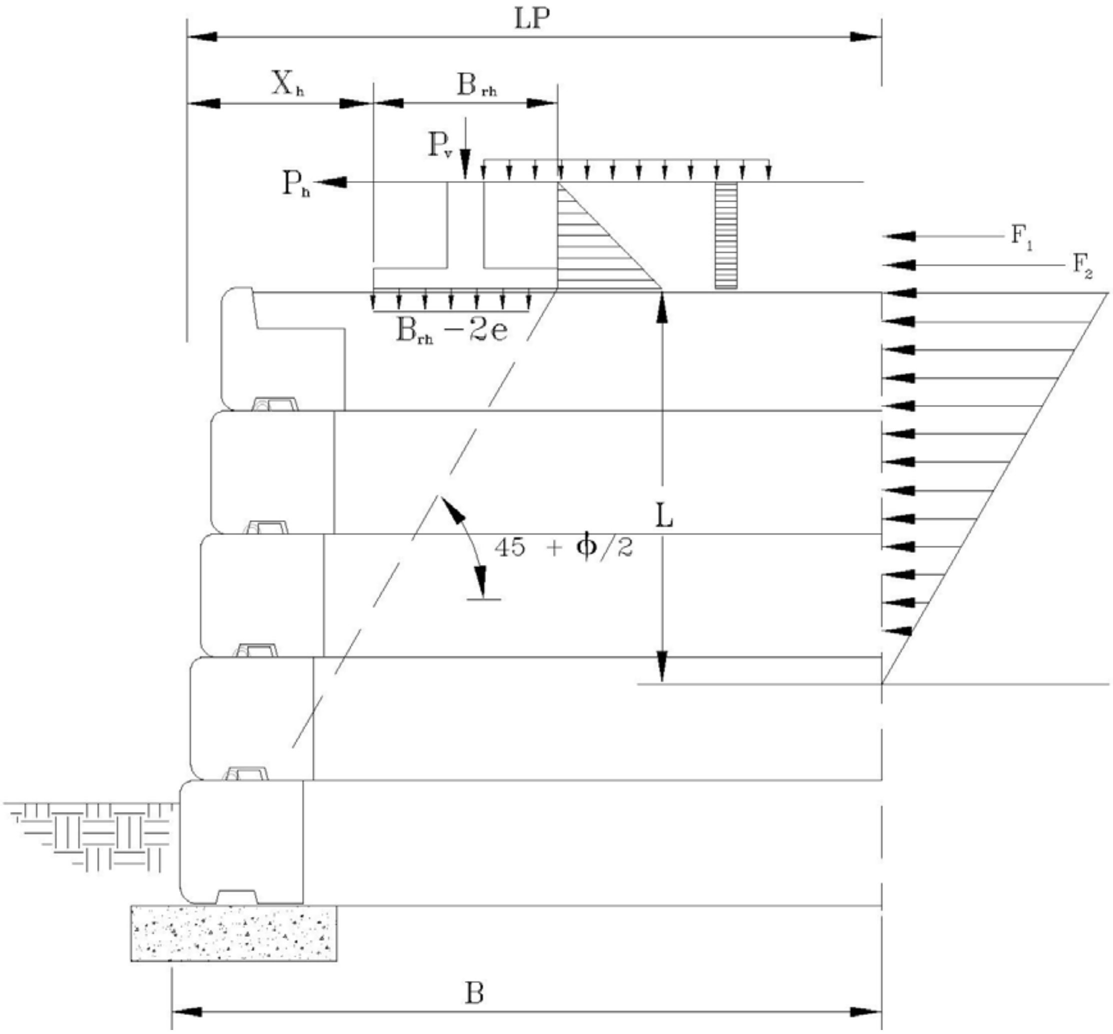


FIGURE 19 Internal Stability - Horizontal Load Stress Distribution

Pullout Failure

Geogrid pullout resistance is checked at each geogrid level. The geogrid length at each level should be adequate beyond the maximum stress zone is order to avoid pullout due to tensile loads. As indicated earlier, the zone of maximum stress is assumed to be located at the boundary between the active zone and the resistant zone as illustrated in Figure 20. The zone of maximum stress is assumed to begin at the back of the facing elements at the toe of the wall.

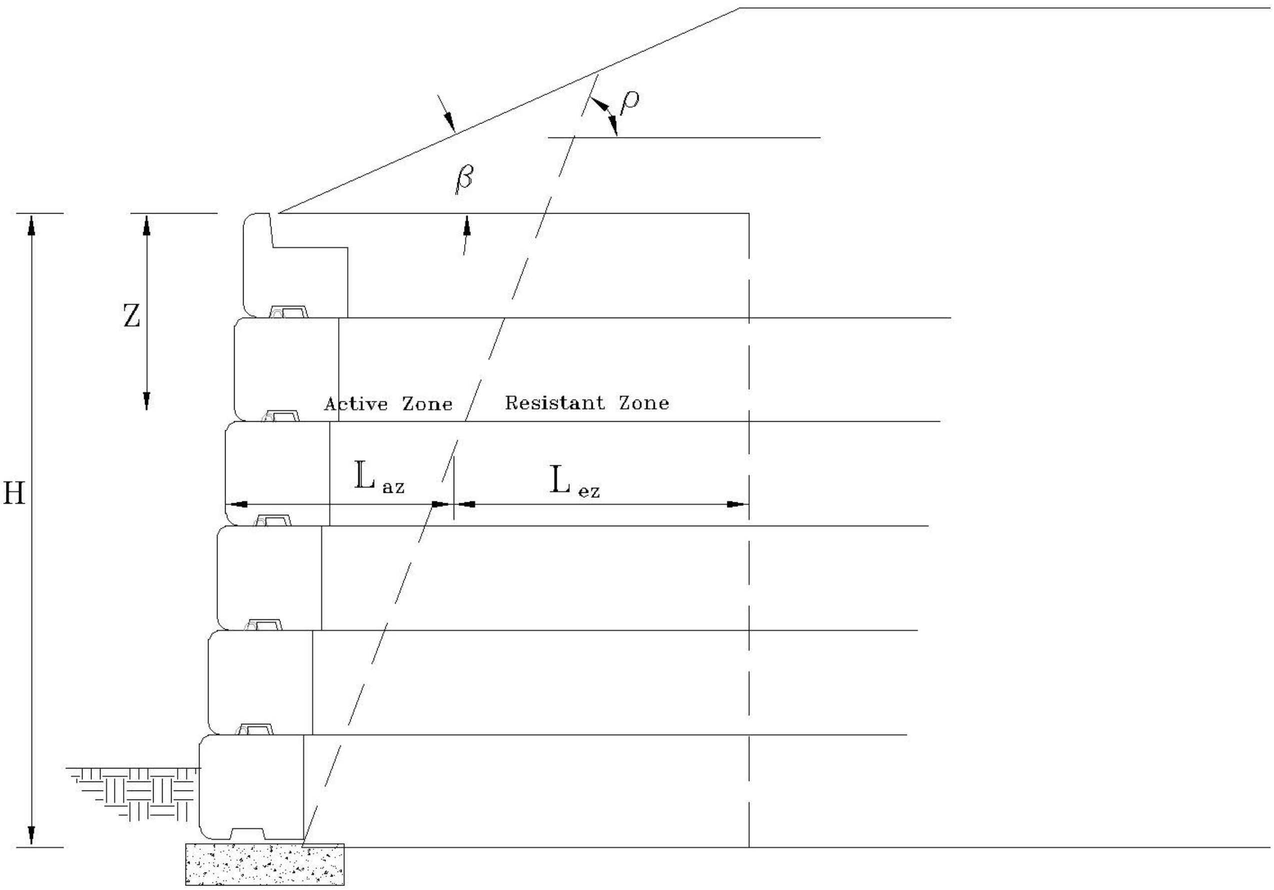


FIGURE 20 Active and Resistant Zones

The geogrid length beyond the active stress zone (resistant zone) is known as the effective pullout length. This effective pullout length should be estimated as shown below and as illustrated in Figure 20. The total geogrid length (L) required to avoid pullout is the sum of geogrid lengths between blocks (a), in active zone (L_{az}) and in resistant zones (L_{ez}). According to AASHTO recommendations, the minimum effective pullout length (L_{ez}) is 3 feet. In addition, the pullout resistance, P_r, should be calculated using the length of the geogrid in the resistance zone as given in following equations. The FS value for pullout resistance is 1.5.

P_r = Pullout resistance (lbs/ft) per unit of reinforcement width
= $2F^*\gamma_r Z L_{ez}\alpha$

Where,

F^* = Pullout resistance factor = $0.8\tan\phi_r$
Z = Depth below top of wall (ft)
α = Scale effect correction factor = 0.8

$L_{ez} = L - L_{az} = L - (H - Z)\tan(90-\rho)$

φ = Inclination of internal failure surface from horizontal (degrees, °) = $45 + \dfrac{\phi_r}{2}$

σ_v = $\gamma_r Z + \Delta\sigma_v$

σ_h = $\sigma_v K_r + \Delta\,\sigma_h$

T_{max} = $\sigma_h\, S_V$

$$FS_{PO} = \frac{P_r}{T_{max}} \geq 1.5$$

Connection Strength

The geogrid is connected to the modular block units utilizing friction between the units and the geogrid. The connection strength (T_{ac}) is therefore specific to each unit/geogrid combination and must be determined by testing for each combination. Connection strength data is generally provided by the geogrid and/or block manufacturer. The connection strength values cannot be assumed if not available. The designer must refer to specific testing graphs for a particular combination of geogrid and block.

Connection strength is defined as the allowable load which can be applied to each soil reinforcement layer per unit width of reinforcement at the connection with the wall face as determined by laboratory pullout tests. The laboratory pull out test results should provide a graph of Normal load (N) vs Connection Capacity. T_{ac} can be estimated from these graphs for each layer of geogrid by estimating the normal load at the depth of the geogrid layer and checking the laboratory graph to determine the corresponding T_{ac} value. The T_{ac} value should always be limited to T_a or to the LTDS value provided by the geogrid manufacturer. The test results for various block/geogrid combinations can be found in Appendix B.

Where:

T_o = Reinforcement tensile load per unit width of wall at each layer of reinforcement (lbs/ft)

$$X = \frac{(H - Z)\left(\frac{1}{2}\tan\left(45 - \frac{\phi_r}{2}\right)\right)}{H}$$

$$1 - X = 1 - \frac{1}{2}\tan\left(45 - \frac{\phi_r}{2}\right)$$

$$\frac{T_o}{T_{max}} = 1 - X$$

As shown in Figure 21:

At the top of the wall:

$$\frac{T_o}{T_{max}} = 1 - \frac{1}{2}\tan\left(45 - \frac{\phi_r}{2}\right)$$

$$T_o = 1 - \frac{1}{2}\tan\left(45 - \frac{\phi_r}{2}\right)T_{max}$$

At the bottom of the wall:

$$T_o = T_{max}$$

Verify that:

$T_o \leq T_a$ and $T_o \leq T_{ac}$

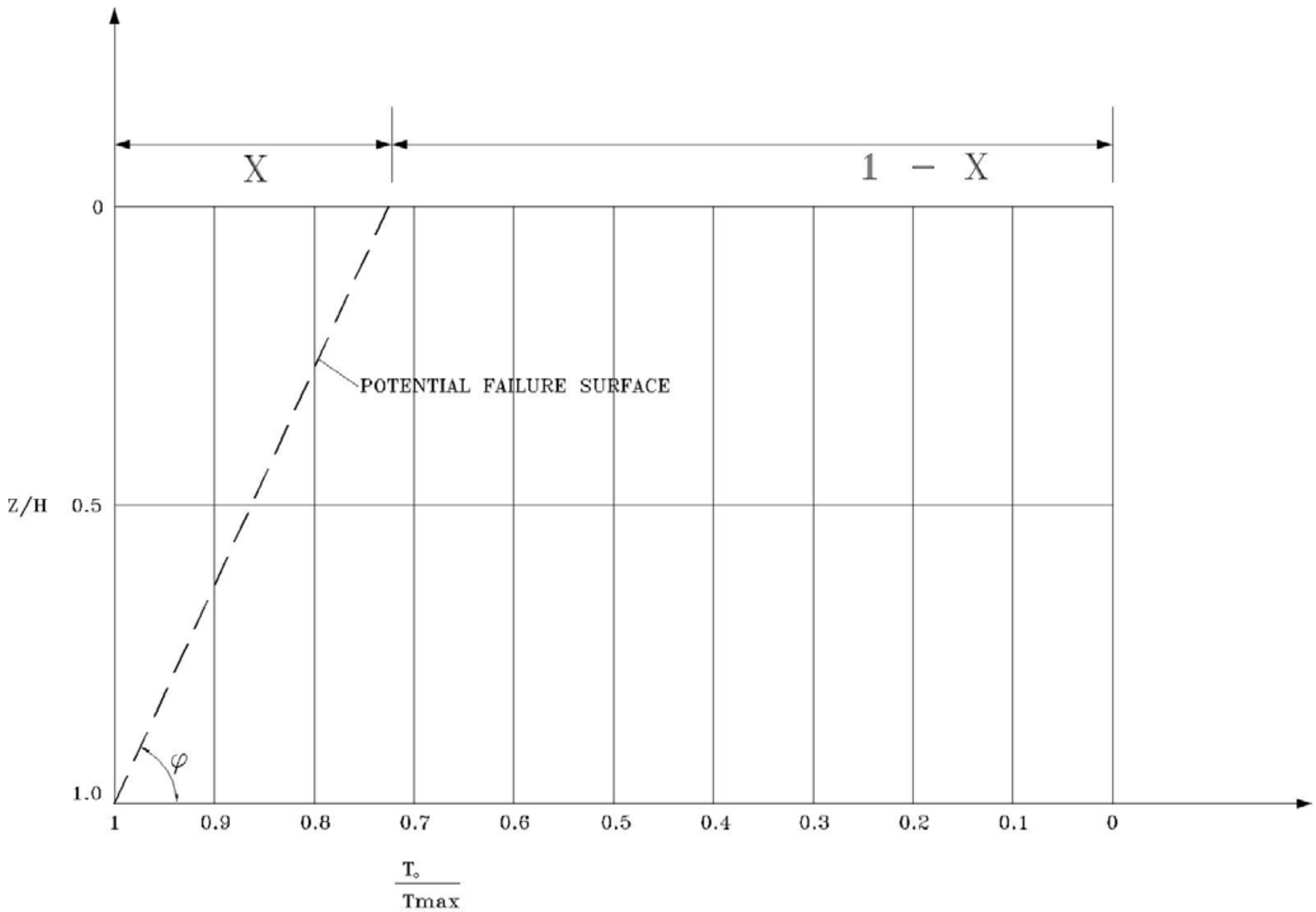


FIGURE 21 Estimation of Connection Tensile Strength

According to AASHTO and NCMA, the maximum allowable movement of the wall connection is ¾" to limit the deformation of the wall face. T_{ac} may be considered as the laboratory tested tensile load at ¾"displacement (connection capacity) for a particular combination of block and geogrid. This criterion is also referred to as the "serviceability" evaluation.

