# Technical Information Sheets

A collection of one page summary documents on specific engineering topics for the design of the Keystone Retaining Wall System.

9/9/22


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# Technical Information Sheets


TIS-01 Gravity Walls

TIS-02 Reinforced Soil Walls

TIS-03 Soil Strength

TIS-04 Coulomb/Rankine Pressure

TIS-05 Equivalent Fluid Pressure

TIS-06 Compaction and Strength of Soils

TIS-07 Soil Density - Standard vs. Modified Proctor

TIS-08 Silt/Clay Soils - Atterberg Limits

TIS-09 Backfill Soil Specifications

TIS-10 Wall Embedment

TIS-11 Bearing Capacity

TIS-12 Unit Drainage Fill

TIS-13 Connection Strength Testing

TIS-14 Connection Strength Discussion

TIS-15 Global Stability

TIS-16 Single Wall - Slope Stability Ratios

TIS-17 Tiered Wall - Slope Stability Ratios

TIS-18 Tiered Wall - Internal Analysis

TIS-19 Seismic Analysis

TIS-20 Water Flow - Mannings Number

TIS-21 Drawdown Analysis

TIS-22 Guard and Barrier Requirements

TIS-23 Railing Design-Direct Mount - 20 plf

TIS-24 Railing Design-Direct Mount - 50 plf

TIS-25 Railing Design - Offset Mount - 50 plf

TIS-26 Guiderail Detail

TIS-27 Traffic Barrier

TIS-28 Quality Assurance Provisions

TIS-29 Tall Walls - Alignment/Adjustments

TIS-30 Settlement - Unit Cracking/Gapping

TIS-31 Corners/Bends - Unit Cracking/Gapping

TIS-32 Backfill Movement and Soil Cracks

TIS-33 Durability Freeze-Thaw

TIS-34 Vertical Reinforcement Spacing

TIS-35 Wall Precautions and Maintenance

TIS-36 Foundation Improvements



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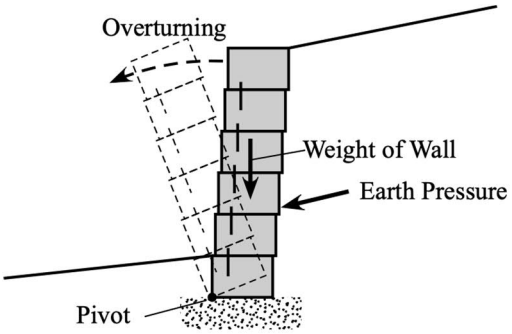
# Gravity Walls

Gravity walls are any coherent structure that rely solely on its mass and geometry to resist the earth pressure forces acting on it. All Keystone landscaping wall products and structural wall units not utilizing soil reinforcement are considered gravity walls by definition.

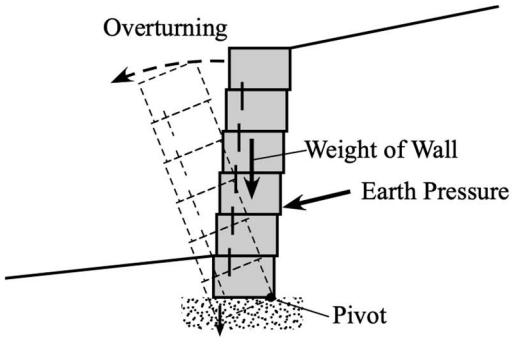
Modular gravity walls rely on weight, depth, wall batter, and inter-unit shear strength to achieve stability. Larger units with more depth provide greater stability and can achieve greater gravity wall heights as indicated in the gravity wall design charts in Keystone design and construction documents.

The principal mode of failure observed in a modular gravity wall is overturning assuming that there is a positive shear connection between units from pins or lips. Overturning failure occurs for a few simple and obvious reasons:

1) A gravity wall is constructed taller than it should be for the size of unit utilized and the design conditions at the wall location.
2) A gravity wall is constructed over poor foundation soils or a poor leveling pad and post construction settlement causes the wall to lean and eventually overturn.
3) Additional surcharge from parking or a structure is placed directly behind the wall.
4) Combinations of the above items.




**Simple Overturning**




**Settlement and Overturning**

Special attention must be given to the foundation soils and leveling pad construction when constructing gravity walls since the foundation provides much of the wall's resistance to overturning. Simple overturning failure can be avoided by limiting wall heights to safe working heights for the size of Keystone unit selected and avoiding additional surcharge conditions. A simple "rule of thumb" is to restrict wall heights to no greater than three times the unit depth unless referring to design charts or calculations for site specific design recommendations. This "rule of thumb" leads to the following rough guidelines for no surcharge landscaping situations:

| Unit | Height | Depth | Max Height | Approx. Courses |
|---|---|---|---|---|
| Country Manor | 6” | 10” | 27” | 4 courses + cap |
| Regal Stone Pro | 8” | 12” | 36” | 4 courses + cap |
| Compac III | 8” | 12” | 36” | 4 courses + cap |
| Standard III | 8” | 18” | 52” | 6 courses + cap |
| Standard III | 8” | 21” | 60” | 7 courses + cap |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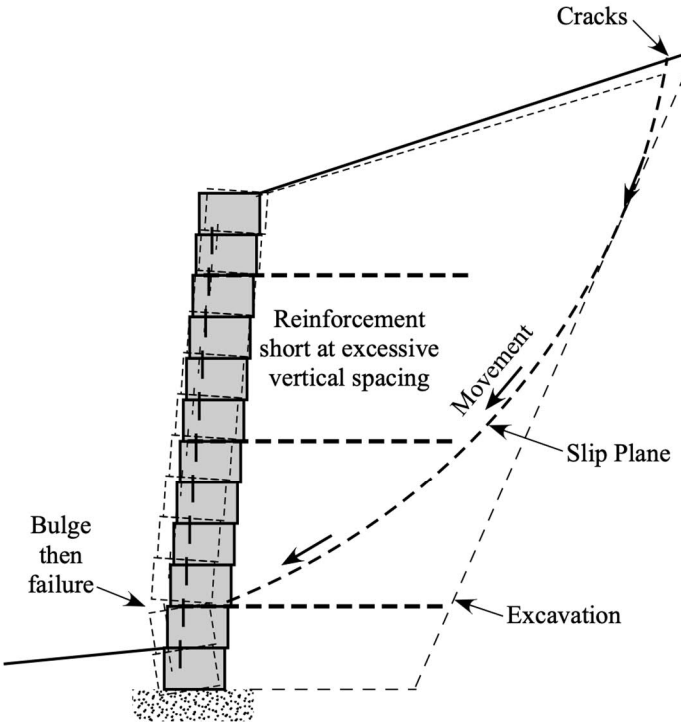
# Reinforced Soil Walls

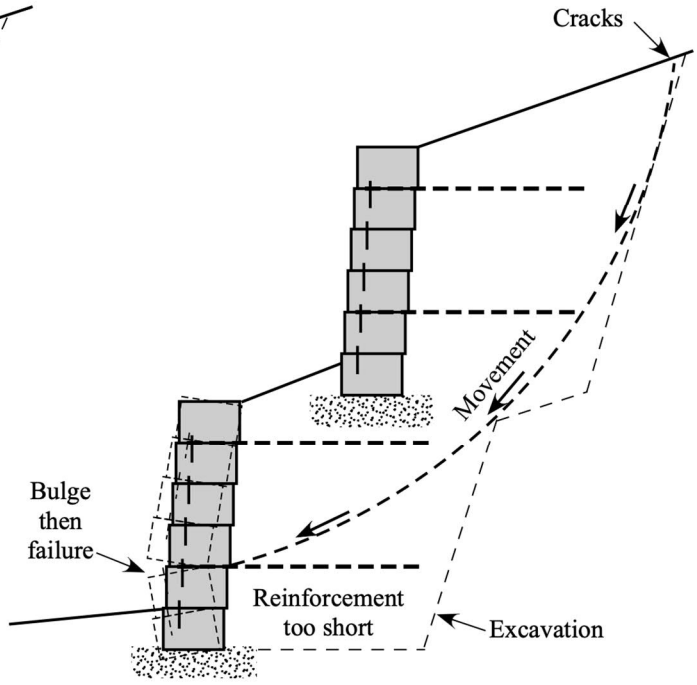
Reinforced soil walls are composite structures which utilize structural Keystone units and geosynthetic soil reinforcement to create a stable mass that can be designed and constructed to much greater heights than simple gravity walls. These structures are commonly referred to as MSE (mechanically stabilized earth) structures or reinforced soil SRW's (segmental retaining walls). Note that Keystone “pinned” or “lipped/lug” units are designed to properly accommodate earth reinforcement and provide facial stability and connection strength for these larger and more critical structures.

Reinforced walls rely on the mass of the composite structure to provide external stability (sliding, overturning, etc.) and the strength of the soil reinforcement, connections, and Keystone units to be internally stable. The principal modes of failure observed in reinforced wall structures are:

1) Inadequate reinforcement length and vertical spacing to prevent internal/external failure.
2) Use of poor quality soils and/or improper placement and compaction of soils
3) Inadequate surface runoff or internal soil drainage provisions (ie: groundwater)
4) Tiered walls not being designed and constructed as complex multi-wall soil structures.




**Internal/External Failure Mechanism**




**Tiered Wall Failure Mechanism**

Reinforced walls are considerably more complex than simple gravity walls and must be designed accordingly. The soil strength and stability component of the design takes on much greater importance as the structures become taller requiring more attention to site specific soils information, proper design considerations, and contractor quality assurance provisions.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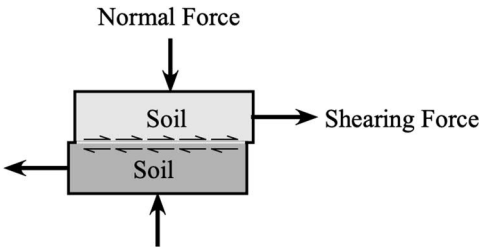
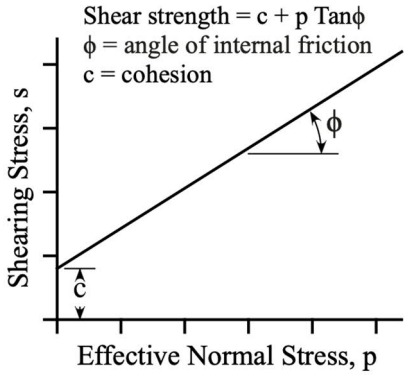
# Soil Strength

Soil strength is a complicated geotechnical concept to simplify due to the inherent complexities of different soil types. Frictional strength, cohesive strength, and pore water pressure relationships are all integral to the effective strength determination of a soil but are only easily identified in the most select granular materials.

For the purpose of this brief discussion, all soils are assumed to be drained with no pore pressure considerations. Cohesion is typically neglected in the simplified design methods and a frictional strength relationship ($\phi$ and $\gamma$ only) is utilized to determine basic driving and resisting forces. The figures below show the basic soil strength relationship.




**Soil Shear Test**




**Plot of Shear Stress**

The values for $\phi$ and c can be determined by direct shear test for granular soils and by triaxial testing for cohesive soils. In-situ unit weight ($\gamma$) can be estimated from Proctor density test data. There are significant differences between the properties of undisturbed in-situ soils, laboratory remolded soil samples, and contractor placed soils so careful evaluation of design properties vs. construction considerations is required.

Laboratory testing of soils is desired but not always practical due to cost and time considerations so the following table is presented to provide approximate design ranges for typical soil types.

## Approximate Soil Design Parameter Ranges

| Wall Backfill Classification | Common Description | UNSC Classification | $\phi$ range | $\phi$ range (moist) | Comments |
|---|---|---|---|---|---|
| Good | Sand, Gravel Stone | GW, GP, GM GC, SW, SP | 32° - 36° | 100 - 135 pcf | Poor grading lowers weight (ie: #57 stone) |
| Moderate | Silty Sands Clayey Sands | SM, SC | 28° - 32° | 110 - 135 pcf | Moisture Sensitive |
| Difficult | Silts, Low Plastic Clays | ML, CL, OL | 25° - 30° | 110 - 125 pcf | PI < 20 LL < 40 |
| Bad | High Plastic Silts & Clays, Organics | CH, MH OH, PT | 0° - 25° | 50 - 110 pcf | PI > 20 LL > 40 |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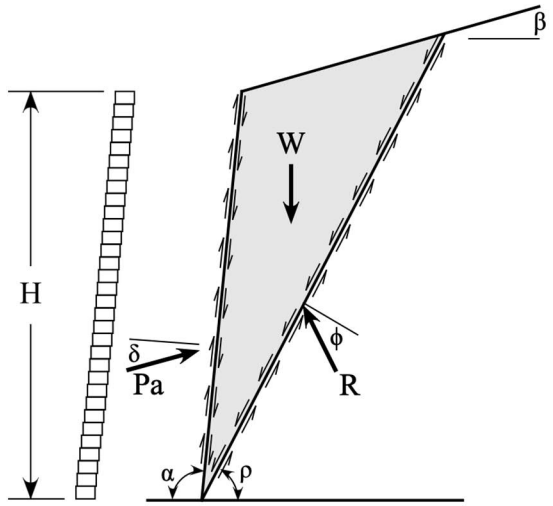
# Coulomb/Rankine Earth Pressure

There are two commonly accepted methods for calculating simple earth pressure, Coulomb and Rankine theory. The Coulomb theory was developed in the 1776 and the Rankine theory was developed in the 1857 and both remain the basis for present day earth pressure calculation.

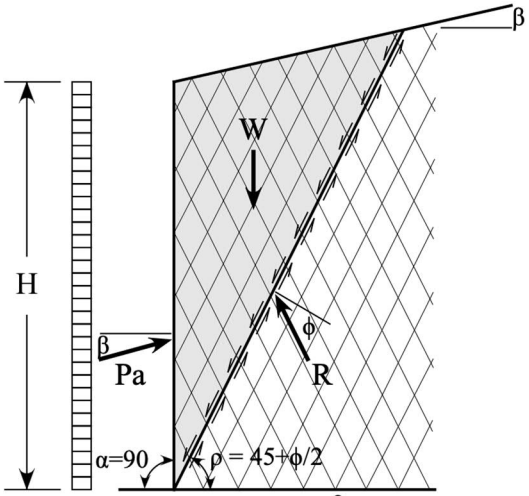
The general equations developed for both theories are based on the fundamental assumptions that the retained soil is **cohesionless** (no clay component), **homogeneous** (not a varying mixture of materials), **isotropic** (similar stress-strain properties in all directions or in practical terms, not reinforced), **semi-infinite** (wall is very long and soil goes back a long distance without bends or other boundary conditions), and **well drained** (to avoid consideration of pore pressures).