The serviceability evaluation is questionable due to inherent problems and difficulties associated with laboratory testing and data collection. Its applicability to the actual structural performance of the segmental wall-geogrid-reinforced soil system is not clear. In the practicing design community, there is no agreement on the amount of displacement to determine the best measurement of serviceability. The designer should refer to current design guidelines and use his or her best engineering judgment to come up with serviceability criterion to ensure that the designed MSE wall will remain serviceable for a given design life. Additionally, block movements should generally occur during compaction of soil behind each block, prior to placing of the next higher grid layer and block layer. The key integrated to StoneTerra™ blocks also limits sliding movement between blocks.

Local Block Stability

The bending of StoneTerra™ units generally does not occur because of the short length (4ft) of units, the continuous support provided by underlying units, and the interlocking effect provided by large built-in shear keys.

StoneTerra™ units are generally made of return concrete, which consistently shows a compressive strength of 2,200 psi (or greater). During the past several years of performance history, minimal to no surface cracking has been noted in exterior faces of StoneTerra™ walls due to temperature changes. A study performed by a materials engineer, Dr. D. R. Morgan, Ph.D., P. Eng. of HBT AGRA, in the Vancouver, B.C., Canada area, demonstrated that standard grade Lock Block units (constructed using the same methods) showed minimal deterioration after 7 or 8 years of exposure to the weather in that area. Therefore, neither flexural nor temperature reinforcement is necessary for individual block stability.

Bulging of blocks may occur due to excessive surcharge during construction. Therefore, the designer should include construction-related surcharge in the design or recommend a setback from the back face of the wall for heavy equipment.

SPECIAL DESIGN CONSIDERATIONS

The StoneTerra™ MSE wall may be designed under special site conditions such as surcharge loading, high groundwater levels, submerged wall conditions, and/or seismic conditions. These special conditions require modifications to the general design methodology (discussed in previous chapters) to ensure external, internal, and overall stability of the wall structure. The most common special site conditions and required modifications to the design methodology are included.

GROUNDWATER LEVEL

As discussed previously, groundwater level in some cases, hydrostatic pressure and seepage forces generated by groundwater may need to be considered. A qualified geotechnical engineer should be consulted to estimate these forces. Otherwise, to reduce hydrostatic pressures, a drainage system should be installed behind the wall. As shown in Figure 22, a drainage system should be chosen based on the groundwater level with respect to the base of the wall. The drain pipe should be a minimum of 4 inches in diameter and with a maximum spacing of 100 feet between cleanouts.

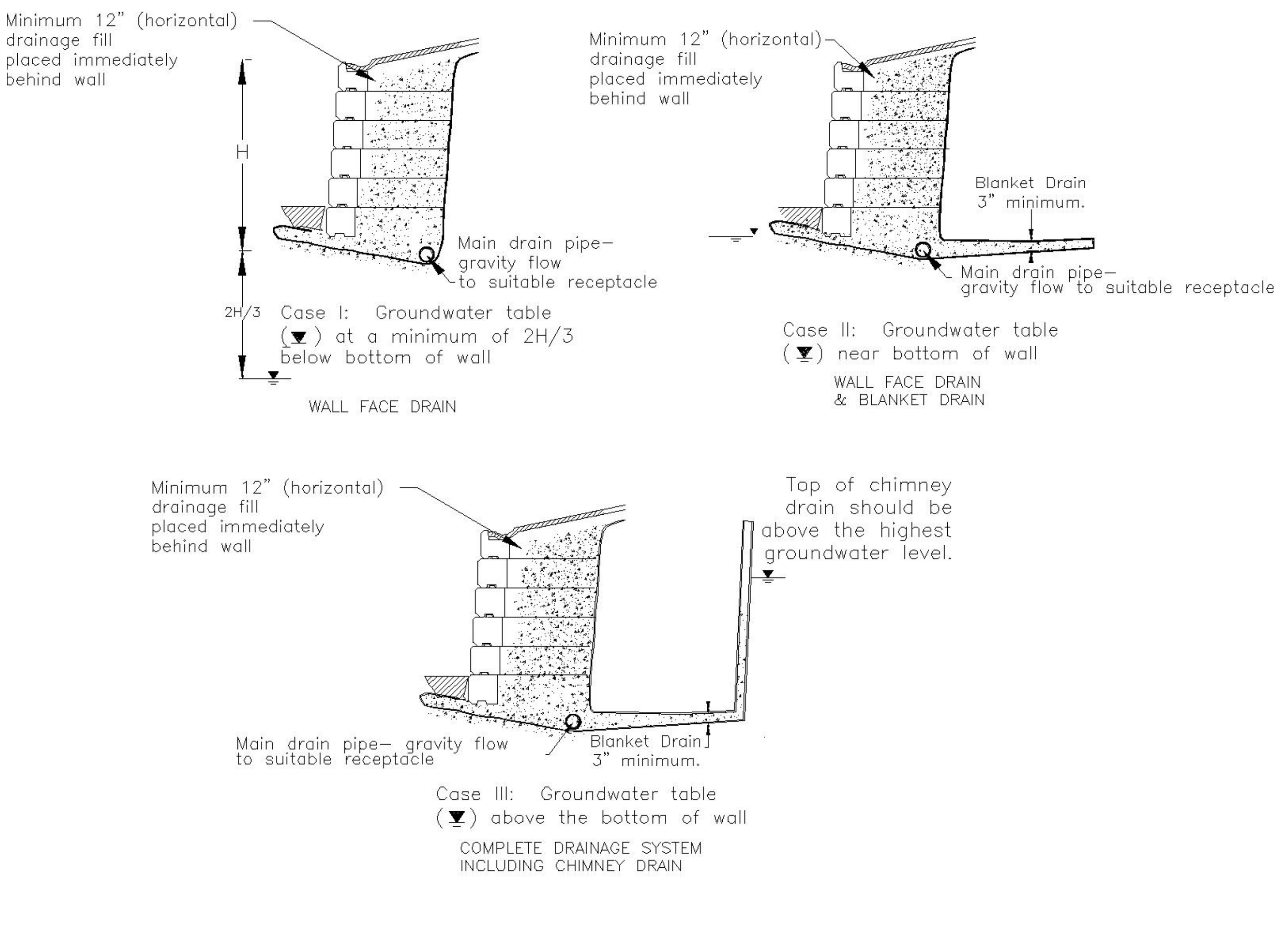


FIGURE 22 Typical Drainage Details

SUBMERGED WALL

If a StoneTerra™ wall is placed permanently or would be periodically submerged in a body of water, the following special precautions must be taken:

- The drainage system should be designed so that free flow of water occurs through the StoneTerra™ wall system. The gradation of drainage material should be such that pumping of fines does not occur through the wall units.

- Effective stress soil parameters, including buoyant soil unit weight, should be used for all soil zones including foundation soil, retained soil, and drainage fill, if applicable.

- The buoyant unit weight for submerged blocks should be used when determining resisting forces.

- Additional destabilizing forces due to hydrostatic pressure should be incorporated in both external and internal stability calculations, if applicable. The magnitude and location of hydrostatic pressure and seepage forces should be determined based upon the mean low water elevation and the maximum anticipated high water elevation.

- The overall (global slope) stability of the submerged StoneTerra™ wall system should be evaluated for the most severe combination of groundwater flow, water feature elevation, rapid drawdown, and any surcharge loading.

- Potential scour beneath the wall and the effects of any possible wave action must be considered.

- Structural durability or the effects of freeze/thaw and ice action on the StoneTerra™ wall units due to exposure to water should be evaluated.

A qualified geotechnical engineer should be consulted to evaluate submerged wall conditions and required modifications discussed above.

TIERED WALLS

StoneTerra™ walls are sometimes placed in a tiered (multi-level) arrangement to limit tier wall heights or to improve aesthetics. A typical cross section for a tiered wall can be found in Figure 23. In a tiered wall condition, the combined upper tier wall acts as a uniformly distributed dead load on lower tier walls.

The following guidelines should be used for designing tiered walls:

- The lower tier design wall height should be greater than the exposed vertical height of the upper wall

- If the horizontal distance (wall face to wall face) between two tiered walls is less than the design height of the lower wall (2H), a surcharge load should be considered on the lower wall. The intensity of surcharge load on the lower wall increases as the horizontal distance between upper and lower wall decreases.

- A surcharge loading of roughly $\gamma H'$ may be used on lower wall due to upper wall with exposed height H' and retaining backfill with unit weight of γ. This is a conservative surcharge approximation used to simplify the design. If accurate estimation of surcharge loading due to upper-tiered wall is necessary, then a qualified geotechnical engineer must be consulted.

- The maximum soil bearing pressure exerted by the lowest tier wall should be checked to make sure it does not exceed the given allowable soil bearing capacity.

- Overall (global slope) stability evaluation is extremely critical for tiered wall system. A qualified geotechnical engineer who is familiar with local soil conditions and practice should be hired for performing the global stability analyses.

- The design of tiered wall system becomes more complicated with an increased number of tiers. Therefore, engineering judgment should be used and the advice and assistance of local qualified geotechnical engineers should be considered before finalizing the design.

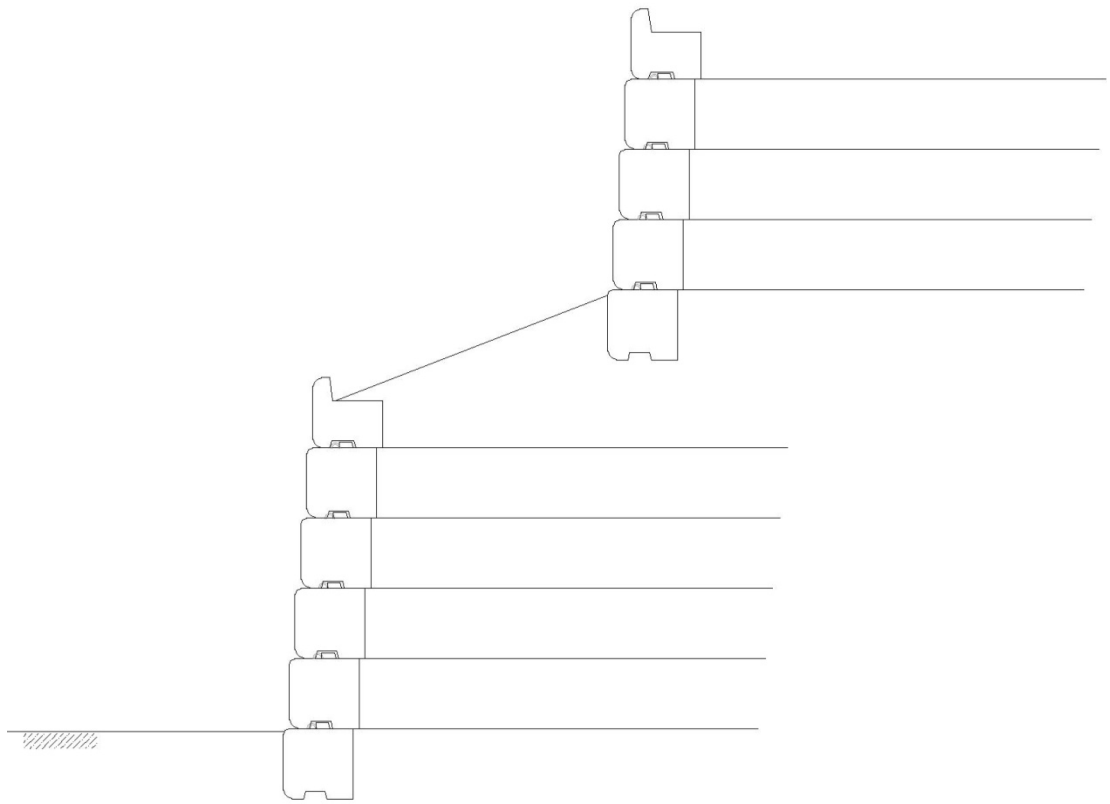


FIGURE 23 Tiered Wall Cross Section

SEISMIC CONDITIONS

External Stability

MSE walls have been found to be earthquake resistant due to their ability to yield during a major seismic event. Surveys conducted by CalTrans and others after recent earthquakes in California indicated that the MSE walls performed well under seismic loading. Seismic design of MSE walls in areas with significant earthquake activity usually results in an increase in grid length and strength relative to the static case. Under seismic conditions, a StoneTerra™ wall is subjected to horizontal inertial force (P_{IR}) and dynamic horizontal thrust (P_{AE}) in addition to the static earth pressures. The horizontal inertial force can be estimated by multiplying the weight of the retained soil mass ($0.5\gamma H^2$) by A_m, where A_m is the acceleration coefficient as described in following paragraphs. The dynamic horizontal thrust can be estimated using the pseudo-static approach presented by Mononobe-Okabe.

A seismic horizontal ground acceleration coefficient "A", is required to estimate the dynamic forces due to seismic activity. A qualified engineer should be consulted to obtain “A” value or local code books and seismic hazard maps may be used to estimate the preliminary value of “A.” The effects of vertical acceleration are generally omitted.

AASHTO recommends the use of a higher ground acceleration coefficient, A_m, to reflect retaining structure acceleration above ground level. This is a conservative approach that can often yield unrealistic/impractical designs for taller walls. A_m and the dynamic forces for horizontal backfill (β=0) and design wall height, H, can be estimated as follows:

$A_m = (1.45\text{-}A)A$
$P_{IR} = 0.5A_m\,\gamma_f\,H^2$
$P_{AE} = 0.375\,A_m\,\gamma_f\,H^2$

For walls with sloping backfill β>O:

$P_{AE} = 0.5\,A_m\,\gamma_f\,(H_2)^2$
$P_{ir} = 0.5\,A_m\,\gamma_f\,H_2H$

where,

$$H_w = H + \frac{(\tan\beta)(0.5H)}{1 - 0.5\tan\beta}$$

$P_{is} = 0.125\,A_m\,\gamma_f\,(H_2)^2\tan\beta$
$P_{IR} = P_{ir} + P_{is}$

According to AASHTO, stability computations should be made by considering, in addition to static forces, the horizontal inertial force (P_{IR}) and 50% of the dynamic horizontal thrust (P_{AE}). The horizontal inertial force (P_{IR}) should be applied at a height of 0.5H and the dynamic horizontal thrust (P_{AE}) should be applied at a height of 0.6H at the back of the wall above the wall base (see Figures 24 and 25). Under seismic conditions, the static FS values for sliding and overturning stability may be reduced by 75%.

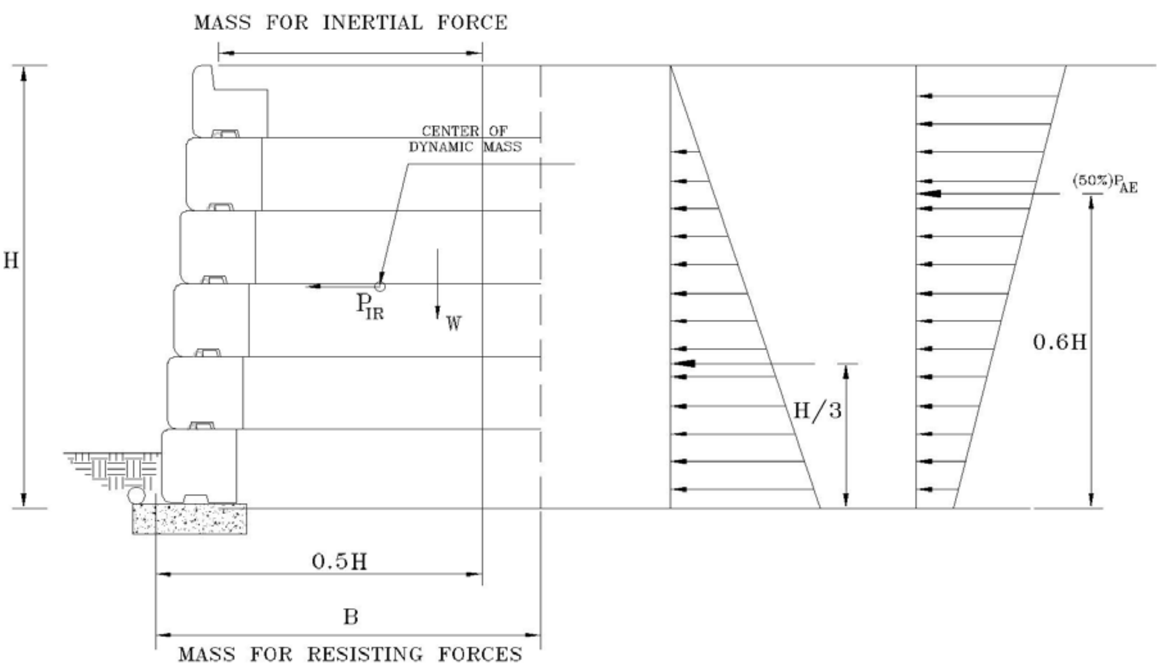


FIGURE 24 Seismic External Stability for Level Backfill

In addition to the sliding and overturning stability evaluation, the potential for liquefaction, lateral spread, and global slope failure must be analyzed by a qualified geotechnical engineer. For critical structures, a site specific seismic evaluation should be performed by a qualified geotechnical engineer to evaluate the potential for seismic hazards mentioned above.

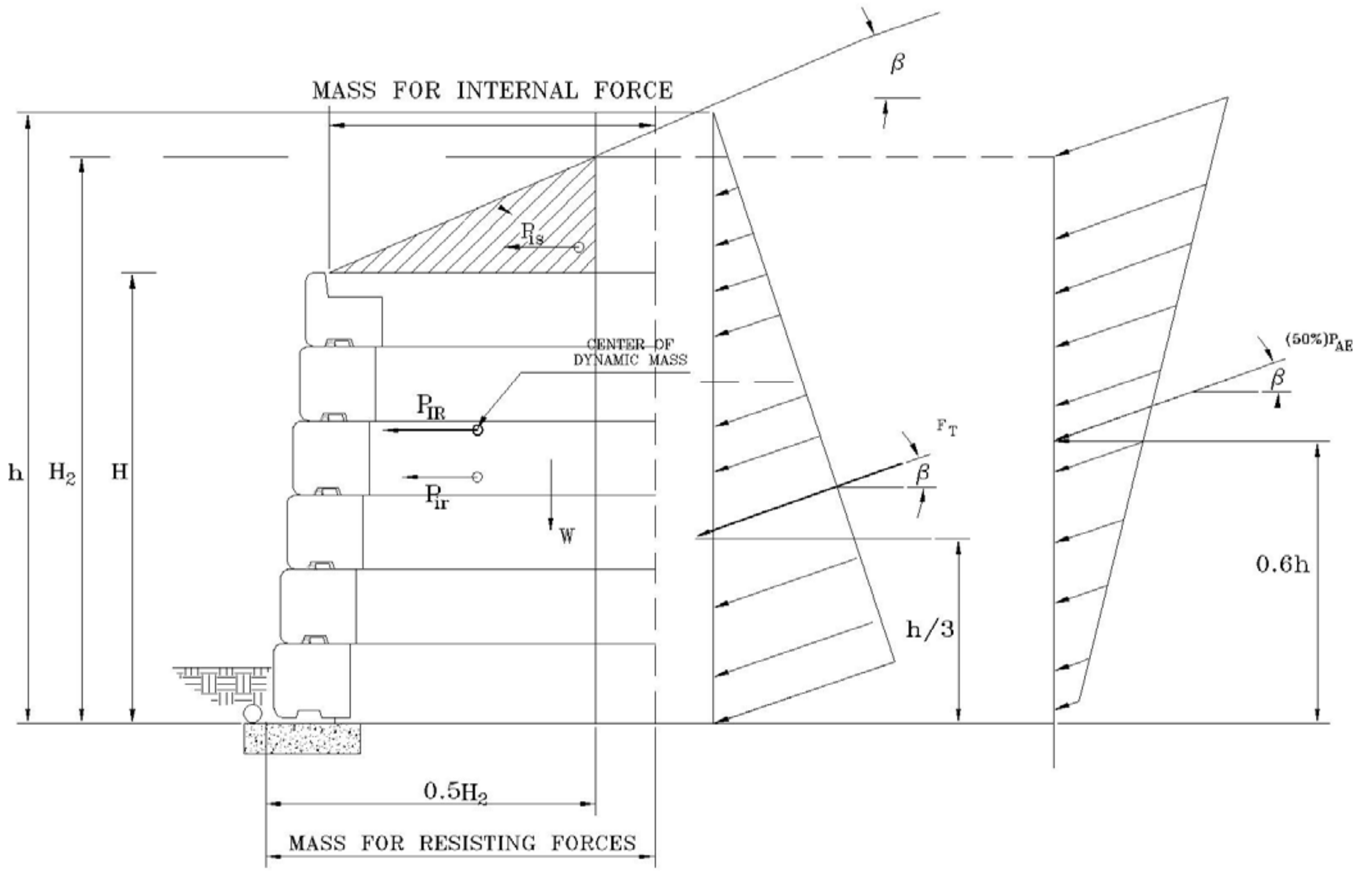


FIGURE 25 Seismic External Stability for Sloping Backfill

Internal Stability

Seismic loads produce an inertial force, P_I, acting horizontally, in addition to the existing static forces. P_I leads to incremental dynamic increases in the maximum tensile forces in the geogrid. The maximum tensile force line does not change during seismic loading. P_I can be calculated as follows:

$$P_I = A_m W_A$$

Where,

W_A = weight of the active zone

Calculate the dynamic increment, T_{md}, induced by P_I in the reinforcements:

$$T_{md} = P_I \left(\frac{L_{ezi}}{\sum_{i=1}^{n} (L_{ezi})} \right)$$

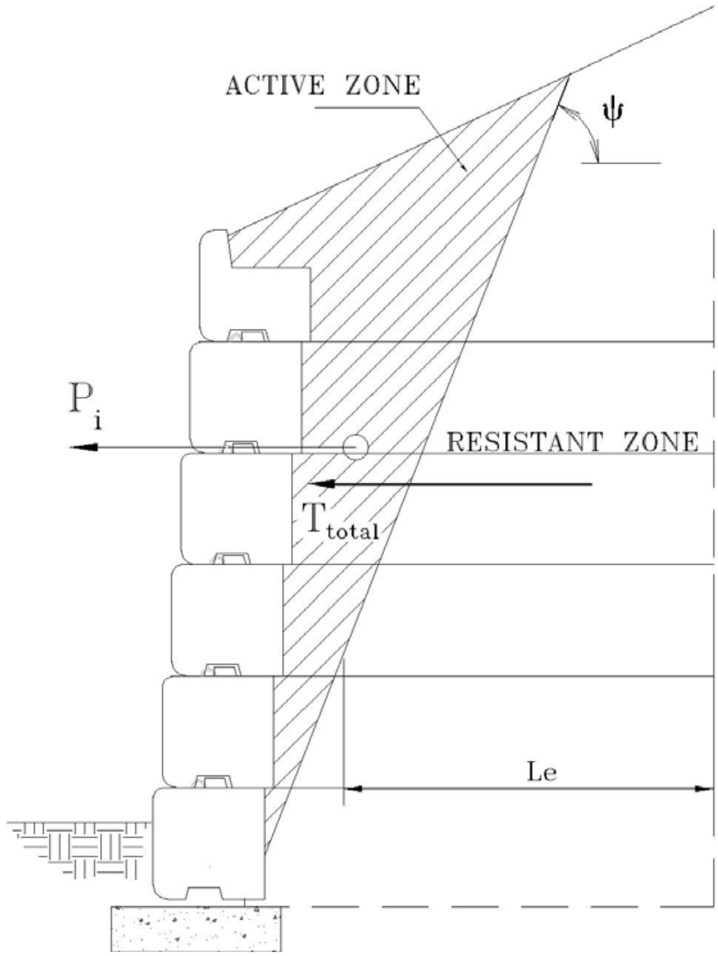


FIGURE 26 Seismic Internal Stability for Sloping Backfill

The reinforcement layers can then be designed for the total load per unit width of wall. The maximum tensile force then becomes:

$$T_{total} = T_{max} + T_{md}$$

For pullout, the value of F* can be reduced to 80% of the static F* value. In addition, the allowable strength of the geogrid reinforcement can be increased by eliminating the creep reduction factor. So,

$$T_{a_{seismic}} = (T_{a_{static}})(RF_{creep})$$

Under seismic conditions, the FS values for external and internal stability may be reduced to 75% of static values. See Design Example II in Appendix C.

In addition to the external and internal stability evaluation, the potential for liquefaction, lateral spread, and global slope failure must be analyzed by a qualified geotechnical engineer. For critical structures, a site-specific seismic evaluation should be performed by a qualified geotechnical engineer to evaluate the potential for seismic hazards mentioned above.

A retaining wall may be designed for seismic conditions if it is a part of a structure that can be classified as an essential facility, hazardous facility, major structure, or special occupancy structure. In addition, a retaining wall may be designed for seismic conditions if required by a building official.

LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

AASHTO and various state Departments of Transportation have recently adopted new design methodologies known as Load and Resistance Factor Design (LRFD). The essence of this method is instead of having a single safety factor applied to the ratio of the driving forces (tending to cause failure) and resisting forces at the end of an analysis, the safety factor is divided into Load Factors and Resistance Factors which are applied to their respective sides of the analytical problem. At the end of the analysis, the factored resisting forces must exceed the factored driving forces.

Load and resistance factors are assigned by the geotechnical engineer based on factors related to the particular portion of the design elements, including the level of uncertainty associated with the parameters in an analysis and the consequences of the failure of an element, among other factors. This is similar to the standard allowable stress design (ASD) practice where a FS of 2.5 or 3 is applied to the ultimate bearing capacity, a FS of 2 is used for wall overturning, and a FS of 1.5 is applied to wall sliding. All three values are reduced by 25% for seismic analyses. Since the allowable values have proven effective in practice, the geotechnical engineer should always make certain that a LRFD design does not differ significantly from the proven design methods. As more experience is gained with LRFD, the approaches may be adjusted.

The AASHTO LRFD Bridge Design Specifications specify a range of suggested factors for various design loads and resistances. Most of these factors are primarily used in bridge design and have little or no application to a gravity retaining wall design.

In keeping with the proven values discussed above, the following values are applicable to and suggested for gravity design of StoneTerra™ MSE walls:

Component	AASHTO Range	Suggested Factor
Active Earth Pressure (EH)	0.90 – 1.50	1.35
Vertical Earth Pressure (EV)	1.00 – 1.35	1.00
Earth Surcharge (ES)	0.75 – 1.50	1.00
Bearing Resistance, clay	0.50	0.50
Bearing Resistance, sand	0.45	0.45
Sliding Resistance, precast concrete	0.90	0.90
Geosynthetic Resistance, static	0.90	0.90
Geosynthetic Resistance, seismic	1.20	1.20

TABLE 2. LRFD Load and Resistance Factors

The use of these LRFD factors should yield solutions similar to the ASD method for sliding analyses. Comparison of the two methods for overturning analyses is not as straightforward and the authors suggest that both ASD and LRFD analyses be used whenever the LRFD procedure is required.

An example of the use of LRFD is provided for a simple case as Design Example III in Appendix C.

SAMPLE CONSTRUCTION AND MATERIAL SPECIFICATIONS

The following paragraphs provide general guidelines on developing construction and material specifications for specific projects. Generally, the owner would hire services of a designer to perform a project-specific design. The design should be performed in accordance with the procedures outlined in this manual.

The general guidelines given below should be modified to incorporate specific StoneTerra™ facing unit criteria and special project-specific requirements and to provide consistency with construction drawings. Any unnecessary requirements given below may be deleted depending upon project details.