The active earth pressure calculations below requires that the wall structure rotates or yields sufficiently to engage the entire shear strength of the soils involved to create the active earth pressure state. The amount of movement required is highly dependent upon the type of soils retained.




$Pa = 1/2\ \gamma\ H^2\ Ka$

$$Ka = \frac{sin^2(\alpha + \phi)}{sin^2(\alpha)sin(\alpha - \delta)\left[1 + \sqrt{\frac{sin(\phi + \delta)\ sin(\phi - \delta)}{sin(\alpha - \delta)\ sin(\alpha + \beta)}}\right]^2}$$




$Pa = 1/2\ \gamma\ H^2\ Ka$

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

## Coulomb Wedge Analysis

Using identical parameters, Coulomb wedge theory calculates less earth pressure than Rankine theory for a level backslope whereas the values converge under backslope conditions when $\delta = \beta$. Coulomb theory calculates a unique failure angle for every design condition whereas application of Rankine theory to reinforced soil structures fixes the internal failure plane at $45 + \phi/2$ from horizontal.

The application of Coulomb active wedge theory and a calculated failure plane is favored by the National Masonry Concrete Association (NCMA) and described in their *Design Manual for Segmental Retaining Walls - Third Edition*. Coulomb is recommended over Rankine for battered wall applications due to the internal fixed failure plane used by Rankine.

The application of Rankine "state of stress" earth pressure theory and fixed failure plane for wall batter $\leq 10^{\circ}$ is favored by the transportation agencies (AASHTO and FHWA) and is described in recent editions of the *AASHTO LRFD Bridge Design Specifications*.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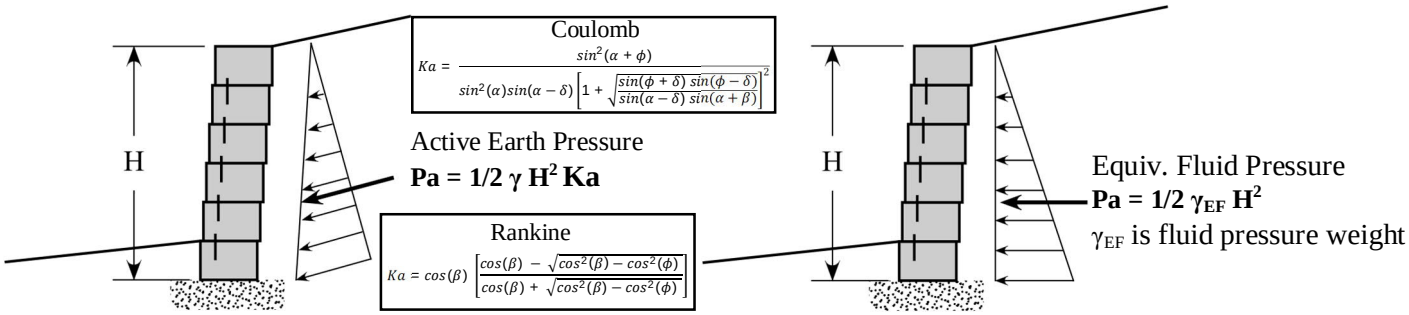
# Equivalent Fluid Pressure

It is common for structural and geotechnical engineers to define the active earth pressure loading for simple retaining wall structures in terms of equivalent fluid pressure such as 40 pcf for ease of calculation. Many design codes define minimum equivalent fluid pressures as a means for establishing a simple retaining wall design criteria without site specific analysis. Since an active earth pressure calculation without considering surcharges or complex loadings yields a simple triangular earth pressure distribution, the similarity to a fluid pressure analysis at some equivalent weight is reasonable.




**Active Earth Pressure** **Equivalent Fluid Pressure**

The limitation of an equivalent fluid pressure analysis is that it is independent of structure geometry such as wall batter and tiered wall configurations, does not permit the proper analysis of surcharge conditions such as broken back slopes and dead/live load combinations, and it avoids complicated stability analysis conditions such as compound failure planes and global stability.

The benefit of equivalent fluid pressure analysis is that it typically creates a easily understood design loading regardless of structure geometry and soil design properties. For example, a heavily battered retaining wall design (10°+) utilizing high assumed soil strengths ($\phi$ = 36°) may result in very low calculated earth pressures (equiv. fluid pressure = 20 pcf) which may be unrealistic and depends upon very favorable conditions to perform adequately. Use of soil cohesion can also create unrealistically low calculated soil pressure if utilized.

Calculated equivalent fluid pressure (pcf) vs. wall design criteria is compared below. Note the significant variance depending on soils strength, wall batter, backslope geometry, and design method compared to simple equivalent fluid pressure.

| | | Rankine-Vertical | | | | Coulomb-1:8 Batter | | | |
|---|---|---|---|---|---|---|---|---|---|
| Backslope > | | Level | 4h:1v | 3h:1v | 2h:1v | Level | 4h:1v | 3h:1v | 2h:1v |
| $\phi$ angle | 34° | 34 | 37 | 39 | 49 | 25 | 29 | 32 | 39 |
| | 30° | 40 | 44 | 48 | 65 | 30 | 36 | 40 | 52 |
| | 26° | 47 | 53 | 59 | 90 | 36 | 45 | 50 | 90 |

Note: Equivalent fluid pressure based on soil weighing 120 pcf. Low $\phi$ angles and steep backslopes create high pressures and do not permit equation solution to earth pressure.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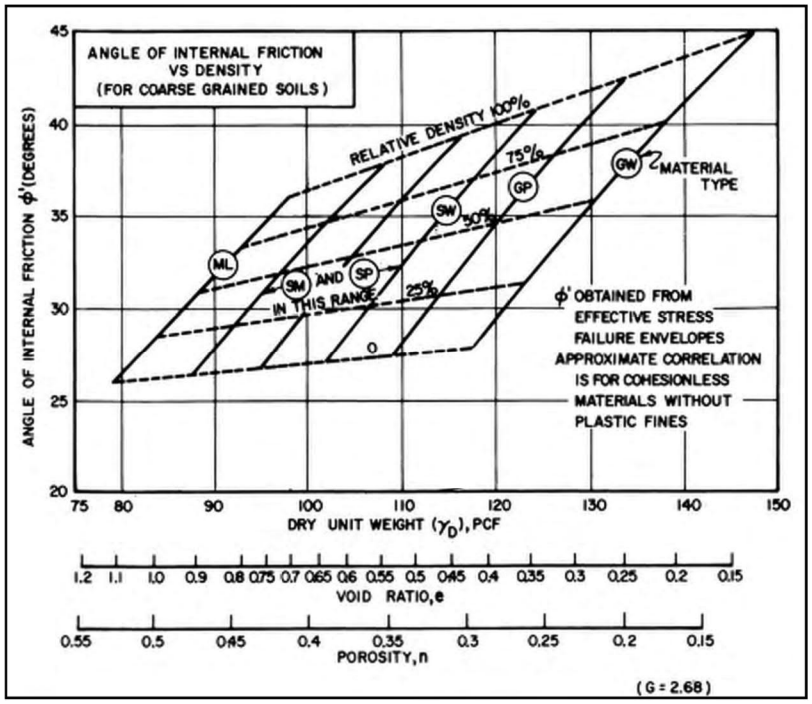
# Compaction and Strength of Soils

Proper placement and compaction of soils is essential to the successful performance of retaining wall structures. Post construction settlement is an obvious concern with poorly compacted materials as well as excessive lateral wall movement and/or insufficient shear strength to perform as intended. Soils must be compacted in lifts to achieve maximum soil shear strength and validate the design.

The chart below indicates a relationship between peak shear strength and soil density for cohesionless granular materials with no plastic fines as shown. As the relative density of the material is increased, significant gains are realized in shear strength. Therefore, it is necessary that levels of compaction and lift thickness be specified and obtained during construction to insure proper structure performance in accordance with the design.




**Correlations of Strength Characteristics for Granular Soils**

(Ref. from NAVFAC DM 7.01)

Clean granular soils are much more tolerant to variations in the placement and compaction process than the finer silts and clays which require close monitoring of moisture content and compaction procedures. The standard minimum soil density specification for the structural fill behind Keystone walls is indicated below. Higher values may be mandated by project specifications and local practice. Moisture content is typically limited to optimum moisture with finer grained materials to avoid overly "wet" soils being placed and compacted behind the wall which can causing wall alignment issues during construction due to temporary pore pressure conditions and "pumping" backfill soils.

| Test Criteria | Minimum % | Moisture % | ASTM Method | AASHTO Method |
|---|---|---|---|---|
| Standard Proctor | 95% | +0, -3 | D698 | T-99 |
| Modified Proctor | 90-95% | +0, -3 | D1557 | T-180 |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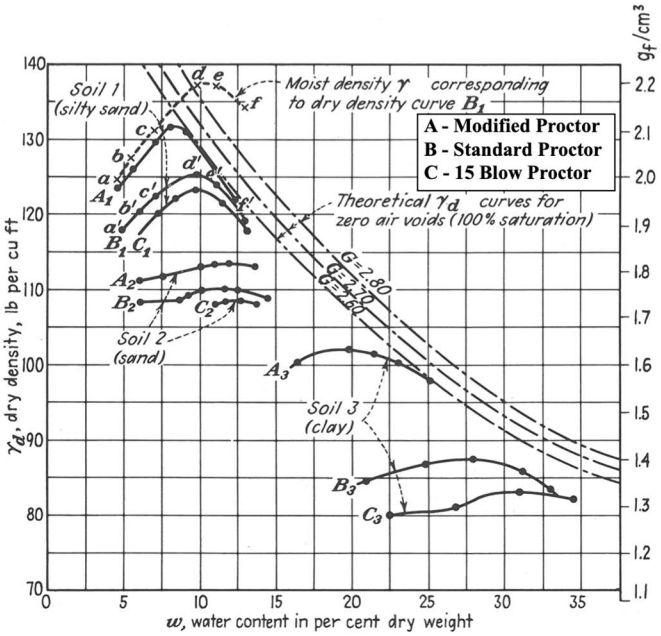
# Soil Density - Standard vs. Modified Proctor

Reinforced soil structures routinely specify that all soils be compacted to a minimum of 95% of the maximum density determined by ASTM D698 - *Standard Proctor Density* for conformance with the design. However, *Standard Proctor* density criteria is typically utilized in the Eastern US whereas *Modified Proctor* density criteria is typically utilized in the Western US which can create some conflicting specifications. The goal is always a well compacted structural fill with the appropriate density testing quality control regardless of method.

Research has been done showing the relationship between *Standard* and *Modified Proctor* density testing for different soils types as indicated below:




**Moisture-Density Relationships**

(Ref. Tschebotarioff - 1973)

**Characteristics of Three Soils**

| Type | Sand | Silt | Clay | LL | PI |
|---|---|---|---|---|---|
| Soil 1-Silty Sand | 80% | 15% | 5% | 17 | 1 |
| Soil 2-Sand | 92% | 5% | 3% | NP | NP |
| Soil 3-Clay | 10% | 28% | 62% | 68 | 47 |

**Summary of Soil Compaction Data**

| Type | Dry Density, pcf | | | Opt. Moisture, % | | |
|---|---|---|---|---|---|---|
| | A | B | C | A | B | C |
| Soil 1-Silty Sand | 132 | 125 | 123 | 8 | 10 | 10 |
| Soil 2-Sand | 113 | 110 | 108 | ind | ind | ind |
| Soil 3-Clay | 102 | 88 | 83 | 20 | 28 | 31 |

**95% Standard vs Mod. Proctor Comparison**

| Type | Standard | 95% Standard | % Modified |
|---|---|---|---|
| Soil 1-Silty Sand | 125 pcf | 119 pcf | 90% |
| Soil 2-Sand | 110 pcf | 105 pcf | 93% |
| Soil 3-Clay | 88 pcf | 84 pcf | 82% |

It is obvious from this limited data that a simple conclusion can not be drawn but some general guidelines can be established when using *Modified Proctor* density testing vs. *Standard Proctor* testing for quality assurance testing of reinforced soil structures:

- 90% - 93% of *Modified Proctor* density is roughly equivalent to the specified 95% *Standard Proctor* density for granular soils. Finer grained soils (ie: silts & clay) can be very responsive to compactive effort and moisture content thus the relative density difference may be significantly greater.
- *Modified Proctor* testing typically requires a lower optimum moisture content for achieving maximum density (curve shifts to the left) which is desirable for Keystone retaining wall construction and performance especially with silty and clayey soils. It also helps avoid pumping conditions.
- The density difference between *Modified Proctor* and *Standard Proctor* density testing appears to increase with the percentage of fines in the soil matrix while the optimum moisture content decreases. It may be prudent to utilize 92% or 95% of *Modified Proctor* as the minimum density and optimum moisture content when working with finer grained soils such as clays for best results.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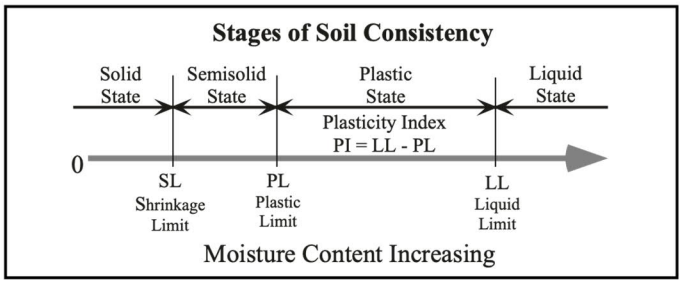
# Silt/Clay Soils - Atterberg Limits

Reinforced soil structures perform best when constructed with granular backfill material. Predictable shear strength, low strain and consolidation characteristics, non-plastic behavior, and better internal drainage make granular soils the superior wall building material. However, most of the site soils in the US consist of less select materials which can challenge the engineer and contractor when utilizing these lower quality site soils due to project imposed economic constraints.