The physical and mechanical properties of StoneTerra™ facing units and geogrid reinforcement properties are important in providing satisfactory long-term performance of the StoneTerra™ MSE wall systems.

The AIA A201-87, CSI 3-Part Format and CSI Page Format were utilized to provide the general conditions and formats.

PART 1: GENERAL

I. Summary Description

A. This section includes StoneTerra™ MSE retaining wall systems consisting of geosynthetic reinforcement (high tensile strength polymeric sheet materials called geogrids) placed between two horizontal layers of StoneTerra™ Segmental Concrete facing Units and unifying compacted soil backfill. Work shall consist of furnishing all materials, labor, equipment, field supervision, and installing a StoneTerra™ MSE wall system in accordance with given specifications. All installations should conform to project drawings provided by the Owner or the Owner's Engineer.

B. Related Sections
   1. Section ________ - Site Preparation
   2. Section ________ - Earthwork

II. Reference Standards

A. Any reference standards that are not applicable to the project should be deleted. If there is a conflict between the given specifications and reference standards, the Owner's Engineer should make the final determination of applicable documents.

B. American Association of State Highway and Transportation Officials (AASHTO)
   1. LRFD Design Specifications, SI Units, 1st Edition, 2007
   2 Task Force 27 Report, In-situ Soil Improvement Techniques, Design Guidelines for Use of Extensible Reinforcements for Mechanically Stabilized Earth Walls in Permanent Applications, 1990
   3. Standard Specifications for Transportation Materials and Methods of Sampling and Testing, 1986

C. American Society for Testing and Materials (ASTM)
   1. ASTM D 422 – Standard Test Method for Gradation of soils
   2. ASTM D 424 – Standard Test Method for Determination of Atterberg Limits of Soils
   3. ASTM DG51 – Standard Test Method for Determination of Soil pH
   4. ASTM D698, D1997 – Standard Specification for Moisture Density Relationship for Soils, Standard Proctor and Modified Proctor Methods
   5. ASTM C33-99 - Standard Specification for Concrete Aggregates
   6. ASTM D3034 – Standard Specification for Polyvinyl Chloride (PVC) Plastic Pipe

7. ASTM D1248 – Standard Specification for Corrugated Plastic Pipe
8. ASTM C150 – Use of Cement in Concrete Blocks
9. ASTM C260 – Use of Admixtures in Concrete
10. ASTM C494 – Use of Admixtures in Concrete
11. ASTM C805 – Schmidt Hammer Test for the determination of Compressive Strength of Concrete

D. Geosynthetic Research Institute (GRI)
1. GRI GG1-87 - Standard of Practice for Geogrid Rib Tensile Strength
2. GRI GG2-87 - Standard of Practice for Geogrid Junction Strength
3. GRI GG5-87 – Standard of Practice for Geogrid Pullout
4. GRI GS-6-88 – Standard of Practice for Interface Friction Determination by Direct Shear
5. GRI GG4A-91- Determination of Long-Term Design Strength of Stiff Geogrids
6. GRI GG4B-91- Determination of Long-Term Design Strength of Flexible Geogrids
7. GRI GT7-92 - Determination of Long-Term Design Strength of Geotextiles
8. GRI GG-3A &B-91 - Tension Creep Testing of Stiff & Flexible Geogrids
9. GRI GT5-92 - Standard of Practice for Tension Creep Testing of Geotextiles

E. National Concrete Masonry Association (NCMA)
1. TEK 2-4A - Specification for Segmental Retaining Wall Units

F. StoneTerra, Inc.
1. Design Manual for StoneTerra™ MSE Wall Systems, June 2008

III. Definitions

A. StoneTerra™ Unit – A segmental concrete unit with shear keys generally made of "surplus concrete" in the yard of ready mix concrete supplier.

B. Gravity Soil Mass - Compacted structural fill placed immediately behind the wall, which contributes to the gravity mass of the wall structure.

C. Drainage Fill - Free-draining, well-graded, coarse-grained aggregates placed immediately behind the blocks to relieve hydrostatic pressures or seepage forces and to prevent clogging of aggregate drainage medium if a geotextile fabric is not used.

D. Retained Soil - Native soils or compacted structural fill situated immediately behind drainage fill. The primary function of the gravity wall is to retain this soil mass without failure.

E. Leveling Pad / Wall Foundation **-** Densely compacted and free draining crushed rock pad for distributing the weight of block wall over a wider area and for providing a working surface during construction.

F. Foundation Subgrade - Competent native soil subgrade or compacted structural fill subgrade for supporting the block wall structure as approved by a qualified geotechnical engineer.

G. Drainage / Discharge Pipe **-** Perforated pipe with adequate flow capacity placed at the rear base of the wall to discharge collected water into a suitable receptacle by gravity flow. Location of discharge pipe behind the wall depends upon the drainage requirements of the wall structure and the design of drainage system.

H. Drainage Swale - A small depression adjacent to the top of the wall to collect surface water run-off and discharge by gravity flow.

I. Geotextile filter – A filter fabric (with adequate permittivity or porosity) placed against the retained soil mass or between drainage media and retained soil mass to minimize clogging of drainage media.

J. Backslope - Retained soil slope behind the wall. Slope inclination, β, is measured counterclockwise from the horizontal plane.

K. Foreslope / Toeslope - Downslope in front of the toe of wall.

L. Reinforced Backfill – Compacted structural fill placed behind the Drainage Fill or directly behind the StoneTerra™ units as outlined on the plans.

M. GeoSynthetic Reinforcement – High tensile strength polymeric sheet materials called geogrids or geotextiles manufactured for soil reinforcement purposes.

N. Geogrid Ultimate Tensile Strength – Breaking tensile strength when tested in accordance with GRI-GG1, as modified by AASHTO Standard Specification for Highway Bridges 1999 Interim, using a single rib having the greater of 3 junctions or 8 inches and tested at a strain rate of 10 percent per minute based on this gauge length. Values shown are minimum average roll values.

O. Geogrid Junction Strength – Breaking tensile strength of junctions when tested in accordance with GRI-GG2, as modified by AASHTO Standard Specification for Highway Bridges, 1999 Interim, using a single rib having the greater of 3 junctions or 8 inches and tested at a strain rate of 10 percent per minute based on this gauge length. Values shown are minimum average roll values.

P. Geogrid Long-Term Design Strength – The maximum allowable stress level of the polymeric grid used in the internal stability design calculations of the retaining wall. Ultimate Tensile Strength reduced by the effects of installation damage and durability.

Q. Geogrid Long-Term Allowable Design Strength – Long-Term Design Strength reduced by the Factor of Safety for design uncertainties.

IV. System Description

A. Design Requirements - Design the retaining wall system in accordance with the design guidelines presented in the StoneTerra™ MSE Wall Design Manual, 1st Edition, June 2009 provided by StoneTerra, Inc. Engage and pay for the services of a Designer to design and develop Design Data for the retaining wall system.

B. Performance Requirements – The contractors, material suppliers, and wall system suppliers shall have sufficient past project experience and shall be approved by the Owner's Engineer at least two weeks prior to the bid opening.

V. Submittals

The following submittals shall be made 30 days prior to the start of construction. In addition, the contractor shall provide a list of successfully completed projects along with related project references.

A. Geotechnical Reports – A geotechnical report prepared in accordance with local regulatory and industry standards shall be submitted for wall areas including any required slope stability analyses results.

B. Product Data – Manufacturer's materials specifications, installation instructions, and general recommendations. The geogrid manufacturer shall submit all strength testing data and results including connection strength test data for StoneTerra™ geogrid combination.

C. Plans – Engineering drawings, cross-sections, elevations, and large-scale details of elevation, typical sections, details, and connections. Plans shall be stamped and signed by a registered and qualified professional engineer in the State of ______________.

D. Quality Control and Certification Submittals - Design calculations and plans for the retaining wall system. All design data shall be stamped by the Designer. The designer shall be a registered and qualified professional engineer in the State of _______________. All certifications regarding ultimate and junction/connection strengths for the specified geogrids shall be submitted by the contractor.

VI. Quality Assurance

A. Pre-Construction Meeting – A meeting between the geotechnical engineer, the wall designer, the contractor, the material supplier, subcontractors, and the owner shall be held at the site in order to review the retaining wall design and construction requirements. A notification shall be sent to all the parties at least three 3 days in advance of the time of the meeting.

B. Designer – The firm designing the wall shall have liability insurance (Errors and Omissions) of at least $1,000,000.00 per occurrence. The designer shall be a registered professional engineer, registered in the state where the project is located.

VII. Delivery, Storage, and Handling

A. At the time of delivery, the contractor shall inspect and confirm proper type and grade of materials. All product specifications shall be reviewed to assure that all specified materials have been delivered.

B. The contractor shall store and handle all materials in accordance with manufacturer's recommendations. The contractor shall avoid excessive mud, wet concrete, epoxy, or other deleterious materials from coming in contact with and affixing to materials. Geogrids shall be stored at temperatures above – 20 degrees F (-29 degrees C). Rolled materials may be laid flat or stood on end. No damage shall occur to stored materials

C. The contractor shall discard all damaged materials and not use them in wall construction.

PART 2: PRODUCTS

I. Manufacturers

A. StoneTerra, Inc. is the sole manufacturer of StoneTerra™ Segmental Concrete Facing Units.

B. Geogrids shall be manufactured by_________________.

C. Substitutions – See section _______________________.

II. Materials

A. StoneTerra™ Segmental Unit

1. The StoneTerra™ units shall have minimum 28-day compressive strength of 2,200 psi. Higher concrete compressive strength can result in higher cost for the blocks. The maximum absorption of 10 pcf and adequate freeze-thaw protection (absorption by weight = 6%) shall, in general, satisfy the listed ASTM standards (C140 and C1262).

2. All individual StoneTerra™ units shall be free of cracks and other defects that would interfere with the placement and locking of units. Specifically, all shear keys shall be free of any damage.

3. StoneTerra™ unit dimensions such as height, width, depth, and batter shall match the details shown on the approved plans. A tolerance of ±1/2 inch for all dimensions except height may be used. A tolerance of ±1/4 inch shall be used for height.

4. Architectural features such as surficial finishes and the color of StoneTerra™ units shall match the details shown on the approved plans.

5. StoneTerra™ units shall have following dimensions (select whatever is applicable). Color application can result in higher cost for the blocks. Generally the blocks are various shades of gray.

    1. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color
    2. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color
    3. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color
    4. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color
    5. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color
    6. StoneTerra™ _________________
        a. Size: b. Weight:
        c. Color

B. Geosynthetic Reinforcements

1. The following geogrids shall be used for soil reinforcement. The type, design strengths, and placement locations shall match details shown on plans.

2. The allowable tensile strength shall be calculated in accordance with guidelines given in Design Manual for StoneTerra™ MSE Wall Systems.
   1. _________________
      a. Ultimate Tensile Strength : _____lb/ft. minimum average roll value.
      b. Allowable Tensile Strength: _____lb/ft. minimum average roll value.
      c. Junction Strength : ____________lb/ft.
      d. Coefficient of Direct Sliding:_______
      e. Coefficient of Interaction:__________
   2. _________________
      a. Ultimate Tensile Strength : _____lb/ft. minimum average roll value.
      b. Allowable Tensile Strength: _____lb/ft. minimum average roll value.
      c. Junction Strength : ____________lb/ft.
      d. Coefficient of Direct Sliding:_______
      e. Coefficient of Interaction:__________
   3. _________________
      a. Ultimate Tensile Strength : _____lb/ft. minimum average roll value.
      b. Allowable Tensile Strength: _____lb/ft. minimum average roll value.
      c. Junction Strength : ____________lb/ft.
      d. Coefficient of Direct Sliding:_______
      e. Coefficient of Interaction:__________

3. Connection strength data shall be part of submittals as stated in Part V.

C. Drainage Materials

1. Drainage fill materials shall consist of free draining, all-weather, coarse-grained material which is placed behind the StoneTerra™ units as specified on the plans. The drainage fill gradation shall be as follows as determined by the ASTM D 422 test procedure:

   100 to 75% passing in a 1-in. sieve
   50 to 75% passing a 3/4-in. sieve
   0 to 60% passing a No 4 sieve
   0 to 50% passing a No 40 sieve
   0 to 5% passing a No 200 sieve

2. The Engineer and/or Architect may specify a substitute such as a drainage composite other equivalent geosynthetic drainage materials to be approved by the designer. The drainage composite shall be – 6 oz. per sq.yd. polypropylene non-woven geotextile, AASHTO M288-96, Class 2, bonded to both sides of a polyethylene net structure, produced by ___________________. Minimum Allowable Transmissivity – Not less than 1.5 gallon per minute per foot of width when tested in accordance with ASTM D4716-95 at a confirming pressure of 10,000 psf.

3. The drainage collection pipe shall be installed as shown on the plans. The pipe shall be a perforated or slotted, PVC or corrugated HDPE pipe. The pipe shall be wrapped in filter fabric. The pipe shall be manufactured in accordance with ASTM D3034.

D. Reinforced Backfill Materials

1. Reinforcement backfill materials shall consist of granular materials (GP, GW, SW, SP, SM) meeting the following gradation as determined by ASTM D 422 test procedure:
   100 to 75% passing in a 2-in. sieve
   100 to 75% passing a 3/4-in. sieve
   100 to 20% passing a No 4 sieve
   0 to 60% passing a No 40 sieve
   0 to 35% passing a No 200 sieve

2. The maximum aggregate size shall be limited to ¾ inch unless appropriate values for geogrid installation damage have been used.

3. The plasticity index of materials passing No. 200 sieve shall be less than 20.

4. The pH value shall be in the range of 2 to 12 as determined by ASTM G51 procedure.

E. Accessories

1. Geotextile Filter Fabric– A polypropylene non-woven geotextile produced by _______ or equal as approved by the designer with grab tensile strength (ASTM D4632) of ______lb/ft and water flow rate (ASTM D4491) of________.

2. Erosion Control Blanket – The StoneTerra™ MSE wall designer must include a reinforced, polymeric, permanent erosion control blanket on all soil structure/slope facings behind, in front, and adjacent to the retaining walls. All components shall be inert to chemicals normally encountered in a natural soil environment. The tensile strength shall be not less than__________ (ASTM D5035-95). The durability criteria shall include retaining a minimum of 80 percent of strength after 1,000 hours of ultraviolet exposure (ASTM D4355-92).