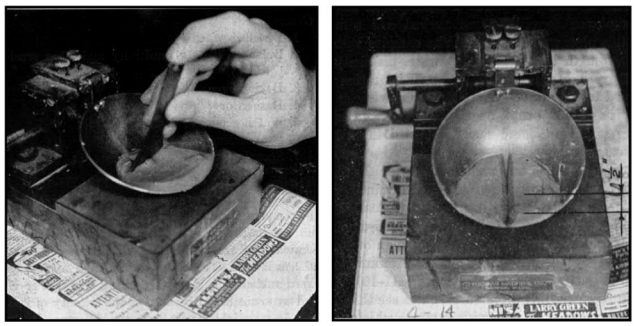
Atterberg limits are a set of index tests performed on fine grained silt/clay soils to determine the relative activity of the soils and their relationship to moisture content. The liquid limit, plastic limit, and shrinkage limits define the relative stages of behavior as indicated below when the soil moves from the solid state to a liquid state. The soil classification of fine grained soils based on these limits is also shown below.

The USC limits of "good clay" vs "bad clay", if there is such a thing, is defined as a Liquid Limit less than 50 and Plasticity Index less than 20 for silts and clays (ML/CL designations). The materials classified as CH, MH, and OH are typically unsuitable for reinforced wall construction and should be avoided. Keystone recommends limiting the LL < 40 and PI < 15 when dealing with plastic soils whenever possible to avoid the transitional zone of normal soil classification.

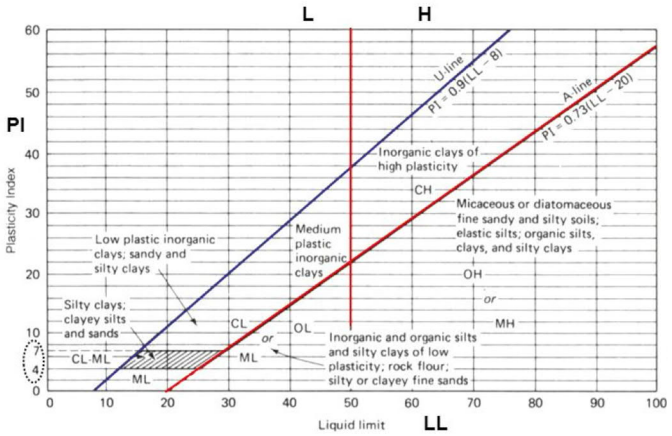
Construction of reinforced soil structures with plastic soils must always proceed carefully due to the potential for wall construction and performance problems and possible long term creep of the soil. Soil drainage issues must also be carefully evaluated. Atterberg limit testing is mandatory for all clay soils and placement and compaction must be carefully monitored.




**Atterberg Limits**

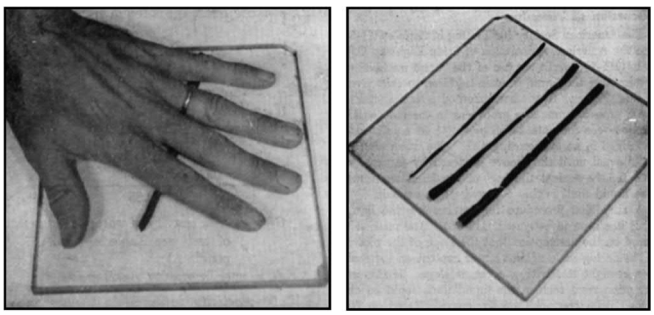


**Liquid Limit Test**




**Plasticity Chart**

(Ref. Holtz and Kovacs, 1981)



**Plastic Limit Test**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# Backfill Soil Specifications

The successful performance of reinforced soil wall structures is largely attributable to the quality of the soils involved and the contractor's experience with soils and structural fill construction. Many wall performance problems can be traced back to the quality, strength, moisture, and density of the in-situ or compacted backfill soils.

The US Bureau of Public Roads introduced the first soil classification system in 1928 attempting to classify soils based on engineering behavior with designations of A-1, A-2, etc. After this system had been used for about 15 years, AASHO (AASHTO) reviewed and adopted a similar system with designations of A-1-a, A-2-4, etc. In 1952, the Unified Soil Classification System (USC) with designations of GW, SM, ML, CL, etc. was adopted by the US Corps of Engineers and the Bureau of Reclamation. The chart references below provides a quick summary of the "granular" materials for use in structures:

**"Granular" Backfill Soil Parameters**

| Group | Backfill Classification | Top Size 100% Passing | #4 Sieve % Passing | #40 Sieve % Passing | #200 Sieve % Passing | Plasticity Index (PI) | Liquid Limit (LL) |
|---|---|---|---|---|---|---|---|
| AASHTO | MSE Select | 4 " max | - | ≤ 60% | ≤ 15% | ≤ 6 | - |
| AASHTO | A-1-a | 3" (tested) | - | ≤ 30% | ≤ 15% | ≤ 6 | - |
| AASHTO | A-1-b | 3" (tested) | - | ≤ 50% | ≤ 25% | ≤ 6 | - |
| AASHTO | A-2-4 | 3" (tested) | - | n/a | ≤ 35% | ≤ 10 | 40 max |
| AASHTO | A-2-6 | 3" (tested) | - | n/a | ≤ 35% | ≤ 40 | 40 max |
| ASTM-USC | GW, GP | - | ≤ 50% | - | ≤ 5% | NP | NP |
| ASTM-USC | GM, GC | - | ≤ 50% | - | 12-50% | 4-20 | 50 max |
| ASTM-USC | SW, SP | - | ≥ 50% | - | ≤ 5% | NP | NP |
| ASTM-USC | SM,SC | - | ≥ 50% | - | 12-50% | 4-20 | 50 max |
| *ASTM-USC* | *ML, CL* | - | - | - | *≥ 50%* | *"A" line* | *50 max* |

The amount of fine material (fine sand, silt and clay) as defined by the #40 and #200 sieves is generally a good indicator of favorable engineering and construction properties. The properties of the fine material as defined by its Atterberg limits (PI and LL) has also been a good indicator of a soil's engineering and construction properties. These soils properties should be clearly defined and limited by specification for any wall installation that is counted on to serve a structural purpose such as supporting a parking lot, building, or roadway.

**Recommended Backfill Parameters (Geogrid)**

| Designation | Top Size 100% Passing | #40 Sieve % Passing | #200 Sieve % Passing | Plasticity Index (PI) | Liquid Limit (LL) |
|---|---|---|---|---|---|
| Select Backfill | 2" | ≤ 60% | ≤ 15% | ≤ 6 | - |
| Semi - Select Backfill | 2" | - | ≤ 35% | ≤ 10 | ≤ 40 |
| Tolerable Silt/Clay | 2" | - | ≤ 65% | ≤ 15 | ≤ 40 |
| Unacceptable Silt/Clay | 2" | - | ≥ 65% | >20 | > 50 |

**Note:** It is easy to consider poor site soils for economic reasons but is not so easy to construct with such soils, nor to expect high performance from marginal soils even though a design can be done on paper. The Owner should be advised and make informed choices regarding potential issues from poor site soil use.


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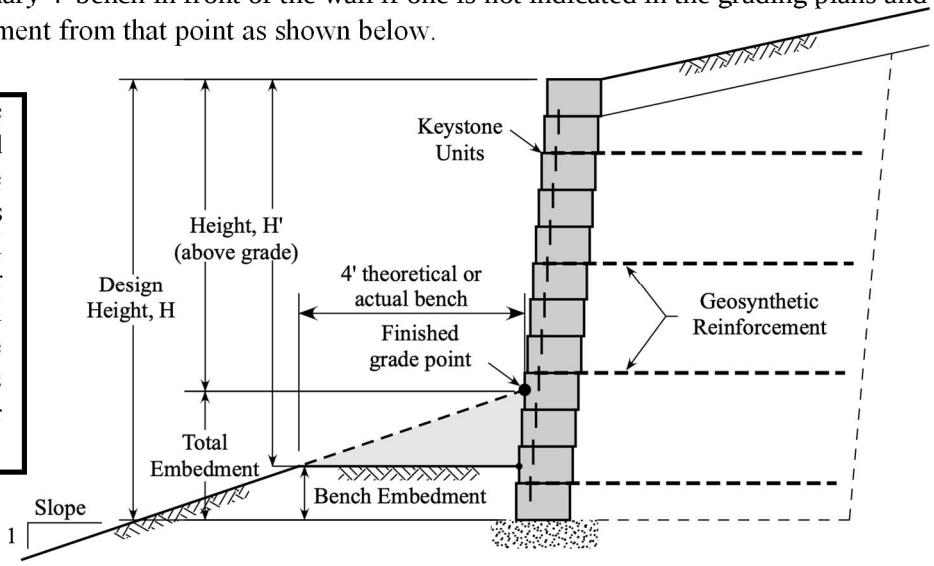
# Wall Embedment

The foundations of all retaining wall systems are placed a specified distance below finished grade to provide adequate erosion protection, frost protection, foundation bearing capacity, and overall global stability when slopes are involved. The design of flexible modular retaining wall systems is not as concerned with frost related issues as with rigid structures but erosion protection, local bearing capacity, and global stability issues must be evaluated for each design situation encountered.

The minimum practical embedment for any small wall structure with a level toe slope is 6" or one block unit below finished grade. As a wall gets taller or is placed in less stable sloping toe conditions, the embedment must be increased to satisfy stability requirements. It is easiest to understand minimum wall embedment criteria when a typical cross section is evaluated.

Typically, a finished grade point is established where the ground in front of the wall intersects the wall face alignment. Sometimes the grading accounts for a level bench in front of the wall but most of the time the finished grade is the intersection of the existing or proposed ground slope with the wall face. It is good practice to construct an imaginary 4' bench in front of the wall if one is not indicated in the grading plans and establish the minimum embedment from that point as shown below.

The recommendations in the table are general in nature and do not replace a comprehensive stability analysis in those areas with erosion or scour, poor soil conditions, submerged toes, or steep toe slopes. Special consideration should always be given to man-made fill slopes which can exhibit poor structural performance.




**Typical Wall Embedment Section**

## Various Embedment Criteria

| Toe Slope Condition | Minimum Embedment | |
|---|---|---|
| | NCMA | AASHTO |
| Horizontal (walls) | H'/20 | H'/20 |
| Horizontal (abuts) | H'/10 | H'/10 |
| 3H:1V | H'/10 | H'/10 |
| 2H:1V | H'/7 | H'/7 |
| 1.5H:1V | - | H'/5 |
| Minimum | 0.5' | *2.0' |

*provision for 1' min. embedment exists in current AASHTO code

Note: NCMA bases minimum embedment on exposed height, H', and does not acknowledge bench considerations. AASHTO bases minimum embedment on total height and requires a 4' bench for sloping toe conditions in addition to the table requirements. FHWA is somewhere in between in its recommendations.

## Suggested Wall Embedment

| Toe Slope Condition | Bench Embedment | Total Embedment |
|---|---|---|
| Level | 10% H' | 10% H' |
| 3H:1V | 10% H' | 1.3' + 10% H' |
| 2H:1V | 10% H' | 2' + 10% H' |
| 1.5H:1V | 10% H' | 2.7' + 10% H' |

Note: H' could be measured from finished grade or bench elevation depending on if bench is actually constructed or not.

(Note: 10% of exposed wall height is a good rule of thumb provided that toe slope conditions are properly evaluated and bench construction is assumed. There can be arguments for more or less embedment with sloping toe conditions which have to be determined on a project specific basis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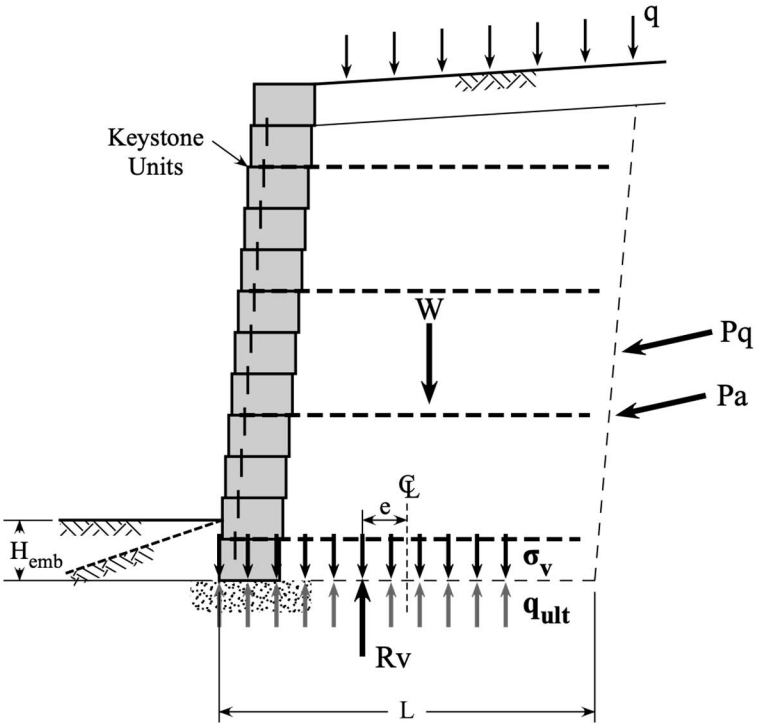
# Bearing Capacity

Many soil reports and building codes tend to dictate maximum bearing pressures that may be placed on certain soil types for sake of simplicity with little regard for the specific structure involved and the relevant theory of soil mechanics being applied to the site soil conditions. A typical example of this is the "3,000 psf" maximum bearing pressure requirement unilaterally being applied to all structures, even though the bearing capacity of soils increase with footing width and depth due to increasing confining pressure and stability.

This maximum bearing pressure issue can be a "compliance" or interpretation problem when applied to larger reinforced soil structures which place high earth loads on the foundation due to the height of fill involved. A 20' tall soil structure calculates over 3,000 psf applied bearing pressure yet calculates high bearing capacity safety factors when the site specific geometry and soil conditions are evaluated.