PART 3: CONSTRUCTION

I. Qualifications

The contractor and the site supervisor shall have successful completed several projects including the installation of StoneTerra™ MSE wall systems. The contractor shall carry adequate insurance and bond.

II. Excavation

A. Prior to the beginning of excavation, a StoneTerra™ supplier's representative experienced in StoneTerra™ wall construction shall assist the contractor regarding wall foundation excavation, specifically the preparation of foundation subgrade for design wall batter and other excavation procedures related to subgrade preparation, placement of blocks, and installation of the drainage envelope behind the wall.

B. The contractor shall provide adequate excavation support during construction in accordance with local, state, and federal safety regulations. It shall be the contractor's responsibility to ensure site safety during excavation and other construction activities.

C. The subgrade shall be excavated to meet design requirements shown on grading plans. Excavations shall be made vertically to the plan elevation and horizontally to the designed geogrid lengths so that over-excavation is minimized. Width of excavation should allow for wall base and drainpipe.

D. Start excavation at the lowest wall level. If the wall steps up in one block height, the base block should be installed at the lowest level in order to establish grade and face location of the second level.

E. Overexcavated or filled areas shall be well compacted, observed, tested, and approved by a qualified geotechnical engineer.

F. A qualified geotechnical engineer shall evaluate and approve excavated materials that are used as backfill in the reinforcement zone. All backfill materials shall be protected from the weather.

III. Foundation Preparation

A. The foundation trench shall be excavated to the dimensions indicated on the construction drawings.

B. A qualified geotechnical engineer shall inspect and approve the reinforced zone and leveling pad foundation soil subgrade in order to ensure adequate bearing capacity. Subgrade soil areas not meeting required bearing strength shall be marked in the field and the contractor shall remove and replace these areas with approved fill materials.

C. Foundation subgrade soils and any backfill materials shall be compacted to a minimum of 95% of Standard Proctor Dry Density in accordance with ASTM D698 before placing the leveling pad.

IV. Leveling Pad Installation

A. The leveling pad shall consist of a minimum of 6 inch thick layer of ¾-inch minus well-graded aggregate compacted to 95% of ASTM D 1557 modified proctor density, unless specified otherwise by the design engineer.

B. A StoneTerra™ supplier's representative experienced in StoneTerra™ wall construction shall assist the contractor regarding leveling pad preparation for achieving specified wall batter. The wall designer shall inspect and approve the leveling pad prior to the placement of blocks.
The cost of the supplier representative and wall designer inspection shall be included in the installers bid.

C. As a minimum, start at the lowest wall level, locate the front face of the wall, run a string about 1 inch in front and 2 inches above the base. Use 2x6 or 2x8 and steel stakes to make a form for achieving design batter. Set the front board in line with the string at base elevation of the wall. Locate and place the backboard at a distance equal to the base width of the wall. Set elevation of backboard so that design batter can be achieved. Without moving the string line, start leap-frogging the boards in line with the string and move forward along the length of the wall. It is best to prepare the entire leveling pad before installing the blocks.

V. Unit/Block Installation

A. A backhoe is the ideal equipment for block installation. A wire rigging with swivel hooks, OSHA approved and rated for weight of the blocks can be attached to the bucket and used for lifting, moving, and placing the blocks.

B. The contractor shall carefully place the first course of StoneTerra™ units only after the leveling pad has been approved by the designer for adequate batter.

C. Block placement should start at the lowest elevation.

D. The StoneTerra™ units shall be free of all protrusions and debris before installing the next course of units and/or placing the geogrid materials.

E. At the completion of the placement of each course, a string line shall be pulled to confirm that the walls geometry is being maintained.

VII. Drainage Fill, Unit Fill, and Drainage Pipe Placement

A. The StoneTerra™ units do not require core fill since there are no voids.

B. The drainage backfill shall be placed within an envelope of 12 inches behind the wall and shall consist of a free draining, coarse-grained granular materials, or open graded materials meeting the requirements of Section 2.II.C.1 unless specified otherwise by the designer.

C. The drainage collection pipe (minimum 4-inch diameter) shall be placed immediately behind the wall at the bottom of the wall with a minimum of 1.5% gradient to maintain a positive gravity flow into a suitable receptacle unless specified otherwise by the designer.

VIII. Reinforced Backfill Placement

A. As shown on the plans, the reinforced backfill material shall be placed in maximum lifts of 10 inches and shall be compacted to a minimum 95% Standard Proctor Dry Density in accordance with ASTM D698.

B. Only hand-operated compaction equipment shall be used within 5 feet of the back face of the StoneTerra™ units. This area shall be compacted to a minimum 90% of Standard Proctor Dry Density in accordance with ASTM D698-98.

C. Soil density testing shall not be performed within 5 feet of the tail of the StoneTerra™ Segmental Concrete facing units.

D. The backfill shall be smooth and level so that the geogrid can be placed tight and in a horizontal plane. At least 6 inches of material shall be placed over the geogrid prior to operating the tracked equipment. Swift turning and high speed of heavy equipment shall be minimized to avoid fill displacement and damage to geogrids. A maximum speed of 10mph shall be used for all heavy vehicles.

E. The excavated trench area in front of the toe of the wall shall be filled and compacted as the wall is being constructed.

F. The fill areas shall be graded or protected so that any surface water run-off is directed away from the wall face.

IX. Geogrid Installation

A. Geogrids shall be installed in accordance with manufacturer's recommendations at the design elevations and orientations as shown on the plans. The designer shall inspect on-site geogrid materials and approve it for installation.

B. Adequate tension shall be applied to geogrids and then staked on a well-prepared horizontal fill subgrade approved by a qualified geotechnical engineer for adequate compaction. The tension shall be maintained until at least 6 inches of fill is placed over the geogrid.

C. The Geogrids shall be oriented so that the design strength direction is perpendicular to the vertical plane consisting of the wall face. Geogrids shall not be overlapped in the design direction. Overlapping in the lateral direction shall be in accordance with the plans.

D. Damaged geogrids shall not be used unless approved by the designer.

X. Tolerance

Wall batter tolerance of ±1/8 in. per ft. maximum shall be allowed.

PART 4: MEASUREMENT AND PAYMENT

I. One vertical square foot of front wall surface shall be used as a unit of measurement. The front wall surface shall be considered from the top of the leveling pad (exposed bottom of wall) to the top of the wall or cap block in-place. The total quantity to be paid shall include all costs for material supply and installation.

II. Any over excavation of unsuitable materials and backfilling as directed and approved by the project geotechnical engineer and the owner shall be paid separately.

III. The quantities as shown on the plans or as approved by the designer shall be used to determine and confirm the in-place constructed wall area.

END OF SECTION

APPENDIX A

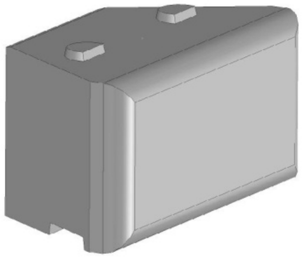
TYPICAL MODULAR UNITS

STONETERRA™ WALL SYSTEM

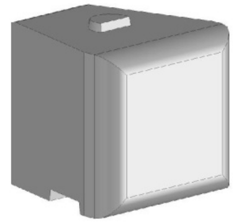
MORE POSSIBILITIES - LESS EFFORT

Main office: 815 NE 172nd Avenue, Vancouver, WA 98685
360-694-0141 / 800-377-3877 / 360-694-0281 fax

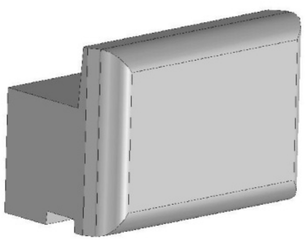
www.stoneterra.net



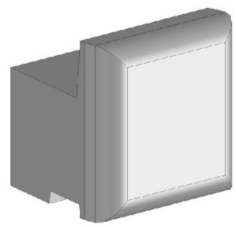
FULL
2' x 2' x 4' Face
1750 lbs.



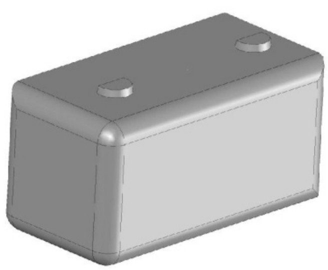
HALF
2' x 2' x 2' Face
905 lbs.



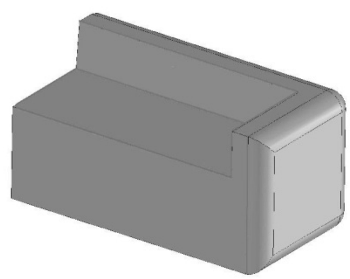
FULL BENCH
2' x 2' x 4' Face
1375 lbs.



HALF BENCH
2' x 2' x 2' Face
695 lbs.



FULL CORNER
Right/Left Face
2300 lbs.



FULL BENCH CORNER
Right/Left Face
1800 lbs.

APPENDIX B

GEOGRID INFORMATION

STRATAGRID 350

Table 1:

Test program:
Connection capacity results for StoneTerra segmental concrete block units with Stratagrid 350

Test number	approximate wall height (feet)	approximate number of blocks	normal load (lb/ft)	connection strength at 3/4 inch displacement (lb/ft)	peak connection strength (lb/ft)
1	6.4	12.7	3248	630	1462
2	10.3	20.6	5245	628	1779
3	6.2	12.4	3166	730	1656
4	3.7	7.4	1877	714	1390
5	6.4	12.8	3252	733	1543
6	2.8	5.7	1448	627	1063
7	9.1	18.2	4642	651	1211
8	7.8	15.7	3995	710	1253
9	7.7	15.4	3926	580	1272

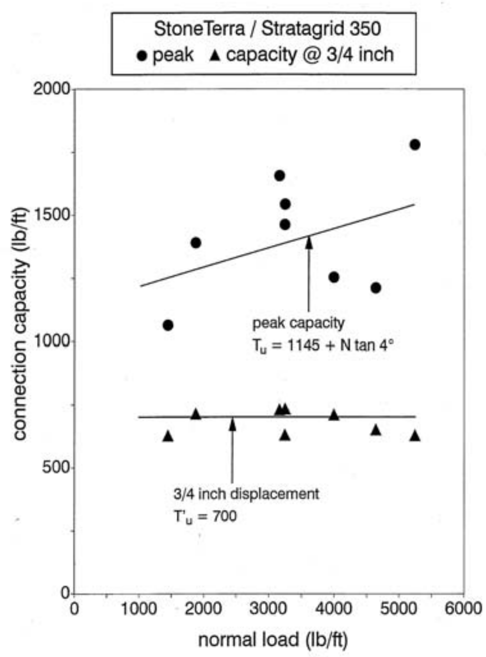


Figure 2: StoneTerra block / Stratagrid 350 connection capacity test results

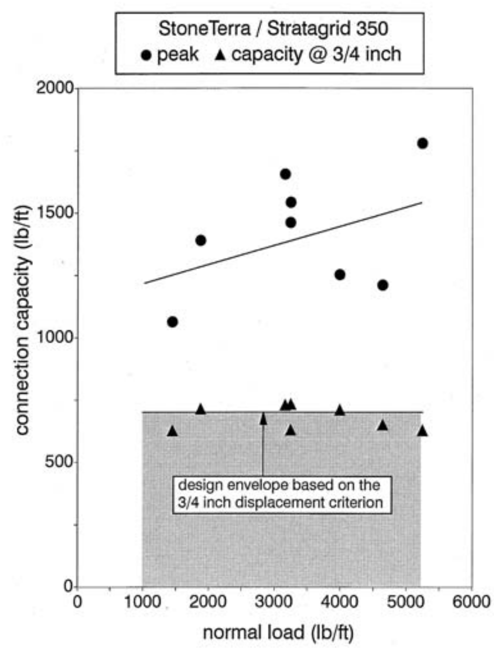


Figure 3: Preliminary design capacity envelope for StoneTerra block / Stratagrid 350 combination

STRATAGRID 550

Table 1:

Test program:
Connection capacity results for StoneTerra segmental concrete block units with Stratagrid 550

Test number	approximate wall height (feet)	approximate number of blocks	normal load (lb/ft)	connection strength at 3/4 inch displacement (lb/ft)	peak connection strength (lb/ft)
1	6.4	3.2	1644	1022	2046
2	10.0	5.0	2565	1155	2222
3	13.8	6.9	3519	989	2194
4	17.6	8.8	4497	1429	2236
5	21.7	10.8	5531	1396	2297
6	14.1	7.0	3592	1331	1956
7	13.9	6.9	3543	1094	2112
8	13.9	7.0	3551	1131	1893
9	25.6	12.8	6517	1376	2193

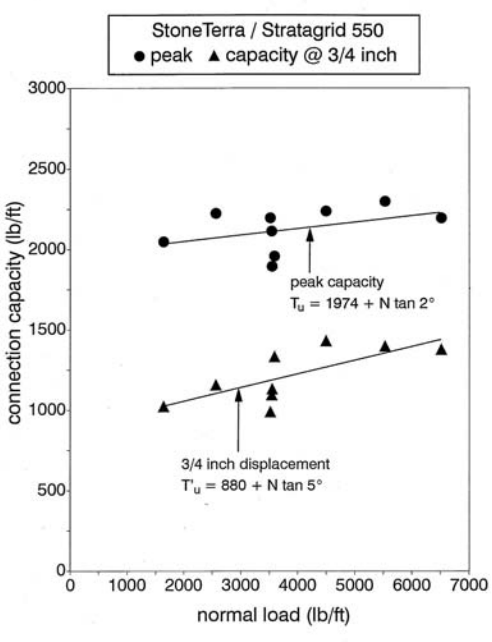


Figure 2: StoneTerra block / Stratagrid 550 connection capacity test results

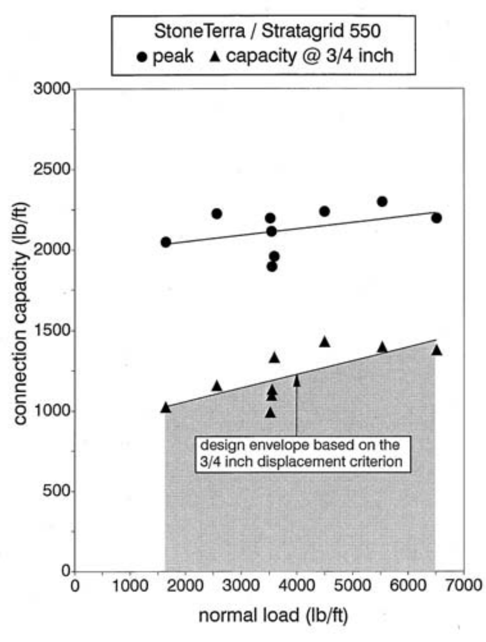


Figure 3: Preliminary design capacity envelope for StoneTerra block / Stratagrid 550 combination

APPENDIX C

DESIGN EXAMPLES

DESIGN EXAMPLE I

DETERMINATION OF STRUCTURAL DIMENSIONS

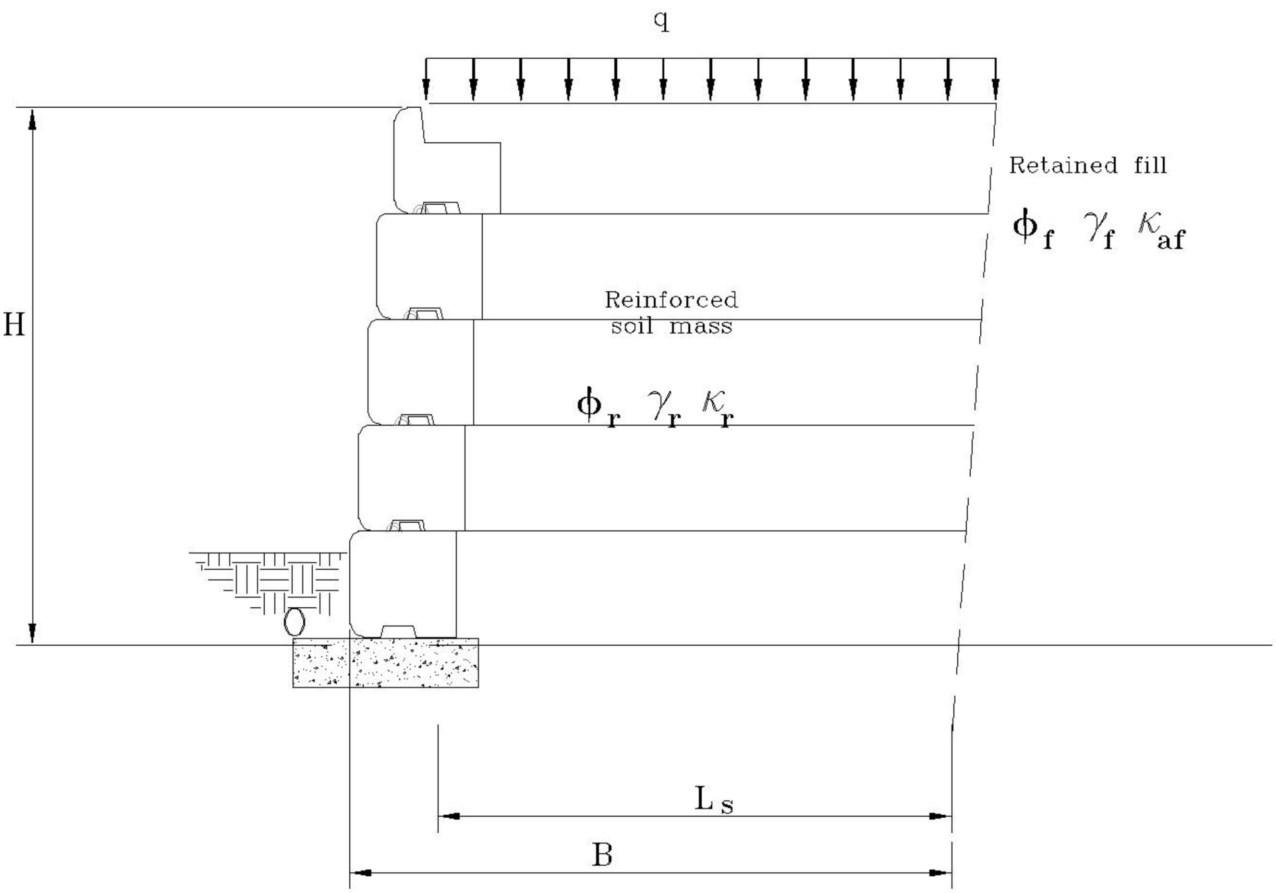


FIGURE C1. Design Example I Cross Section

Given:

H = 10 ft
h = H = 10 ft since $\beta = 0^o$
B = 0.7H = 0.7(10) = 7.0 ft, Try 7.5 ft
B_{block} = 2.0 ft
L_s = B - B_{block} = 7.5 - 2.0 = 5.5 ft
β = 0^o
δ = 20^o
α = 2.4^o (1:24 batter)
θ = $90 + \alpha = 92.4^o$
q = 250 psf
γ_{block} = 135 pcf

DETERMINATION OF EARTH PRESSURES

ϕ_r = 30^o
ϕ_f = 30^o
γ_f = 125 pcf
γ_r = 125 pcf

$$k_{af} = \frac{\sin^2(\theta+\phi_f)}{\sin^2\theta\sin(\theta-\delta)\left(1+\sqrt{\frac{\sin(\phi_f+\delta)\sin(\phi_f-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right)^2}$$

$$= \frac{\sin^2(92+30)}{\sin^2 92\sin(92-20)\left(1+\sqrt{\frac{\sin(30+20)\sin(30-0)}{\sin(92-20)\sin(92-0)}}\right)^2}$$

= 0.280

F_T = Resultant of active earth pressure = F_1

$$= \frac{k_{af}\gamma_f h^2}{2} = \frac{(0.280)(125)(10.0)^2}{2} = 1750 \text{ lbs/ft of wall}$$

V_1 = γ_r HL + γ_{block} (H)(B_{block}) = (125)(10.0)(5.5) + (135)(10.0)(2.0) = 9575 lbs/ft of wall

EVALUATION OF EXTERNAL STABILITY

Overturning

M_R = Resisting moment (lb-ft/ft of wall) = $V_1\left(\frac{B}{2}\right) = (9575)\left(\frac{7.5}{2}\right)$

= 35906 lb-ft per ft of wall

F_2 = qHk_{af} = $250(10.0)(0.280)$ = 700 lbs/ft of wall

M_D = Driving moment (lb-ft/ft of wall) $= F_1\left(\frac{H}{3}\right)+F_2\left(\frac{H}{2}\right) = 1750\left(\frac{10.0}{3}\right)+700\left(\frac{10.0}{2}\right)$

= 9328 lb-ft per ft of wall

Factor of safety against overturning,

$$FS_{OT} = \frac{M_R}{M_D} = \frac{35906}{8833} = 3.8 > 2.0$$ **OK**

Sliding

$$\rho = \tan^{-1}\left(\frac{2}{3}\tan(\phi_r)\right) = \tan^{-1}\left(\frac{2}{3}\tan(30)\right) = 21^o$$

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall) = $V_1\tan(\rho)$ = (9575)tan(21)
= 3675 lbs/ft of wall

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall) = $F_1 + F_2$ = 1750 + 700
= 2450 lbs/ft of wall

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} = \frac{3675}{2450} = 1.5 \geq 1.5$$

Bearing Capacity

B' = Effective base width (ft) = B = 7.5 ft
H_{emb} = 1 ft
ϕ_{found} = 32^o
c_{found} = 0 psf
γ_{found} = 125
N_c = 35.49
N_{γ} = 30.22
N_q = 23.18

q = γ_{found} (H_{emb}) = (125)(1) = 125 psf

q_{ult} = $cN_c + 0.5\gamma_{found} B' N_{\gamma} + q N_q$ = (0)(35.49) + (0.5)(125)(7.5)(30.22) + (125)(23.18) = 17063 psf

q_{all} = $\frac{q_{ult}}{FS} = \frac{17063}{3.0}$ = 5688 psf

R = V_1+qB = 9575 + 250(7.5) = 11450 lbs/ft of wall

$$e = \frac{F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)}{R} = \frac{1750\left(\frac{10.0}{3}\right) + 700\left(\frac{10.0}{2}\right)}{11450} = 0.81 \text{ ft}$$

Verify that:

$$\frac{R}{B-2e} \leq q_{all} \Rightarrow \frac{11450}{7.5-2(0.77)} = 1947\, psf \leq \text{ 5688 psf } \textbf{OK}$$

EVALUATION OF INTERNAL STABILITY

Maximum Tensile Forces

k_r = k_{af} = 0.280

S_V = 2.0 ft

Geogrid: Stratagrid 350

T_a = 2000 lbs/ft

σ_h = $\sigma_v k_r + \Delta\, \sigma_h$

σ_v = $\gamma_r Z + q + \Delta\sigma_v$
T_{max} = $\sigma_h S_V$

Verify that,

$T_a \geq T_{max}$

Z (ft)	γ_r (psf)	$\gamma_r Z$ (psf)	$\Delta\sigma_v$ (psf)	q (psf)	σ_v (psf)	σ_h (psf)	T_{max} (lbs/ft)	T_a (lbs/ft)	FS
2.0	125	250	0	250	500	140	280	2200	7.9
4.0	125	500	0	250	750	210	420	2200	5.2
6.0	125	750	0	250	1000	280	560	2200	3.9
8.0	125	1000	0	250	1250	350	700	2200	3.1

Table C1. Maximum Reinforcement Loads

Pullout

F^* = $0.8\tan\phi_r = 0.8\tan(30) = 0.46$
α = 0.8
L_{az} = $(H - Z)\tan(90-\varphi) = (10.0\text{-}Z)\tan(90-60) = (10.0\text{-} Z)\tan(30^o)$
L_{ez} = $B - L_{az}$
φ = $45 + \frac{\phi_r}{2} = 45 + \frac{30}{2} = 60^o$
P_r = $2F^*\gamma_r Z L_{ez}\alpha = 2(0.46)(125)(Z)(L_{ez})(0.8) = (92)ZL_{ez}$
σ_v = $\gamma_r Z + \Delta\sigma_v$
σ_h = $\sigma_v k_r + \Delta\,\sigma_h$
T_{max} = $\sigma_h S_V$

$$FS_{PO} = \frac{P_r}{T_{max}} \geq 1.5$$

Z (ft)	σ_v (psf)	σ_h (psf)	T_{max} (lbs/ft)	L_{az} (ft)	L_{ez} (ft)	P_r (lbs/ft)	FS
2.0	250	70	140	3.4	2.1	386	2.8
4.0	500	140	280	2.3	3.2	1178	4.2
6.0	750	210	420	1.1	4.4	2208	5.3
8.0	1000	280	560	0.0	5.5	4048	7.2

Table C2. Pullout Resistance

Connection Strength

T_o = Reinforcement tensile load per unit width of wall at each layer of reinforcement (lbs/ft)

$$X = \frac{(H-Z)\left(\frac{1}{2}\tan\left(45-\frac{\phi_r}{2}\right)\right)}{H} = \frac{(10.0-Z)\left(\frac{1}{2}\tan\left(45-\frac{30}{2}\right)\right)}{10.0}$$

$$= \frac{(10.0 - Z)(0.29)}{10.0}$$

$$\frac{T_o}{T_{max}} = 1 - X$$

Peak Strength:

T_{ac} = $1145 + \gamma_r Z \tan 4^o$ (from Appendix B – Stratagrid 350 connection capacities)

Serviceability Strength:

T_{ac} = $700 + \gamma_r Z \tan 0^o$ (from Appendix B – Stratagrid 350 connection capacities)

At the top of the wall:

$$\frac{T_o}{T_{max}} = 1-\frac{1}{2}\tan\left(45-\frac{\phi_r}{2}\right) = 1-\frac{1}{2}\tan\left(45-\frac{30}{2}\right) = 0.71$$

T_o = $0.71\ T_{max}$

At the bottom of the wall:

T_o = T_{max}

Verify that: $T_o \le T_a$ and $T_o \le T_{ac}$

Z (ft)	γ_rZ (psf)	X	$\frac{T_o}{T_{max}}$	T_{max} (lbs/ft)	T_o (lbs/ft)	Allowable Connection Load, Peak StrengthT_{ac} (lbs/ft)	FS
2.0	250	0.23	0.77	280	216	1162	5.4
4.0	500	0.17	0.83	420	349	1180	3.4
6.0	750	0.12	0.88	560	493	1197	2.4
8.0	1000	0.06	0.94	700	658	1215	1.8

Table C3. Connection Strength – Peak Strength

Z (ft)	$\gamma_r Z$ (psf)	X	$\frac{T_o}{T_{max}}$	T_{max} (lbs/ft)	T_o (lbs/ft)	Allowable Connection Load, Serviceability StrengthT_{ac} (lbs/ft)	FS
2.0	250	0.23	.77	280	216	700	3.2
4.0	500	0.17	0.83	420	349	700	2.0
6.0	750	0.12	0.88	560	493	700	1.4
8.0	1000	0.06	0.94	700	658	700	1.1

Table C4. Connection Strength – Serviceability Strength

Summary

1. A uniform geogrid length of 7.5 ft is adequate for external stability.
2. Stratagrid 350 is adequate in terms of allowable tensile capacity.
3. Stratagrid 350 is adequate in terms of allowable connection strength based on peak and serviceability connection strength values obtained from laboratory tests. Although the serviceability safety factor is low for the bottom geogrid layer since it is generally considered acceptable to have a single layer with a low factor of safety, the design as a whole is acceptable.