**Typical Wall Section**

**Foundation Soil**

$\phi_f$ - shear strength

$c_f$ - cohesion

$\gamma_f$ - unit weight

**Vesic/Meyerhof Equations**

$N_q = e^{\pi \tan\phi} \tan^2(45+\phi/2)$

$N_c = (N_q\text{-}1) \cot(\phi)$

$N_\gamma = 2(N_q+1) \tan(\phi)$

**Bearing Capacity Factors (thru Vesic)**

| $\phi_f$ | Nc | Nq | Nγ |
|---|---|---|---|
| 20° | 14.8 | 6.4 | 5.4 |
| 22° | 16.9 | 7.8 | 7.1 |
| 24° | 19.3 | 9.6 | 9.4 |
| 26° | 22.3 | 11.9 | 12.5 |
| 28° | 25.8 | 14.7 | 16.7 |
| 30° | 30.1 | 18.4 | 22.4 |
| 32° | 35.5 | 23.2 | 30.2 |
| 34° | 42.2 | 29.4 | 41.1 |
| 36° | 50.6 | 37.8 | 56.3 |

Note: Bearing capacity can be reduced by up to 50% if toe is sloping or below water table.

## Applied Bearing Pressure

Vertical Forces, $R_v = \Sigma W + \Sigma q\ (+ \Sigma P_v)$

Applied Pressure, $\sigma_v = \dfrac{R_v}{L - 2e}$

## Bearing Capacity

(Level Toe)

$$q_{ult} = c_f N_c + H_{emb}\gamma_f N_q + 0.5(L\text{-}2e)\gamma_f N_\gamma$$

This term may be omitted if the soil in front of wall is to be ignored such as AASHTO suggests.

## Factor of Safety

$$FS_{bc} = \frac{q_{ult}}{\sigma_v} > 2.0 - 2.5$$

Note: Factor of safety for bearing capacity is typically 3.0 for rigid structures but is reduced to 2.0 (NCMA) or 2.5 (AASHTO) for flexible soil structures. LRFD works in a similar manner for MSE walls with CDR > 1.0 after appropriate load and resistance factors are applied.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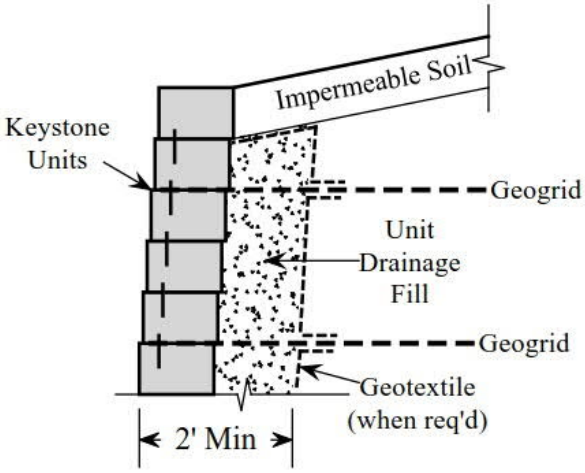
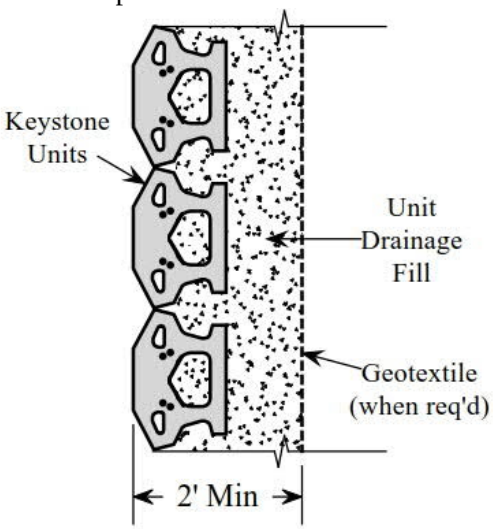
# Unit Drainage Fill

Unit drainage fill is defined as a free draining aggregate material such as ASTM designation No. 57 or 67 stone which is small enough (1" minus clean material) to easily fill unit cores and the gaps between units while containing minimal fine material (sands and silts) that could pipe through wall joints from occasional water flow. Unit drainage fill can be used in conjunction with geotextile filter fabrics to provide positive filtration and soil retention, most commonly in areas where water flow is expected such as with detention basin and flood plain structures.




## Unit Drainage Fill Section




## Unit Drainage Fill Plan

Unit drainage fill provides significant technical benefit for modular wall performance and construction:

- Prevents the buildup of hydrostatic pressures near the wall face through a 2 foot deep porous drainage zone.
- Provides a non-frost susceptible zone within and directly behind the wall units.
- Provides a high friction and easily compacted material within and behind the wall units that enhances facing zone stability.
- The unit drainage fill placed within and between units improves inter-unit shear and geogrid connection strength for units with thru cores and tapered sides.

Unit fill drainage material is typically described as a 1/2" - 3/4" clean stone (1" minus or No. 57 stone are common). While many granular materials can be described as "free draining", the following gradation is recommended by Keystone based on experience:

| Sieve | % passing |
|---|---|
| 1" | 100 |
| 3/4" | 75 - 100 |
| #4 | 0 - 10 |
| #50 | 0 - 5 |

The intent of this specification is to limit the top size to 1" and restrict the sand and silt component to less than 10% to avoid migration of fines and for ease of placement and compaction in confined spaces.

Note: All Keystone Unit connection testing is performed with unit cores completely filled with unit drainage fill. Therefore, replicating this in the field is important to unsure the connection test data used in the design is consistent with the constructed structure. For this reason, Keystone recommends placing unit drainage fill after each consecutive course. If additional courses are placed prior to filling unit cores, it will be difficult to properly fill unit cores, especially when geogrid layers are present which will prevent core fill from entering the lower courses.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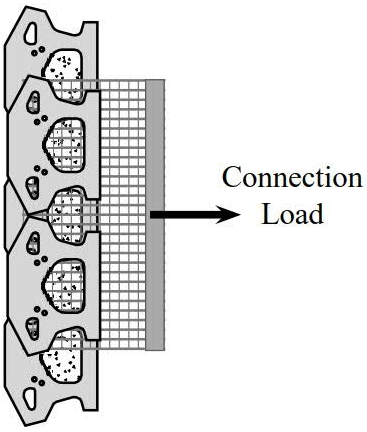
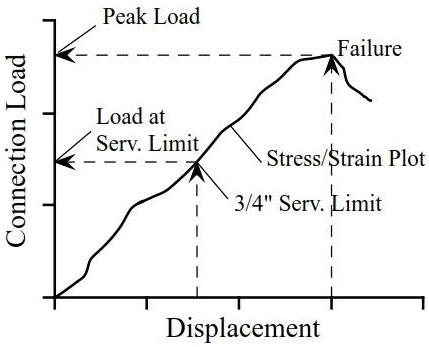
# Connection Strength Testing

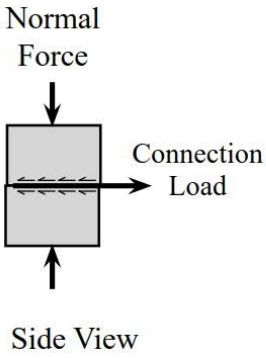
The connection capacity between geosynthetic soil reinforcement and a modular concrete wall unit is a complex interaction that can only be determined by laboratory testing. NCMA proposed a test method, SRWU-1, now ASTM D6638, which outlines the accepted practice for connection testing and evaluation of the results which is utilized by Keystone. A schematic of the connection testing setup and laboratory developed load curves are shown below.



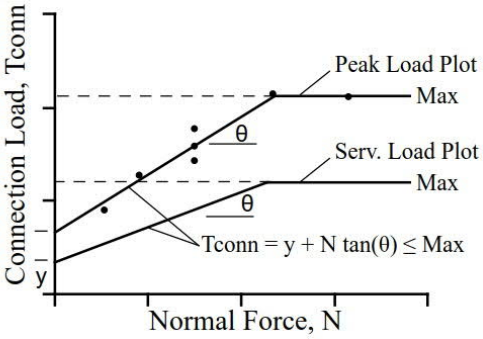

**Top View**




**Load Test at One Normal Force**




**Connection Test Setup**




**Multiple Load Tests at Varying Normal Loads**

Full laboratory connection testing has been performed with Keystone Pinned and Lip/Lug units and several different geosynthetic soil reinforcement manufacturers. Note that connection curves can also be bi-linear and have two slopes vs. one shown in figure above. This data is utilized in each wall design to determine the maximum permissible connection value that can be utilized at each reinforcement level.

Note: The increased depth of the Standard III unit (18” to 21” depth) has considerably more connection capacity than the Compac III unit ( 12” depth) and should be considered where maximum design safety and high performance is required. Connection capacity is a limiting factor in many taller wall designs and should be carefully evaluated to conform with design standards.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# Connection Strength Discussion

The evaluation of the structural connection between geosynthetic soil reinforcement and modular block retaining wall facing system has been a significant design consideration since the publication of the 1993 National Concrete Masonry Association (NCMA) Design Manual for Segmental Walls and the American Association of State Highway and Transportations Official's (AASHTO) Standard Specifications for Highway Bridges (design of mechanically stabilized earth (MSE) structures).

Structural and civil engineers have become acutely aware of the need to properly evaluate "connections" as a result of the Hyatt walkway collapse in Kansas City many years ago. The authors of these documents recognize that the design of a structural system must be comprehensive and include an analysis of all its structural components, especially those items not easily determined such as the connection strength between a geosynthetic reinforcement and a wall facing system and the wall facing stability during the construction process.

These documents require that the connection strength between a specific geosynthetic reinforcement and specific wall facing system be evaluated in a laboratory for its ultimate strength and strain characteristics under conditions that simulate the field installed condition. The load capacity of the connection at a specific location is compared to the maximum calculated load in the soil reinforcement and a factor of safety (FS) or capacity-demand ratio (CDR) is calculated. This connection strength calculation is made at each reinforcement level and a minimum FS >1.5 or CDR > 1.0 against rupture must be maintained. AASHTO specifications require additional considerations for sustained load capacity over time in their guidelines.

The typical result of the connection strength analysis in taller walls is that the geosynthetic reinforcement to modular wall unit connection controls the wall design process and limits the maximum tensile load that wall system reinforcement can accept at various levels. The designer must then utilize stronger soil reinforcement and/or closer vertical spacing of the soil reinforcement to resist or lower the tensile loads in each element in order to maintain acceptable connection factors of safety in accordance with the appropriate design methodologies.

Since the connection strength analysis can be a limiting design factor and requires additional soil reinforcement costs to satisfy the required design standards, those not skilled in retaining wall design sometimes ignore, neglect, or down play the connection strength evaluation as a means of reducing cost, increasing competitive position, or otherwise hiding a potential structural limitation of the proposed retaining wall system.

Keystone Retaining Wall Systems has laboratory tested all major geosynthetic soil reinforcement types with the Keystone Pinned and Lip/Lug series wall units and will continue to evaluate the connection strength requirements of each structure as required by good engineering practice and design standards. The connection strength evaluation is an integral part of the design process and can not be neglected.


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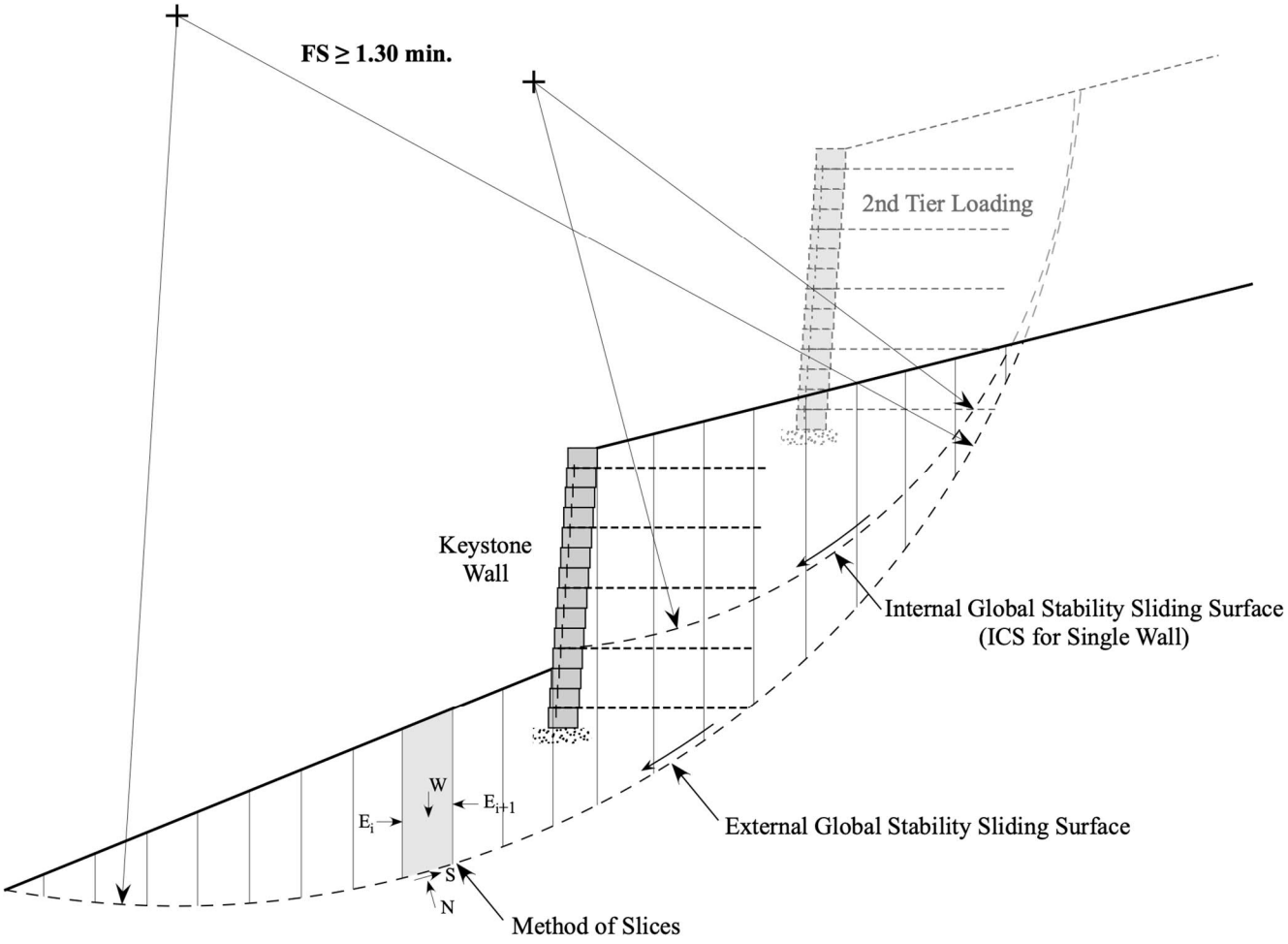
# Global Stability

A simple sliding and overturning analysis may be adequate for many simple retaining wall structures. However, an overall or global stability analysis is required for those more complex structures involving slopes, ground water conditions, poor soils, and/or tiered wall sections. Global stability analysis looks at a rotational or compound failure mechanism of walls and slopes which is significantly different than a simple sliding and overturning analysis. Global stability analysis recognizes the inherent instability of walls on slopes and tiered wall configurations, and can also find potential failure planes through flexible wall systems when soil reinforcement spacing and length is inadequate.