DESIGN EXAMPLE II

DETERMINATION OF STRUCTURAL DIMENSIONS

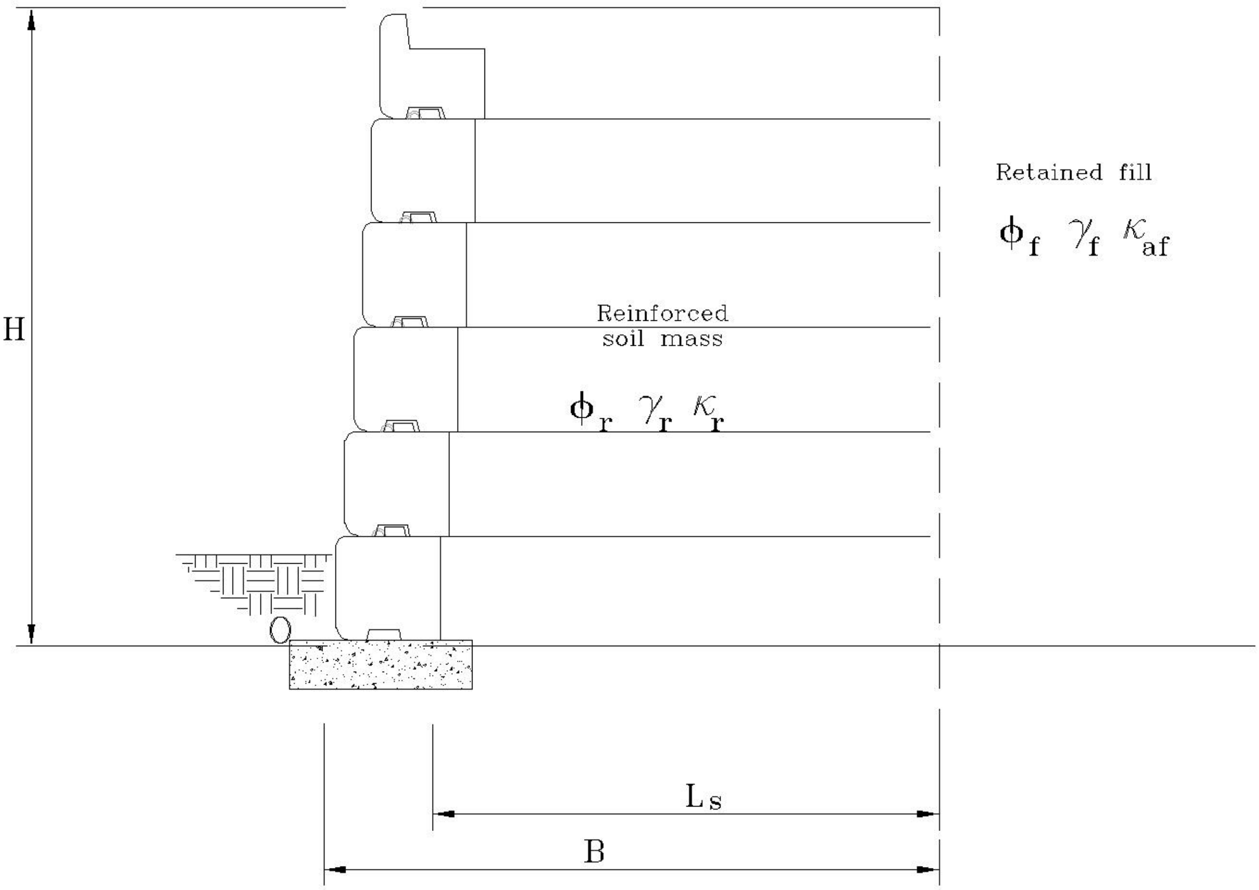


FIGURE C2. Design Example II Cross Section

Given:

H = 12.0 ft
h = H = 12.0 ft since $\beta = 0^o$
B = 0.7H = 0.7(12.0) = 8.4 ft, Try 11 ft
Note: Due to high sliding and internal stability requirements, the geogrid length for seismic conditions should be a minimum of 25% longer than what's required for static conditions.
B_{block} = 2.0 ft
L_s = $B - B_{block}$ = 11 - 2.0 = 9.0 ft
β = 0^o
δ = 24^o
α = 2.4^o
θ = $90 + \alpha = 92.4^o$
q = 250 psf
γ_{block} = 135 pcf

DETERMINATION OF EARTH PRESSURES

ϕ_f = 35^o

γ_f = 125 pcf

$$k_{af} = \frac{\sin^2(\theta + \phi_f)}{\sin^2\theta\sin(\theta-\delta)\left(1+\sqrt{\dfrac{\sin(\phi_f+\delta)\sin(\phi_f-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right)^2}$$

$$= \frac{\sin^2(92+35)}{\sin^2 92\sin(92-24)\left(1+\sqrt{\dfrac{\sin(35+24)\sin(35-0)}{\sin(92-24)\sin(92+0)}}\right)^2}$$

= 0.228

F_T = Resultant of active earth pressure = F_1

$$= \frac{k_{af}\gamma_f h^2}{2} = \frac{(0.228)(125)(12.0)^2}{2} = 2052 \text{ lbs/ft of wall}$$

V_1 = $\gamma_f HL_s + (\gamma_{block})(H)(B_{block})$ = (125)(12.0)(9.0) + (135)(12.0)(2.0) = 16740 lbs/ft of wall

DETERMINATION OF SEISMIC PARAMETERS

A = 0.15g

A_m = (1.45-A)A = 0.195

P_{IR} = $0.5A_m\,\gamma_f\,H^2$ = $0.5(0.195)(125)(12.0)^2$ = 1755 lbs/ft of wall

P_{AE} = $0.375\,A_m\,\gamma_f\,H^2$ = $0.375(0.345)(125)(12.0)^2$ = 1316 lbs/ft of wall

EVALUATION OF EXTERNAL STABILITY

For seismic,

1. Apply 50% of P_{AE} at 0.6H above the bottom of the wall.
2. Apply P_{IR} at 0.5H above the bottom of the wall.

Overturning

M_R = Resisting moment (lb-ft/ft of wall) $= V_1\left(\frac{B}{2}\right) = (16740)\left(\frac{11.0}{2}\right)$

= 92070 lb-ft per ft of wall

F_2 = $qHk_{af} = 250(12.0)(0.228)$ = 684lbs/ft of wall

M_D = Driving moment (lb-ft/ft of wall) $= F_1\left(\frac{H}{3}\right)+F_2\left(\frac{H}{2}\right)+0.5P_{AE}(0.6H)+P_{IR}(0.5H)$

$$= 2052\left(\frac{12.0}{3}\right)+684\left(\frac{12.0}{2}\right)+0.5(1316)(0.6)(12.0)+1755(0.5)(12.0)$$

= 27580 lb-ft per ft of wall

Factor of safety against overturning,

$$FS_{OT} = \frac{M_R}{M_D} = \frac{92070}{27580} = 3.3 > 1.5$$

Sliding

$$\rho = \tan^{-1}\left(\frac{2}{3}\tan(\phi_r)\right) = \tan^{-1}\left(\frac{2}{3}\tan(35)\right) = 25^o$$

F_r = Sum of forces providing resistance to sliding (lbs/ft of wall) = $V_1\tan(\rho)$ = (16740)tan(25)
= 7814 lbs/ft of wall

F_d = Sum of forces driving the wall in sliding (lbs/ft of wall) = $F_1 + F_2 + (0.50)P_{AE} + P_{IR}$
= 2052 + 684 + (0.50)1316 + 1755
= 5149 lbs/ft of wall

Factor of safety against sliding,

$$FS_{SL} = \frac{F_r}{F_d} = \frac{7814}{5149} = 1.52 \geq 1.125$$

Bearing Capacity

B' = Effective base width (ft) = B = 11 ft
H_{emb} = 1 ft
ϕ_{found} = 32^o
c_{found} = 0 psf
γ_{found} = 125
N_c = 35.49
N_{γ} = 30.22
N_q = 23.18

q = γ_{found} (H_{emb}) = (125)(1) = 125 psf

q_{ult} = $cN_c + 0.5\gamma_{found} B' N_\gamma + q N_q$ = (0)(35.49) + (0.5)(125)(11)(30.22) + (125)(23.18) = 23674 psf

$$q_{all} = \frac{q_{ult}}{FS} = \frac{23674}{1.5} = 15783\,psf$$

R = V_1+qB = 16740 + 250(11) = 19490 lbs/ft of wall

$$e = \frac{F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)}{R} = \frac{2052\left(\frac{12.0}{3}\right) + 684\left(\frac{12.0}{2}\right)}{19490} = 0.63 \text{ ft}$$

Verify that:

$$\frac{R}{B-2e} \leq q_{all} \Rightarrow \frac{19490}{11-2(0.63)} = 2001\,psf \leq 15783\,psf$$

OK

EVALUATION OF INTERNAL STABILITY

Maximum Tensile Forces

k_r = k_{af} = 0.228

S_V = 2.0 ft

Geogrid: Stratagrid 550

$T_{a(static)}$ = 3500 lbs/ft

$T_{a(seismic)}$ = (3500)(1.5) = 5250 lbs/ft

P_I = $A_m W_A$ = 0..195(4685) = 914 lbs/ft

Where,

W_A = weight of the active zone

$$= \gamma_r \frac{1}{2} H^2 \tan\left(45 - \frac{\phi_r}{2}\right) = (125)\frac{1}{2}(12.0)^2 \tan\left(45 - \frac{35}{2}\right) \;\; = 46857 \text{ lbs/ft}$$

$$T_{md} = P_I \left(\frac{L_{ezi}}{\sum_{i=1}^{n} (L_{ezi})} \right) = 914\left(\frac{L_{ezi}}{34.7}\right) = (26.3) L_{ezi}$$

T_{total} = $T_{max} + T_{md}$

For pullout, the value of F* can be reduced to 80% of the static F* value. In addition, the allowable strength of the geogrid reinforcement can be increased by eliminating the creep reduction factor. So,

$T_{a_{seismic}} = (T_{a_{static}})(RF_{creep})$

σ_h = $\sigma_v k_r + \Delta\, \sigma_h$

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

T_{max} = $\sigma_h S_V$

Verify that,

$T_a \geq T_{total}$

Z (ft)	γ_r (psf)	$\gamma_r Z$ (psf)	$\Delta\sigma_v$ (psf)	q (psf)	σ_v (psf)	σ_h (psf)	T_{max} (lbs/ft)	T_{md} (lbs/ft)	T_{total} (lbs/ft)	T_a (lbs/ft)	FS
2.0	125	250	0	250	500	114	228	128	356	3500	9.8
4.0	125	500	0	250	750	171	342	290	632	3500	5.5
6.0	125	750	0	250	1000	228	456	346	802	3500	4.4
8.0	125	1000	0	250	1250	285	570	402	972	3500	3.6
10.0	125	1250	0	250	1500	342	684	458	1142	3500	3.1

Table C5. Maximum Reinforcement Loads

Pullout

$F^*_{seismic}$ = $(0.8)(0.8)\tan\phi_r = (0.8)(0.8)\tan(35) = 0.45$
α = 0.8
L_{az} = $(H - Z)\tan(90-\varphi) = (12.0-Z)\tan(90-62.5) = (12.0- Z)\tan(27.5^o)$
L_{ez} = $B - L_{az}$
φ = $45 + \frac{\phi_r}{2} = 45 + \frac{35}{2} = 62.50^o$
P_r = $2F^*\gamma_r Z L_{ez}\alpha = 2(0.45)(125)(Z)(L_{ez})(0.8) = (90)ZL_{ez}$
σ_v = $\gamma_r Z + \Delta\sigma_v$
σ_h = $\sigma_v k_r + \Delta\, \sigma_h$
T_{max} = $\sigma_h S_V$

$$FS_{PO} = \frac{P_r}{T_{max}} \geq 1.125$$

Z (ft)	σ_v (psf)	σ_h (psf)	T_{max} (lbs/ft)	T_{md} (lbs/ft)	T_{total} (lbs/ft)	L_{az} (ft)	L_{ez} (ft)	P_r (lbs/ft)	FS
2.0	500	114	228	83	311	4.1	4.9	871	2.8
4.0	750	171	342	186	528	3.1	5.9	2115	4.0
6.0	1000	228	456	240	696	2.0	7.0	3732	5.4
8.0	1250	285	570	292	862	1.0	8.0	5722	6.6
10.0	1500	342	684	344	1028	0.0	9.0	8086	7.9

Table C6. Pullout Resistance

Connection Strength

T_o = Reinforcement tensile load per unit width of wall at each layer of reinforcement (lbs/ft)

$$X = \frac{(H - Z)\left(\frac{1}{2}\tan\left(45 - \frac{\phi_r}{2}\right)\right)}{H} = \frac{(12.0 - Z)\left(\frac{1}{2}\tan\left(45 - \frac{35}{2}\right)\right)}{12.0}$$

$$= \frac{(10.0 - Z)(0.26)}{10.0}$$

Peak Strength:
T_{ac} = $1974 + \gamma_r Z \tan 2^o$ (from Appendix B – Stratagrid 550 connection capacities)

Serviceability Strength:
T_{ac} = $880 + \gamma_r Z \tan 5^o$ (from Appendix B – Stratagrid 550 connection capacities)

At the top of the wall:

$$\frac{T_o}{T_{max}} = 1-\frac{1}{2}\tan\left(45-\frac{\phi_r}{2}\right)=1-\frac{1}{2}\tan\left(45-\frac{35}{2}\right)=0.74$$

$T_o = 0.74\ T_{max}$

At the bottom of the wall:

$T_o = T_{max}$

Verify that: $T_o \le T_a$ and $T_o \le T_{ac}$

Z (ft)	γ_rZ (psf)	X	$\frac{T_o}{T_{total}}$	T_{total} (lbs/ft)	T_o (lbs/ft)	Allowable Connection Load, Peak StrengthT_{ac} (lbs/ft)	FS
2.0	250	0.22	0.78	471	369	1983	5.4
4.0	500	0.17	0.83	638	527	1991	3.8
6.0	750	0.13	0.87	803	699	2000	2.9
8.0	1000	0.09	0.91	970	886	2009	2.3
10.0	1250	0.04	0.96	1136	1087	2018	1.9

Table C7. Connection Strength – Peak Strength

Z (ft)	γ_rZ (psf)	X	$\frac{T_o}{T_{max}}$	T_{total} (lbs/ft)	T_o (lbs/ft)	Allowable Connection Load, Serviceability StrengthT_{ac} (lbs/ft)	FS
2.0	250	0.22	0.78	471	369	902	2.4
4.0	500	0.17	0.83	638	527	924	1.7
6.0	750	0.13	0.87	803	699	946	1.4
8.0	1000	0.09	0.91	970	886	967	1.1
10.0	1250	0.04	0.96	1136	1087	989	0.91

Table C8. Connection Strength – Serviceability Strength

Summary

1. A uniform geogrid length of 11 ft is adequate for external stability.
2. Stratagrid 550 is adequate in terms of allowable tensile capacity and pullout.
3. Stratagrid 550 is adequate in terms of allowable connection strength based on peak and serviceability connection strength values obtained from laboratory tests, with the exception of the bottom layer since it is generally considered acceptable for a single layer to have a low factor of safety, the design as a whole is considered acceptable.