**Global Stability Section**

Global stability analysis is best accomplished through computer modeling with the aid of commercially available slope stability software such as G-Slope, Slope-W, and UTexas4 which can include soil reinforcing elements and perform Bishop and Janbu methods of analysis. Global stability analysis is very sensitive to soil design parameters and requires proficiency with proper modeling techniques and soils evaluation to arrive at reasonable answers and solutions.

A minimum stability factor of safety of 1.3 is typically required for retaining structures. However, this factor may be increased to 1.5 for supporting critical wall structures such as bridge abutments per the AASHTO code.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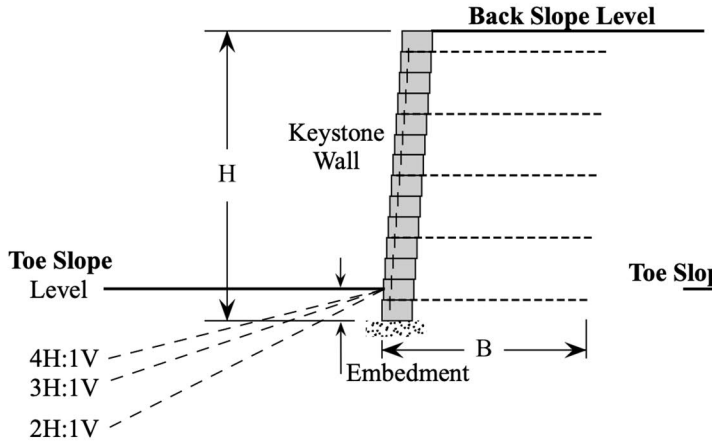
# Single Wall - Slope Stability Ratios

The following figures and graphs provide a guide to the relationship between walls and slopes and the B to H ratio required to estimate basic global stability requirements for simple ϕ only soil strength criteria. Slopes 2H:1V and greater require special attention to soil design parameters. Slope Stability Ratios are not to be used for construction.

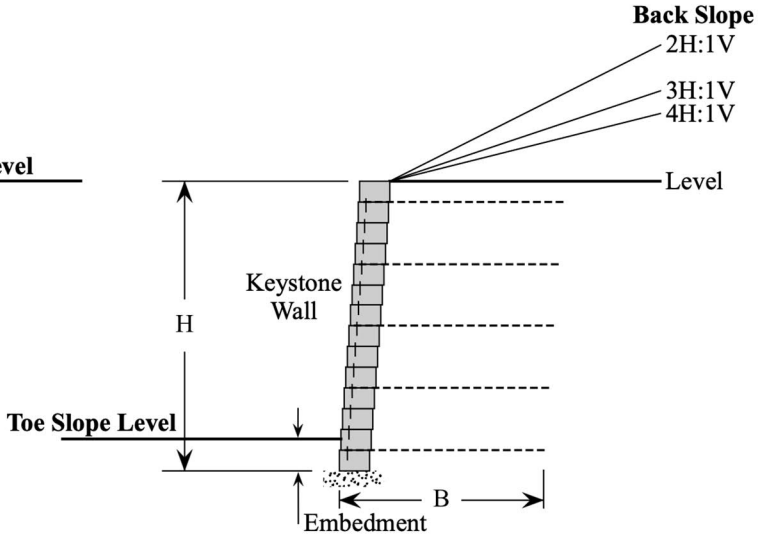
**Assumptions of Simple Stability Analysis**

FS > 1.3 min, Bishop,
No signifcant surcharge, γ = 120 pcf, all soils same strength
Vertical reinforcement spacing = 2 ft max.
Lowest reinforcement = 1 ft from bottom max.
LTDS of Reinforcement: > 1,300 plf - Upper 10 ft.
> 2,000 plf - Next 10 ft, etc.



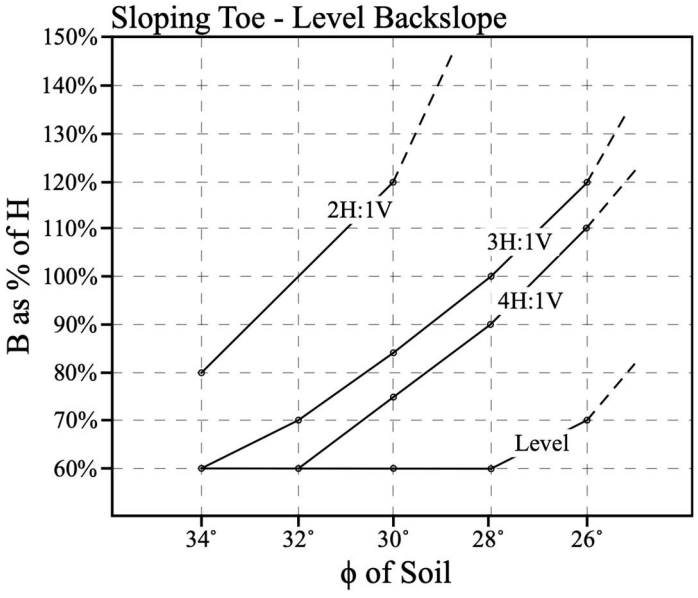

**Min. Embedment for Toe Slope**

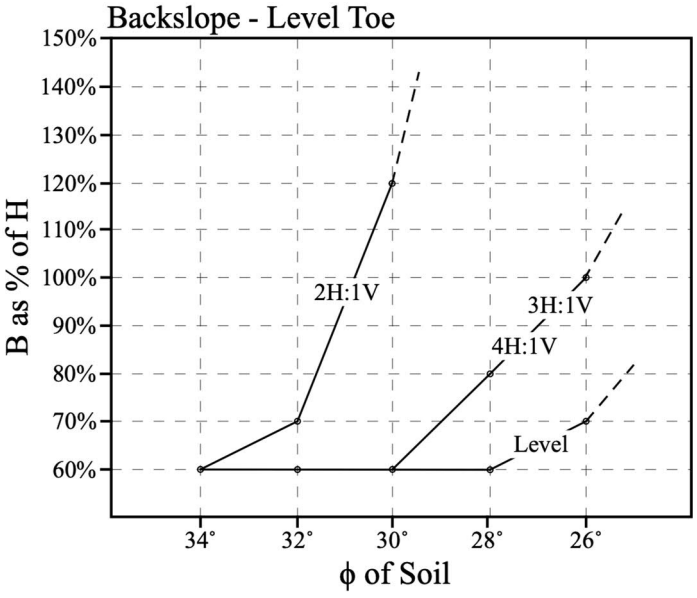
Level 10% H
3H:1V 1.3' + 10% H
2H:1V 2.0' + 10% H




**Min. Embedment for Back Slope**

Level 10% H
3H:1V 10% H
2H:1V 10% H




**Note: The Slope Stability Ratios are for preliminary estimation only and should not be used for construction without review by a qualified engineer.**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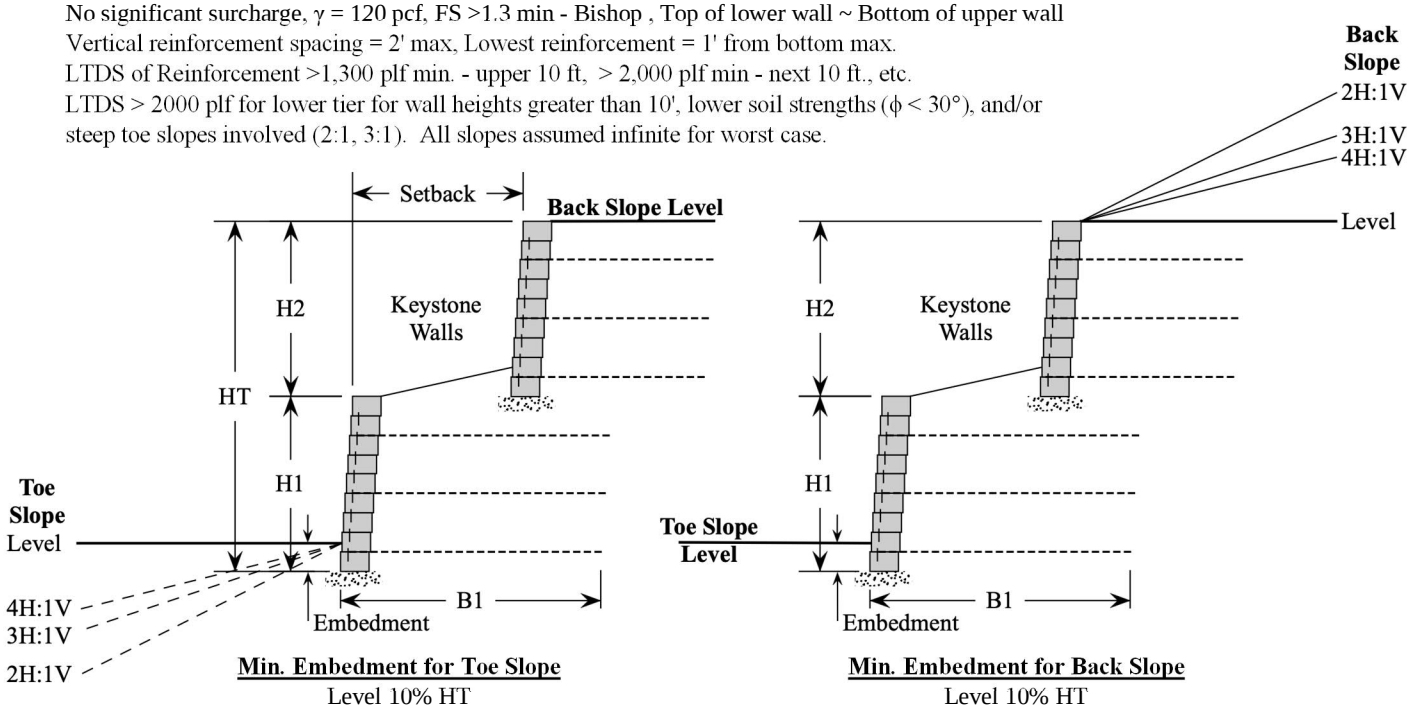
# Tiered Wall - Slope Stability Ratios

The following figures and graphs provide a guide to the relationship between tiered walls and slopes and the B1 to HT ratio required to estimate basic global stability requirements for simple ϕ only soil strength criteria. Slopes 2H:1V and greater require special attention to soil design parameters. Slope Stability Ratios are not to be used for construction.

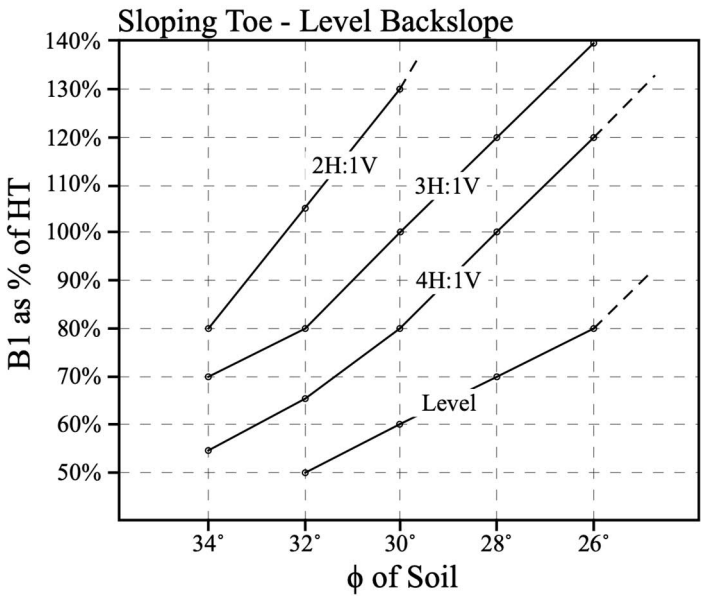
**Assumptions of Stability Analysis**

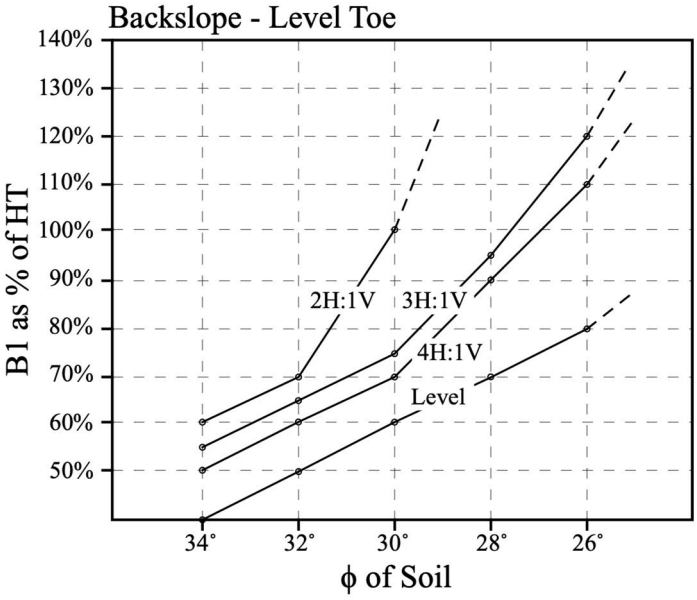
H1 ~ H2 ~ Setback. Note: Closer wall spacing is better for global stability, worse for tensile load.
No significant surcharge, γ = 120 pcf, FS >1.3 min - Bishop , Top of lower wall ~ Bottom of upper wall
Vertical reinforcement spacing = 2' max, Lowest reinforcement = 1' from bottom max.
LTDS of Reinforcement >1,300 plf min. - upper 10 ft, > 2,000 plf min - next 10 ft., etc.
LTDS > 2000 plf for lower tier for wall heights greater than 10', lower soil strengths (ϕ < 30°), and/or steep toe slopes involved (2:1, 3:1). All slopes assumed infinite for worst case.




**Min. Embedment for Toe Slope**
Level 10% HT
3H:1V 1.3' + 10% HT
2H:1V 2.0' + 10% HT

**Min. Embedment for Back Slope**
Level 10% HT
3H:1V 10% HT
2H:1V 10% HT




**Note: The Slope Stability Ratios are for preliminary estimation only and should not be used for construction without review by a qualified engineer.**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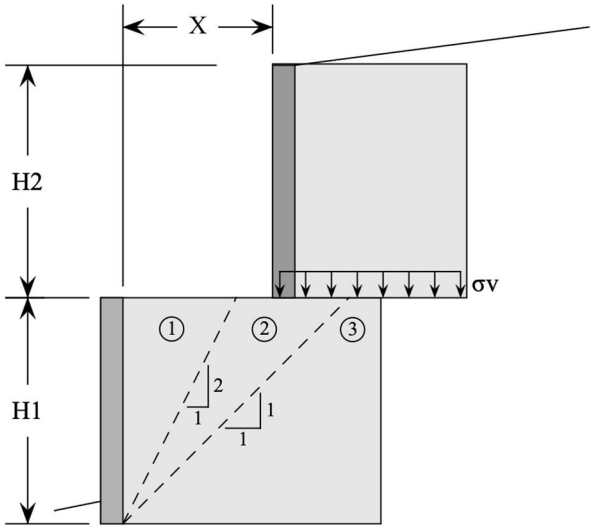
# Tiered Wall - Internal Analysis

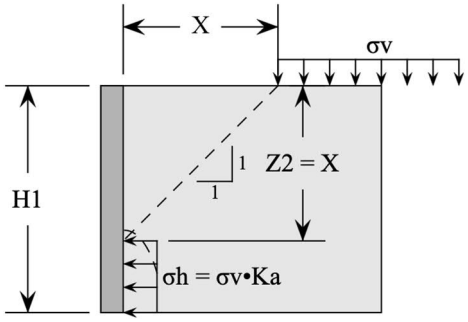
The external stability analysis for tiered or terraced wall structures is primarily accomplished by global stability analysis software used in conjunction with wall design software. Global stability analysis should also check for internal failure planes passing through the lower wall, insuring that the reinforcement is long and strong enough. However, determining the actual internal load distribution is another matter. The internal analysis of the lower tier(s) becomes considerably more difficult as there is little agreement on how upper walls actually surcharge lower reinforced soil walls.

A trial wedge approach is probably best suited for determining internal reinforcement loads on a level by level basis in tiered configurations but this method can be difficult to model and calculate without the aid of a special computer analysis. Approximation techniques like shown below can also be utilized but may be conservative due to the limitations of such approaches.

The figure below describes the three zones of influence and an approximation technique for distributing loads by superposition in addition to the normal earth pressure loads on the lower wall:




**Zones of Influence**




**Simplified Surcharge Load Approximation**

**Surcharge Zone Discussion**

① When the upper wall setback falls within Zone 1 (X < H1/2), the upper wall fully surcharges the lower wall and the lower wall could be designed as the bottom part of a larger wall keeping in mind that connection strength is affected by splitting the walls apart a short distance.

② When the upper wall setback falls within Zone 2 (H1/2 < X < H1), the upper wall surcharges the lower portion of the lower wall as indicated and the reinforcement design of the lower wall should account for the additional surcharge where applicable.

③ When the upper wall falls outside the 1:1 influence line drawn for the back of lower wall Zone 3 (X > H1), there is no direct internal surcharge on the lower wall and reinforcement lengths and strengths should be checked with a global stability analysis.

Note: This approximation or trial wedge method may not be appropriate for highly concentrated loads behind walls within Zone 1 and maybe Zone 2 that may be best suited for elastic analysis such as Boussinesq theory.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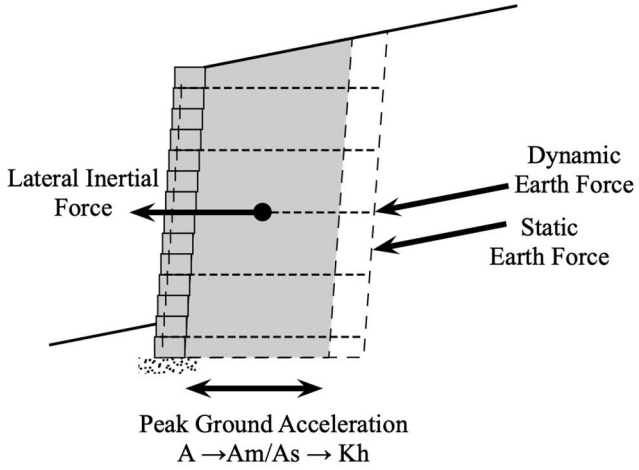
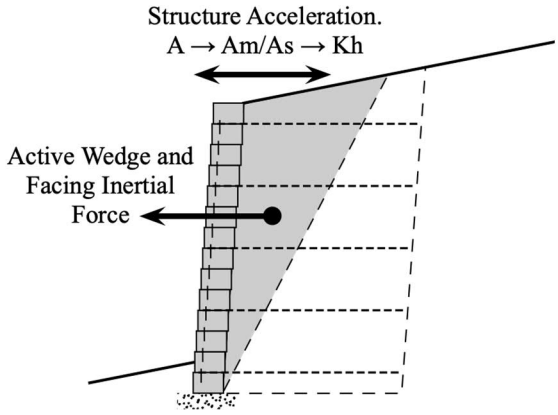
# Seismic Analysis

Mechanically stabilized earth (MSE) retaining structures, like Keystone Retaining Walls, have proven to be earthquake resistant due to the system's mass and inherent flexibility which permits minor yielding during a major seismic event. The lack of observed performance problems with retaining structures after major earthquakes has resulted in little attention being given to improving seismic design methods and codes compared to more sensitive building and bridge structures. AASHTO has taken the most aggressive approach in recent years and reduced the design loading for the seismic analysis of retaining structures and even suggesting that seismic analysis may not be required in zones with a A < 0.40g if no other considerations are present.

Most suggested seismic analysis methods describe a psuedo-static method of analysis based on the Mononobe-Okabe application of conventional earth pressure theory. However, the selection of the appropriate Kh value to use in the internal and external analysis based on displacement considerations is still not well understood. A schematic of psuedo-static analysis considerations is shown below as it pertains to soil reinforced structures. The specific calculation of the various load components varies between methods but the concepts are similar.




**External Stability**




**Internal Stability**

A seismic design must evaluate the combined loading condition of static, dynamic, and inertial forces acting on the structure, both externally and internally, and provide sufficient resistance to mitigate failure during the design event. It is customary to utilize 75% of the normal static design safety factors (ie; 75% of 1.5 min = 1.1 min) for the combined loading condition analysis. AASHTO LRFD has recently gone to a load factor equal to 1.0 for all external load components yet retaining partial load factors for the internal analysis.

Sliding, overturning, and bearing pressure are analyzed in the conventional manner including the additional driving components of dynamic earth pressure and structure inertial force. Peak bearing pressure and eccentricity can also be checked but there is no particular acceptance criteria for these items. Soil liquefaction and slope stability can also be a factor in seismic analysis which must be considered as part of the site geotechnical investigation.

Internally, the soil reinforcement strength, connection to the facing system, and soil pullout are checked to insure that rupture or pullout will not occur during the design event. Additionally, local stability of the upper units is checked to insure that the top of wall will not overturn as a small gravity structure.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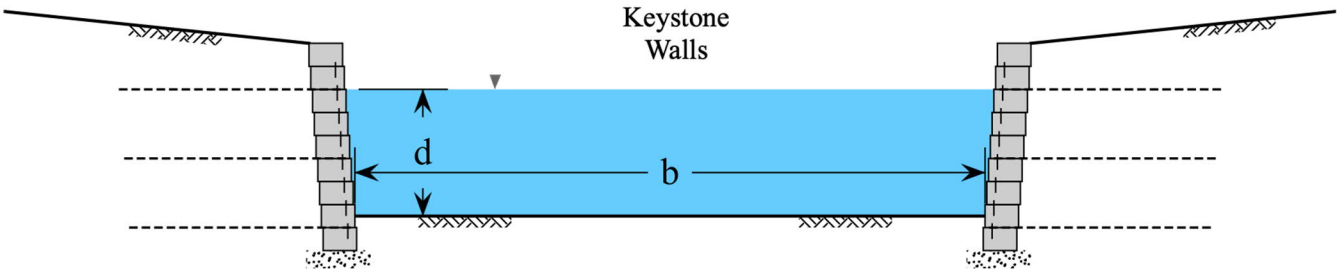
# Water Flow - Manning's Number

Keystone walls have been utilized for water channelization projects due to low cost and ease of installation as well as providing obvious technical and aesthetic benefits. Water resource engineers have always asked what the roughness coefficient or Manning's "n" value is for the tri-planer fractured face of a typical Keystone structural unit to insure that their flow calculations are correct.

A typical channel cross section and Manning's "n" values are provided below. Since only the tri-planer split-faced units were flume tested, we believe that straight split-faced units would provide slightly lower values due to less facial relief if required.




## Manning Equation, $V = 1.49/n\ R^{2/3}\ S^{1/2}$

Where: V = velocity (feet per second)

n = Manning roughness coefficient

R = hydraulic radious ( area / wetted perimeter)

S = slope of channel

## Manning's Roughness Coefficient, n

| Lining Category | Lining Type | n-value (d>2' depth) |
|---|---|---|
| Rigid | Concrete | 0.013 |
| | Grouted Rip Rap | 0.028 |
| | Stone Masonry | 0.030 |
| | Asphalt | 0.016 |
| | Keystone, tri-split | 0.023 |
| Unlined | Bare Soil | 0.020 |
| | Rock Cut | 0.025 |
| Rock Rip-Rap | 6 inch, D50 | 0.035 |
| | 12 inch, D50 | 0.040 |

Ref: Design Procedures for Channel Protection and Streambank Stabilization - IECA
1996 Water Effects on Keystone Units - Utah State 1991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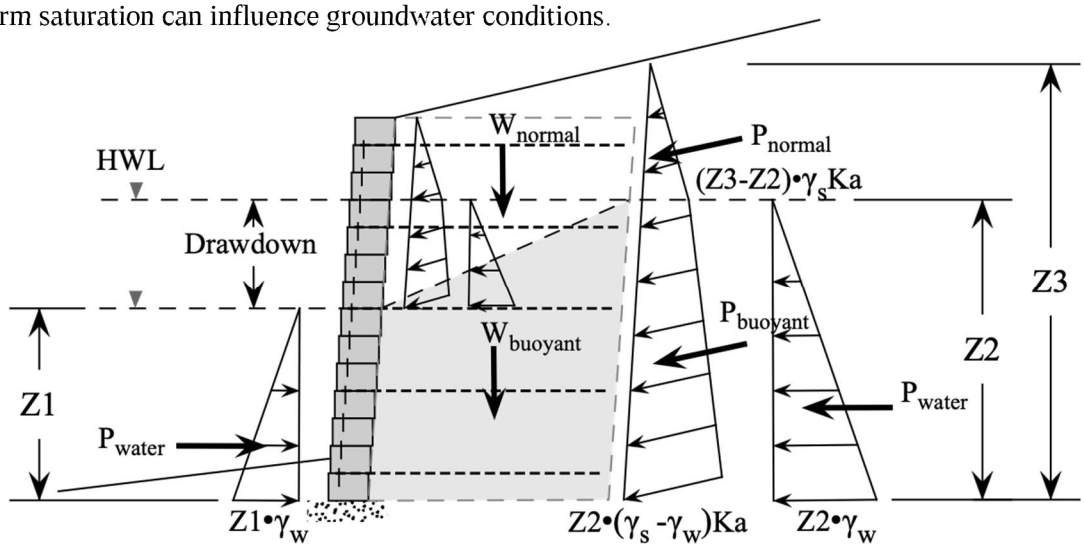
# Drawdown Analysis

Wall structures constructed adjacent to water can experience a wide range of conditions and instability as water levels rise and fall. The extremes may run from a simple retaining structure constructed along a relatively static pond to a flood stabilization project and where the structures are in the dry 99% of the time yet completely submerged a few times a year.

Clearly, the risk associated with water applications can be significant. Therefore, the design should be addressed in a comprehensive and conservative manner. Building codes do not typically address water applications due to their unique nature and expect that the engineer will provide the correct solution for the conditions based on standards of practice.

As a guide, excerpts from the 2015 AASHTO code says "For structures along rivers and streams, a minimum differential hydrostatic pressure equal to 3.0 ft of water shall be considered for the design .... or alternatively rapidly draining backfill material such as shot rock or open graded gravel can be used as backfill...". Keystone generally recommends free draining reinforced zone backfill such as #57 stone wrapped in a geotextile to the high water level for walls that would be subject to varying water levels.

It is important to properly address internal and external stability of a reinforced soil structure for the drawdown condition when applicable. Internally, the hydrostatic pressure differential can be eliminated by the use of free draining backfill. Externally, high water that effectively raises the groundwater table reduces the effective weight of the mass (buoyancy) and increases driving forces due to the water level imbalance. Global stability and foundation stability must also be reviewed against the varying water conditions when long term saturation can influence groundwater conditions.