DESIGN EXAMPLE III

DETERMINATION OF STRUCTURAL DIMENSIONS

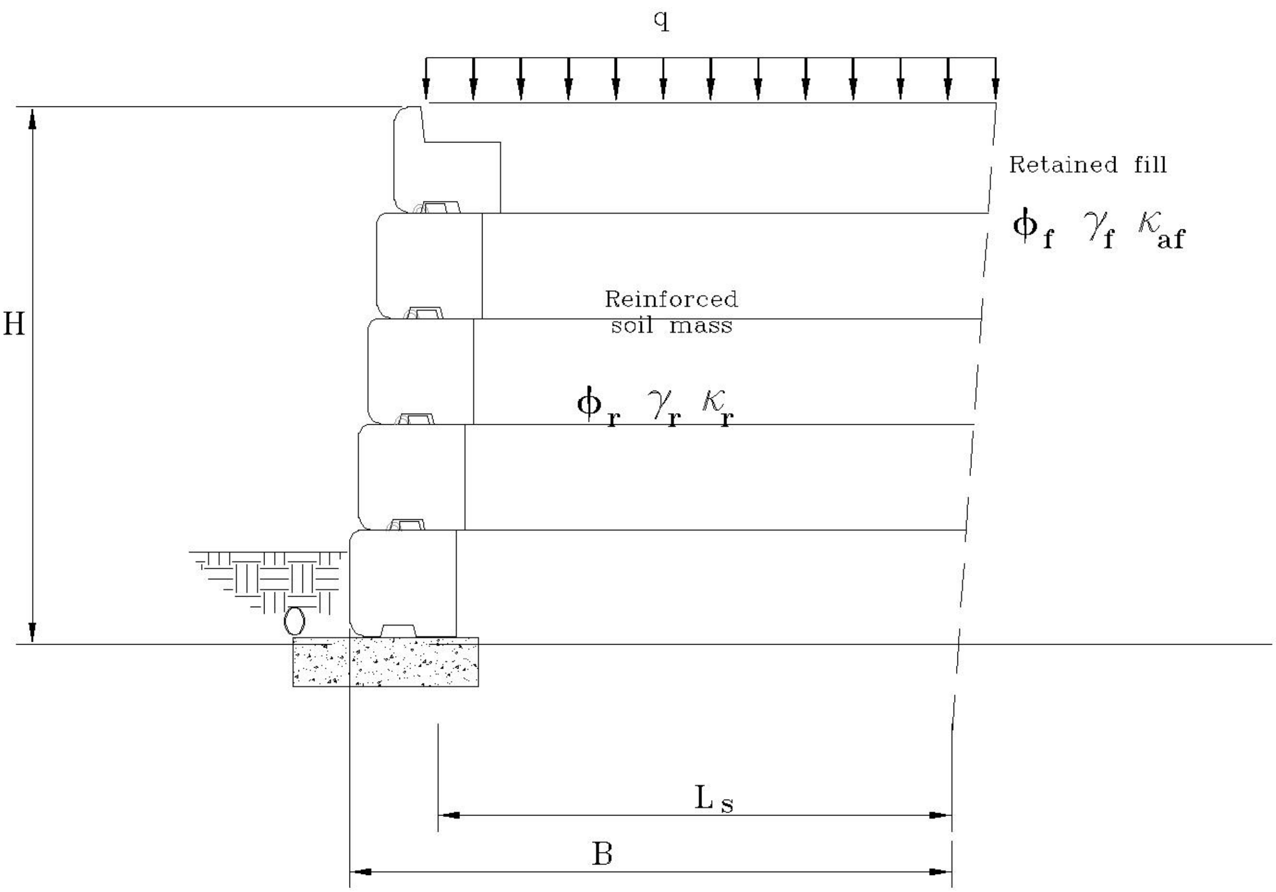


FIGURE C3. Design Example III Cross Section

Given:

H = 10 ft
h = H = 10 ft since $\beta = 0^o$
B = 0.7H = 0.7(10) = 7.0 ft, Try 7.5 ft
B_{block} = 2.0 ft
L_s = B - B_{block} = 7.5 - 2.0 = 5.5 ft
β = 0^o
δ = 20^o
α = 2.4^o (1:24 batter)
θ = $90 + \alpha = 92.4^o$
q = 250 psf
γ_{block} = 135 pcf

DETERMINATION OF EARTH PRESSURES

ϕ_r = 30^o
ϕ_f = 30^o
γ_f = 125 pcf
γ_r = 125 pcf

$$k_{af} = \frac{\sin^2(\theta+\phi_f)}{\sin^2\theta\sin(\theta-\delta)\left(1+\sqrt{\dfrac{\sin(\phi_f+\delta)\sin(\phi_f-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right)^2}$$

$$= \frac{\sin^2(92+30)}{\sin^2 92\sin(92-20)\left(1+\sqrt{\dfrac{\sin(30+20)\sin(30-0)}{\sin(92-20)\sin(92-0)}}\right)^2}$$

= 0.280

F_T = Resultant of active earth pressure = F_1

$$= \frac{k_{af}\gamma_f h^2}{2} = \frac{(0.280)(125)(10.0)^2}{2} = 1750 \text{ lbs/ft of wall}$$

V_1 = γ_r HL + γ_{block} (H)(B_{block}) = (125)(10.0)(5.5) + (135)(10.0)(2.0) = 9575 lbs/ft of wall

LRFD FACTORS

Load Factors:

EH = Active Earth Pressure Load Factor = 1.35
EV = Vertical Earth Pressure Load Factor = 1.00
ES = Earth Surcharge Load Factor = 1.00

Resistance Factors:

RF_B = Bearing Resistance Factor = 0.45
RF_{SL} = Sliding Resistance Factor = 0.90
RF_{OT} = Overturning Resistance Factor = 0.70
RF_{GST} = Geosynthetic Resistance Factor (static) = 0.90
RF_{GEQ} = Geosynthetic Resistance Factor (seismic) = 1.20

EVALUATION OF EXTERNAL STABILITY

Overturning

M_R = Resisting moment (lb-ft/ft of wall) = $V_1\left(\frac{B}{2}\right) RF_{OT} = (9575)\left(\frac{7.5}{2}\right)0.7$

= 25,134 lb-ft per ft of wall (factored resistance)

$$F_2 = qHk_{af} = 250(10.5)(0.280) = 700 \text{ lb/ft of wall}$$

$$M_D = \text{Driving moment (lb-ft/ft of wall)} = F_1\left(\frac{H}{3}\right)EH + F_2\left(\frac{H}{2}\right)ES = 1750\left(\frac{10.0}{3}\right)1.35 + 700\left(\frac{10.0}{2}\right)1.0$$

M_D = 10,767 lb-ft per ft of wall (factored load)

$M_R > M_D$ **OK**

Sliding

$$\rho = \tan^{-1}\left(\frac{2}{3}\tan(\phi_r)\right) = \tan^{-1}\left(\frac{2}{3}\tan(30)\right) = 21^o$$

F_r = Sum of forces providing resistance to sliding (lb/ft of wall) = $V_1\tan(\rho)\ RF_{SL}$ = (9575)tan (21)0.90
= 3,308 lb/ft of wall (factored resistance)

F_d = Sum of forces driving the wall in sliding (lb/ft of wall) = $F_1EH + F_2ES$ = 1750(1.35) + 700(1.0)
= 2899 lb/ft of wall (factored load)

$F_R > F_D$ **OK**

Bearing Capacity

B' = Effective base width (ft) = B = 7.5 ft
H_{emb} = 1 ft
ϕ_{found} = 32°
c_{found} = 0 psf
γ_{found} = 125
N_c = 35.49
N_γ = 30.22
N_q = 23.18

q = γ_{found} (H_{emb}) = (125)(1) = 125 psf

q_{ult} = $cN_c + 0.5\gamma_{found}\,B'\,N_\gamma + q\,N_q$ = (0)(35.49) + (0.5)(125)(7.5)(30.22) + (125)(23.18) = 17063 psf

Q_R = q_{ult} RF = 17063(.45) = 7678 psf (factored resistance)

$$Q_D = \frac{V_1(EH)+q(B)ES}{B} = \frac{9575(1.35) + 250(7.5)(1.0)}{7.5} = 1974 \text{ psf (factored load)}$$

$Q_R > Q_D$ **OK**

Check eccentricity per ASD - see Design Example I

$$e = \frac{F_1\left(\frac{H}{3}\right) + F_2\left(\frac{H}{2}\right)}{R} = \frac{1750\left(\frac{10.0}{3}\right) + 700\left(\frac{10.0}{2}\right)}{11450} = 0.81 \text{ ft}$$

Verify that: $\frac{R}{B-2e} \le q_{all} \Rightarrow \frac{11450}{7.5-2(0.81)} = 1947\,psf \le 5688$ psf **OK**

EVALUATION OF INTERNAL STABILITY

Maximum Tensile Force in Reinforcement

k_r = k_{af} = 0.280

S_V = 2.0 ft

Geogrid: Stratagrid 350

T_a = 2200 lb/ft (could also start with T_{ult} and factor)

T_R = $T_a RF_{SL}$ = 2200(0.9) = 1980 lb/ft (factored resistance)

σ_v = $\sigma_v k_r + \Delta\, \sigma_h$

σ_h = $(\gamma_r Z + q + \Delta\sigma_v)$ EV (could argue for EH but loads are all vertical – get same OK results either way)

T_L = $\sigma_h S_V$ (factored load)

Verify that,

$T_R \geq T_L$

Z (ft)	γ_r (psf)	$\gamma_r Z$ (psf)	q (psf)	σ_v (psf)	σ_v (psf) factored	σ_h (psf)	T_L (lb/ft)	T_R (lb/ft)	
2.0	125	250	250	500	500	140	280	1980	OK
4.0	125	500	250	750	750	210	420	1980	OK
6.0	125	750	250	1000	1000	280	560	1980	OK
8.0	125	1000	250	1250	1250	350	700	1980	OK

Table C9. Maximum Reinforcement Loads

Pullout

F^* = $0.8\tan\phi_r = 0.8\tan(30) = 0.46$

α = 0.8

L_{az} = $(H - Z)\tan(90-\varphi) = (10.0-Z)\tan(90-60) = (10.0- Z)\tan(30^o)$

L_{ez} = $B - L_{az}$

φ = $45+\frac{\phi_r}{2} = 45+\frac{30}{2} = 60^o$

P_r = $2F^*\gamma_r Z L_{ez}\alpha\ RF_{GST} = 2(0.46)(125)(Z)(L_{ez})(0.8)(0.90) = (83)ZL_{ez}$ (factored resistance)

σ_v = $(\gamma_r Z + \Delta\sigma_v)$EV

σ_h = $\sigma_v k_r + \Delta\, \sigma_h$

T_L = $\sigma_h S_V$ (factored load)

Verify that,

$P_R \geq T_L$

Z (ft)	σ_v (psf)	σ_h (psf)	T_L (lb/ft)	L_{az} (ft)	L_{ez} (ft)	P_r (lb/ft)	
2.0	250	70	140	3.4	2.1	342	OK
4.0	500	140	280	2.3	3.2	1067	OK
6.0	750	210	420	1.1	4.4	2177	OK
8.0	1000	280	560	0.0	5.5	3671	OK

Table C10. Pullout Resistance

Connection Strength

T_o = Reinforcement tensile load per unit width of wall at each layer of reinforcement (lb/ft)

$$X = \frac{(H-Z)\left(\frac{1}{2}\tan\left(45-\frac{\phi_r}{2}\right)\right)}{H} = \frac{(10.0-Z\ )\left(\frac{1}{2}\tan\left(45-\frac{30}{2}\right)\right)}{10.0}$$

$$= \frac{(10.0 - Z)(0.26)}{10.0}$$

$$\frac{T_o}{T_L} = 1 - X$$

Peak Strength:

T_{ac} = $1145 + \gamma_r Z \tan 4^o$ (from Appendix B – Stratagrid 350 connection capacities)

Serviceability Strength:

T_{ac} = $700 + \gamma_r Z \tan 0^o$ (from Appendix B – Stratagrid 350 connection capacities)

Both strength values should be factored by RF_{GST}

At the top of the wall:

$$\frac{T_o}{T_L} = 1-\frac{1}{2}\tan\left(45-\frac{\phi_r}{2}\right)=1-\frac{1}{2}\tan\left(45-\frac{30}{2}\right)=0.71$$

T_o = $0.71\ T_L$

At the bottom of the wall:

T_o = T_L

Verify that: $T_o \le T_{ac}$

Z (ft)	$\gamma_r Z$ (psf)	X	$\frac{T_o}{T_L}$	T_L (lb/ft)	T_o (lb/ft)	Peak StrengthT_{ac} (lb/ft)	Tac Factored (lb/ft)	
2.0	250	0.23	0.77	280	216	1162	1046	OK
4.0	500	0.17	0.83	420	349	1180	1062	OK
6.0	750	0.12	0.88	560	493	1197	1077	OK
8.0	1000	0.06	0.94	700	658	1215	1094	OK

Table C11. Connection Strength – Peak Strength

Z (ft)	$\gamma_r Z$ (psf)	X	$\frac{T_o}{T_L}$	T_L (lb/ft)	T_o (lb/ft)	Serviceability StrengthT_{ac} (lb/ft)	Tac Factored (lb/ft)	
2.0	250	0.23	0.77	280	216	700	630	OK
4.0	500	0.17	0.83	420	349	700	630	OK
6.0	750	0.12	0.88	560	493	700	630	OK
8.0	1000	0.06	0.94	700	658	700	630	OK

Table C12. Connection Strength – Serviceability Strength

Summary

1. A uniform geogrid length of 7.5 ft is adequate for external stability.
2. Stratagrid 350 is adequate in terms of allowable tensile capacity.
3. Stratagrid 350 is adequate in terms of allowable connection strength based on peak and serviceability connection strength values obtained from laboratory tests, with the exception of the bottom layer since it is generally considered acceptable for a single layer to have a low factor of safety, the design as a whole is considered acceptable.

APPENDIX D

BEARING CAPACITY FACTORS

ϕ_{found}	N_c	N_q	N_γ	ϕ_{found}	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67	20.63	25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	20.6	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.3	48.03
10	8.36	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.2	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88				

TABLE D1. Bearing Capacity Factors (from Vesic, 1975)

APPENDIX E

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