**Typical Design Section**

**Drawdown Design Guidelines**

- 3' drawdown condition is a typical design requirement when required.
- Drawdown evaluation can utilize lower safety factor for combined loading analysis. (COE Guideline FS > 1.33).
- Drawdown condition affects wall between normal water and high water elevations as level is varied. Global stability condition can be worse with submerged toe.
- Liberal use of free draining backfill material minimizes internal drawdown conditions and associated pressure differentials.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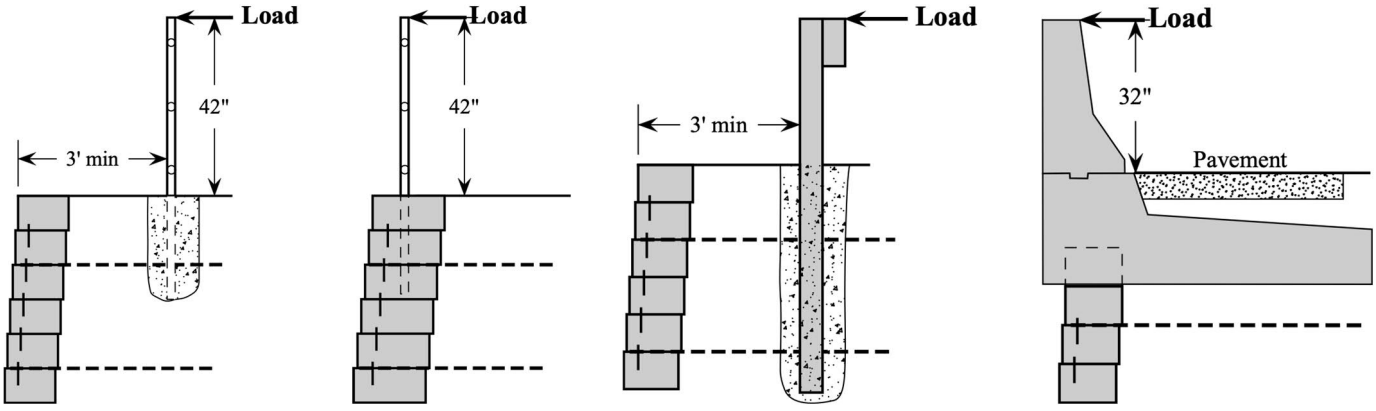
# Guard and Barrier Requirements




**Railing Offset** **Railing Direct Mount** **Guiderail** **Traffic Barrier**

## Introduction

Railing, fencing, guiderail, and traffic barrier requirements for retaining walls are not clearly defined in design codes nor are they properly addressed in many site plans. Many times guards and barriers are added as an afterthought which can become a costly and logistical issue when no provisions are made in the original retaining wall layout and site design.

Guards and barriers require a common sense approach by the site designer considering the proximity of a wall structure to people and traffic. Sufficient space must be reserved for such installations with modular wall systems accounting for wall batter and generally placing foundations behind the wall facing system. The design of these systems is a function of their anticipated use and location.

## Guards

IBC 2018: *"Guards shall be located along open-sided walking surfaces, including mezzanine, equipment platforms, aisles, stairs, ramps, and landings that are located more than 30" measured vertically to the floor or grade below any point with 36" horizontally to the open side."* The application of this provision to retaining walls is certainly not clear and there are exemptions for certain situations.

AASHTO: AASHTO is equally unclear as to when and where guards are required for retaining walls but transportation projects tend to be more clear in their bid documents though the actual design may not be.

| Railing/Guard Design Requirements | | |
|---|---|---|
| | **IBC 2018** | **AASHTO 2020** |
| Handrails and guards | 50 lbs/ft or 200 lb load (whichever governs) | 50 lbs/ft and 200 lb load (simultaneously) |
| Except: | | |
| 1 and 2 family | 200 lb load only | n/a |
| I-3, F, H, S occupancies (less than 50, not public accessible) | 20 lbs/ft or 200 lb load (whichever governs) | n/a |
| Vehicle Barrier | 6,000 lbs load (passenger vehicles) | 13,500 lbs - 175,000 lbs Impact (pseudo static load = 10,000 lbs typ) |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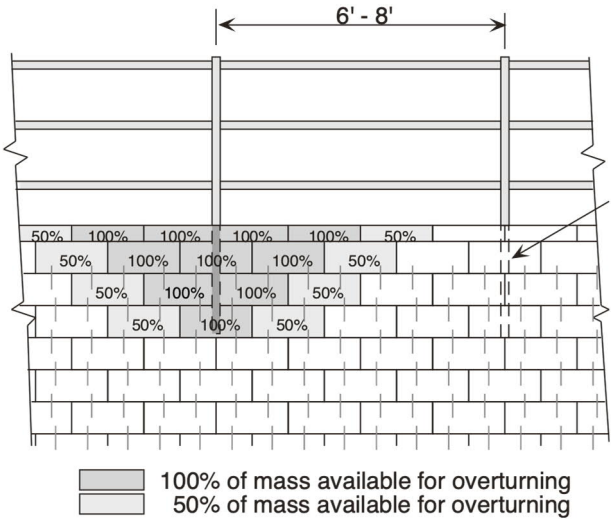
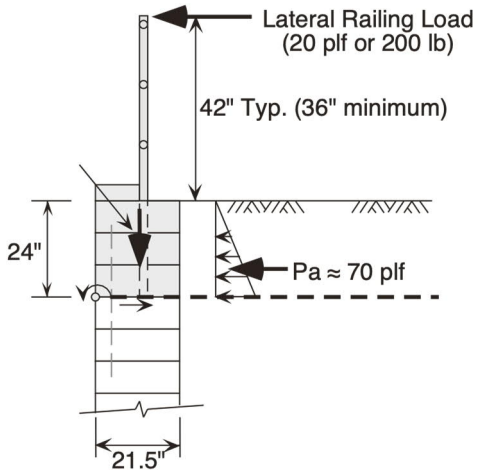
# Typical Railing Design

## Direct Mount - 20 plf—Standard Units




**Typical Railing Elevation**




**Typical Railing Section**

### Introduction

It is difficult for a railing design to satisfy structural design requirements when considering the direct mounting on or into the Keystone modular wall system. The small unit size and mass provides minimal resistance to overturning by itself so a number of units must be engaged to provide the required resistance. The Keystone Standard Unit is typically large enough to satisfy a 20 plf or 200 lb/post minimum UBC loading provided that the post is grouted into the upper three courses as shown.

### Railing Analysis

Shear resistance of Standard units (>1000 plf) exceeds the driving forces by a wide margin in gravity wall applications and is not a critical evaluation. Overturning at the top of wall (local stability) is the critical evaluation. A 200 lb point load on each post typically controls with a 20 plf design criteria. The higher 50 plf loading required by IBC for public facilities and by AASHTO for highway projects requires that the railing design and analysis be treated differently.

Driving Moments (20 plf or 200 lb point load)

Railing 200 lbs x 5.5' arm = 1100 ft-lbs
Soil 70 lb/ft x 0.67' x 2' = 94 ft-lbs
Total = 1194 ft-lbs

Resisting Moments (grouted posts, units with gravel)

6 units x 215 lbs/ea x 100% x 0.89' = 1148 ft-lbs
6 units x 215 lbs/ea x 50% x 0.89' = 574 ft-lbs
4 caps x 50 lbs/ea x 100% x 0.45' = 90 ft-lbs
2 caps x 50 lbs/ea x 50% x 0.45' = 23 ft-lbs
Total = 1835 ft-lbs

FSot = 1835/1194 = 1.53 ≥ 1.50 minimum, OK

**Design Note:**

Keystone Standard units are always recommended in situations where railings are considered for direct mounting on the wall system.

Alternate railing designs that include extra geogrid levels purposely installed between the upper three courses to assist in resisting the overturning forces may be considered. These design alternatives require fully grouted cells with lateral reinforcement to provide a coherent mass for a 50 plf or greater loading. The reinforcement and top of wall detailing is more critical in these situations and the design more questionable.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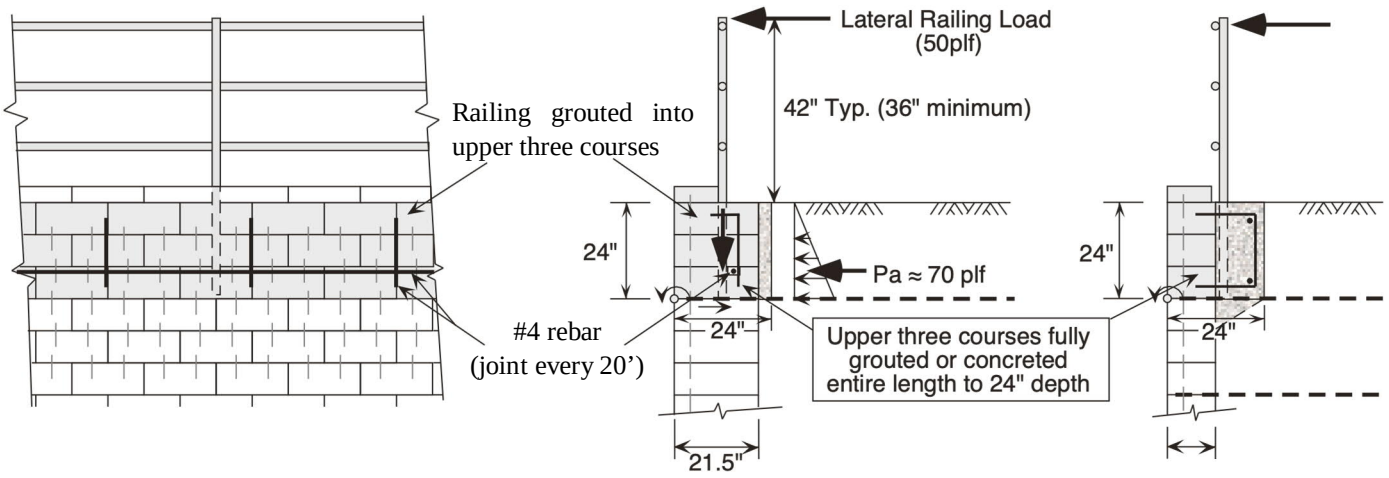
# Typical Railing Design

## Direct Mount - 50 plf




**Typical Railing Elevation** **Standard Unit Section** **Alternate 12" Depth Section**

### Introduction

It is difficult for a railing design to satisfy structural design requirements when considering the direct mounting on or into the Keystone modular wall system. The small unit size and mass provides minimal resistance to overturning by itself so additional mass must be engaged to provide the required resistance. Modular wall units are typically not large enough to satisfy an IBC lateral design loading without additional structure.

### Railing Analysis

Shear resistance of Standard units (>1000 plf) and 12" Depth units (>600 plf) exceeds the driving forces by a wide margin in gravity wall applications and is not a critical evaluation. Overturning at the top of wall (local stability) is the critical evaluation. A 50 plf or greater loading typically requires the addition of concrete and reinforcement for mass and strength. The higher 50 plf and 200lb load combination required by AASHTO for highway projects requires that the railing design and analysis be treated differently.

Driving Moments (50 plf load)

Railing 50 plf x 5.5' arm = 275 ft-lbs/ft
Soil 70 plf x 0.67' = 47 ft-lbs/ft
Total = 322 ft-lbs/ft

Resisting Moments (units filled with grout @ 140 pcf)

2' x 2' x 140 pcf x 1.0' = 560 ft-lbs/ft
0.33' x 0.88' x 120 pcf x 0.44' = 15 ft-lbs/ft
Total = 575 ft-lbs/ft

FSot = 575/322 = 1.78 < 1.50 minimum, OK

**Design Note:**

Keystone Standard units are always recommended in situations where railings are considered for direct mounting on the wall system.

Keystone 12" Depth units require additional reinforcement and concrete to provide the overturning mass necessary to resist design loadings. 12" Depth unit designs should consider offset railings as a simpler and more economical alternative.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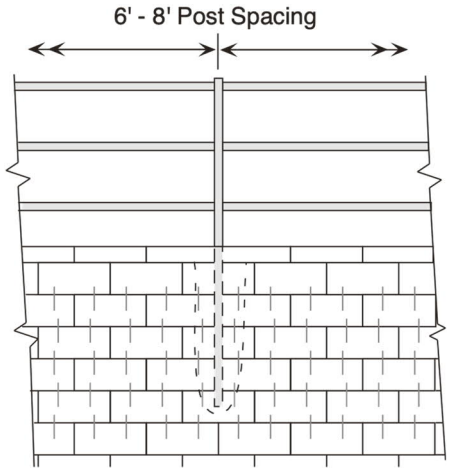
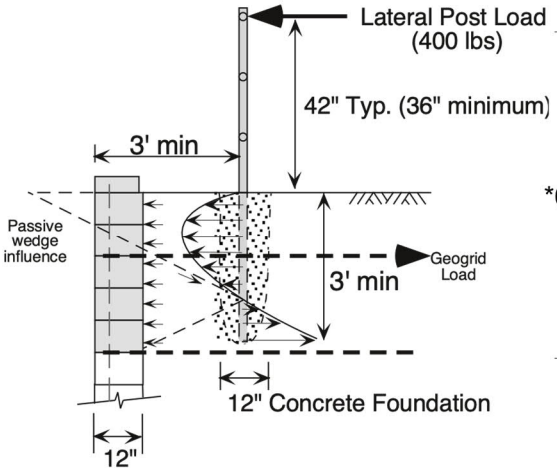
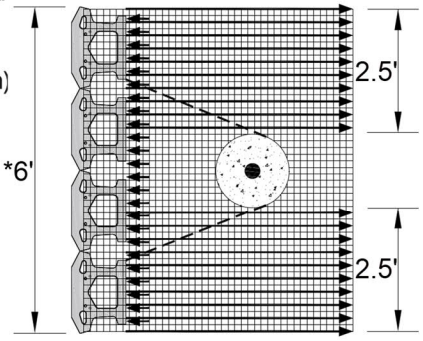
# Typical Railing Design

## Offset Installation - 50 plf - 12” Depth Unit




**Typical Railing Elevation**




*Due to the pins, lips, or lugs present with Keystone units, the influence of the post load is assumed to extend an additional unit.

**Typical Railing Sections**

### Introduction

The effect of offset railings on the wall design is a function of the depth and size of the foundation and distance back from the wall. Offset foundations have no significant effect on the wall structure if they fall outside the passive wedge that is created when soil is pushed. The benefit of soil reinforcement is obvious from the design section where the reinforcement picks up the lateral thrust from the foundation.

### Railing Analysis

Shear resistance of Keystone units is not a critical evaluation when soil reinforcement is utilized. The placement of a reinforcement level (geogrid with LTDS = 1200 plf) within the upper 2' of the wall permits the facing to absorb the distributed loading through tensile and pullout resistance.

Driving Thrust (200 lb load & soil)

Railing 400 lbs x 5.5'/1.0'± ratio = 2200 lbs
Soil 0.5 x 120 x (2.33)2 x 0.3 x 6' = 587 lbs
Total = 2787 lbs

Resisting Strength @ Post

Geogrid 5' x (LTDS @ 1200 plf) = 6000 lbs
Geogrid 5' x (Connection @ 960 plf) = 4800 lbs
Passive Soil Shear
2' x 1.5' x 1' x 120 pcf x Tan 30°= 207 lbs

FS = (4800+207)/2787 = 1.80 < 1.50 minimum, OK

Note: Only thrust is calculated above. Moments could also be calculated but the concept is to have the reinforcement placed high enough to absorb the thrust.

**Design Note:**

The design shown is the minimum requirement. Larger loadings will require greater offset, greater depth, and/or additional reinforcement to provide local design stability.

The same approach can be applied to fence and post foundation loadings where the influence of the upper foundation section is distributed to the wall system and sufficient resistance is provided.

AASHTO 2020 requires a 50 plf and 200 lb post load to be applied simultaneously. With a 6’ post spacing and the additional 200 lb post load included, the resulting factor of safety decreases to the minimum 1.50 requirement.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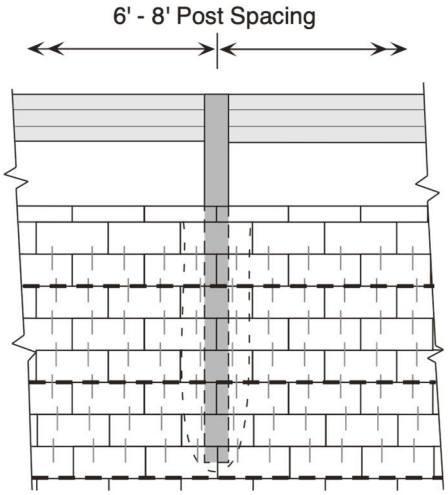
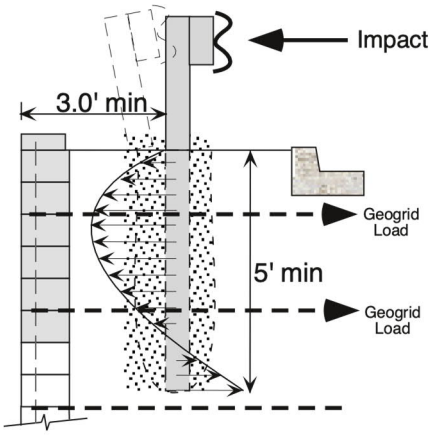
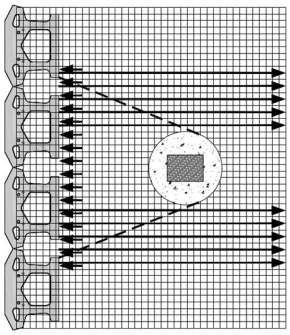
# Guiderail Detail



**Typical Guiderail Elevation**

**Typical Guiderail Section**

### Introduction

Guiderails or flexible post and beam barriers are not "designed" in the conventional sense as a guiderail is not expected to resist traffic impact loadings and remain serviceable like other barriers. The guiderail is designed to be sacrificed during impact and the energy absorbed through resistance and displacement that redirects or halts the vehicle after yielding and failure of the system. Typically, progressive resistance is developed as the first post fails and load is transferred to the adjacent posts through tension on the rail.

There have been numerous guiderail configurations developed over the years by regional transportation agencies and engineers in an attempt to balance cost and performance. They are all similar with varying degrees of stiffness and displacement characteristics exhibited upon impact. The design criteria is relatively simple and only requires that a flexible barrier system be provided sufficient lateral space to displace under impact loading. This space requirement is typically 3.0' minimum which can be a problem when not properly accounted for in project site design plans.

### Guiderail Analysis

The analysis of a Keystone wall structure with a guiderail placed as shown is problematic in that the guardrail is designed to fail under impact. Therefore, there may or may not be some localized displacement of the soil and upper wall units as a result of a major impact. The AASHTO load design criteria appears to be a reasonable consideration although the extent of displacement cannot be predicted.

### AASHTO 2015

Flexible post and beam barriers, when used, shall be placed a minimum distance of 3.0' from the wall face, driven 5.0' below grade and spaced to miss the reinforcements.... The upper two rows of reinforcement shall be designed for an additional horizontal load of 300 plf ... 50% to each layer of reinforcement.

### Design Note

Two levels of reinforcement are required in the upper 3' - 4' of wall to provide resistance against the loads suggested by AASHTO. Posts holes are either augered through the geogrid reinforcement or the posts placed in tubes previously installed during wall construction unless a drive point is used to cut through the reinforcement.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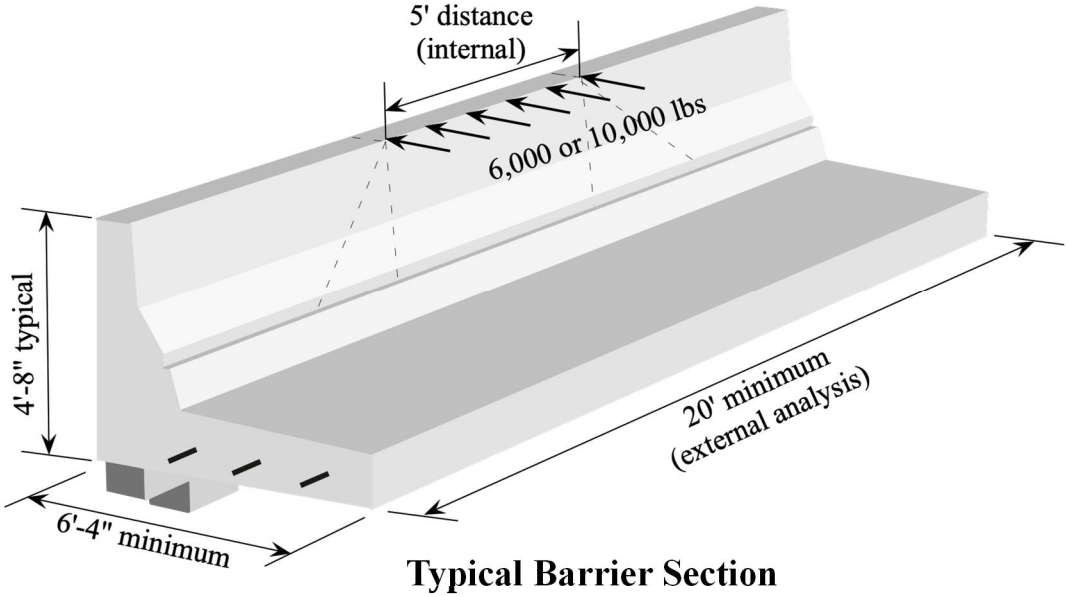
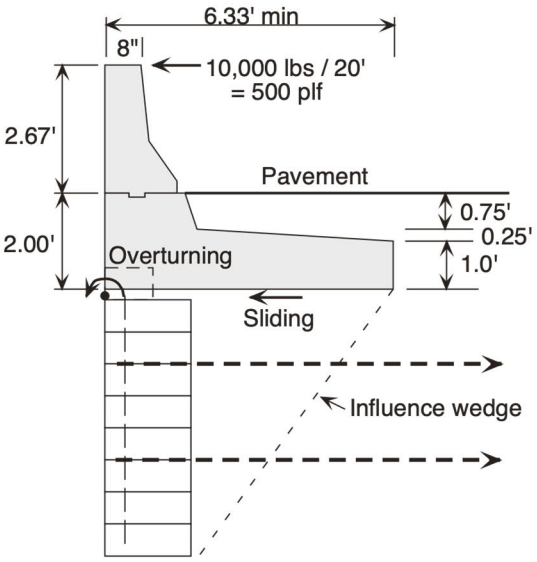
# Traffic Barrier




**Typical Barrier Section**




**Typical Design Section**

Barrier can impart additional load to the upper reinforcement levels. AASHTO testing of steel reinforcement systems requires that 2300 and 600 plf be added to the top two reinforcement levels regardless of slab configuration and type of soil reinforcement. This probably overstates the loads considerably for geosynthetic reinforcement.

"Jersey" crash barriers are typically designed as independent structures on top of MSE wall structures to avoid negative interaction between the flexible wall system and a rigid barrier system. A flexible pavement design section is shown with the "leg" under the pavement. Concrete pavement designs are similar but can incorporate the pavement or sidewalk as the "leg".

Typically, the momentary loading condition of a traffic impact to the barrier does not impart a significant loading to the MSE wall system due to the inertial mass of the large heavily reinforced section. However, AASHTO design criteria establishes a pseudo-static analysis of 10,000 lbs applied over a 5' width for rigid traffic barrier systems which becomes a 500 plf loading when transmitted through the junction slab to the wall system as shown (pour lengths are 20' minimum, 30' maximum, and the barrier section is separated from the wall system). Note that each state may have their own barrier design requirements based on TL ratings for roadways. Therefore any barrier design should reference each individual state code.

The analysis of the upper two reinforcement levels for this additional loading may consider reduced factors of safety or elimination of creep factors for geogrid materials since the loading condition is momentary and may never occur over the life of the structure. Pullout resistance may consider the entire length of reinforcement.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# Quality Assurance Provisions

## General Design Notes

1. The following effective strength design parameters were assumed in the preparation of structural calculations for the Keystone retaining wall system:

| | ϕ | c(psf) | γ(pcf) | Soil Type |
|---|---|---|---|---|
| Reinforced Soil | 30° | 0 | 120 | SM - Silty Sand |
| Retained Soil | 30° | 0 | 120 | SM - Silty Sand |
| Foundation Soil | 30° | 0 | 120 | SM - Silty Sand |

   Soil types and design properties shall be confirmed by the site geotechnical engineer prior to wall construction. Keystone accepts no responsibility for the interpretation or verification of subsurface conditions.

2. The system has been evaluated for internal stability and simple external sliding and overturning. Refer to design calculations for specific minimum factors of safety utilized for the wall design.
3. The walls are designed to support the following maximum surcharge loadings:

| | | |
|---|---|---|
| Live Load: | 250 psf | Wall 1 |
| Dead Load: | 50 psf | Wall 1 |
| Back slope: | Level | Wall 1 |
| | 3H:1V | Wall 2 |
| Seismic: | As = 0.20g | All Walls |

4. The wall foundation soils at each wall location shall be capable of safely supporting 3000 psf or as indicated on the wall elevations without failure or excessive settlement. Local bearing capacity shall be confirmed by the site engineer.
5. The Contractor shall provide surface and subsurface drainage, grading, and erosion control during and after wall construction to avoid damage to the wall structure
6. The Contractor is responsible for obtaining all permits and easements necessary for wall construction. The Contractor is responsible for protecting adjacent property from wall construction activities.

## Quality Assurance Provisions

1. Wall construction shall be monitored by a qualified Engineer to verify field conditions. If this work is not performed by the site geotechnical engineer, the geotechnical engineer shall be consulted in those matters pertaining to soil conditions and wall performance.
2. The foundation soils at each wall location shall be inspected by the Engineer and any unsuitable soils or improperly compacted embankment material removed and replaced as directed by the Engineer prior to wall construction to provide adequate bearing capacity and minimize settlement.
3. All wall excavation and retained soils shall be inspected for groundwater conditions and any additional drainage provisions required in the field shall be incorporated into the wall construction as directed by the Engineer.
4. Wall backfill material shall be tested and approved by the Engineer for use in the reinforced soil zone meeting the minimum requirements of the approved design plans.
5. All soil backfill shall be tested by the Engineer for moisture, density, and compaction periodically (every 2’ vertically, 100’-200’ c/c) and shall meet the minimum requirements of the approved design plans or project specifications.
6. Wall construction shall be periodically inspected by the Engineer to insure the geogrid reinforcement elevations and lengths are installed in accordance with the approved design plans.
7. All wall elevations, grades, and back slope conditions shall be verified by the Engineer in the field for conformance with the approved design plans. Any revisions to the structure geometry or design criteria shall require design modification prior to proceeding with construction.


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# Tall Walls - Alignment Monitoring &Adjustments

The “art” of constructing tall Keystone walls requires constant attention to vertical and horizontal alignment due to the accumulation of minor construction and manufacturing variations when thousands of wall units are placed and backfilled. Taller walls magnify any construction and manufacturing variations and require much closer attention to alignment monitoring to achieve the same results as smaller walls.

## The Problem

Wall units are initially set to an alignment determined by the contractor that accounts for wall batter. The precision of setting the first course is proportional to the amount of time spent on this function. Subsequent block courses are set and backfilled which begins to introduce error to the original wall alignment due to the minor variations caused by the setting, backfilling, and compaction process and slight dimensional variations of the units. If these small variations are not corrected as they begin to accumulate, a misalignment becomes increasingly noticeable to all involved and becomes a very difficult problem to correct at a later time without a significant dismantling of the wall structure.

## Horizontal Alignment

Horizontal misalignment is usually obvious when sighting down a wall. The cause is not always apparent since the initial courses may not have been set to the proper alignment and grade, subsequent courses may have not been set properly, the units may not fit perfectly straight and level, and/or the wall alignment may have been disturbed during backfilling and compaction.

The initial correction for most horizontal alignment problems is to identify the cause so that further construction procedures can then be modified to correct the misalignment. Realignment can be accomplished by adjusting wall setback using different alignment pin hole locations or redrilling new pin holes to achieve the desired alignment. Small adjustments to horizontal alignment can be made with little structural or aesthetic concern when alignment problems are noted early. The key is to check alignment often and make "small" corrections as needed.

## Vertical Alignment

Vertical misalignment is not as obvious as horizontal misalignment but contributes equally to the problems noted in taller walls. Every course of wall units should be checked for levelness with minor shimming done every course or two to insure that the units remain true to the design batter with a slight back. Walls can tend to lean either forward and backward or both (results in horizontal misalignment as wall height changes) depending upon the tendencies of the wall facing units to get out of level due to fabrication or construction considerations. This problem gets greater with height when not corrected and a wall will continue to "roll" forward or backward until it is obviously Shimming should be limited to approximately 1/8" per course and spread over a number of courses to avoid structural or aesthetic concerns.

Shimming materials should be somewhat flexible to compress slightly and distribute load evenly while avoiding hard materials which can create load concentration points. Geogrid or geotextile material can be folded to varying thicknesses and used for this purpose. Other common building materials such as roofing shingles and pieces of thick rubber or PVC liner material have been used for shimming with success.


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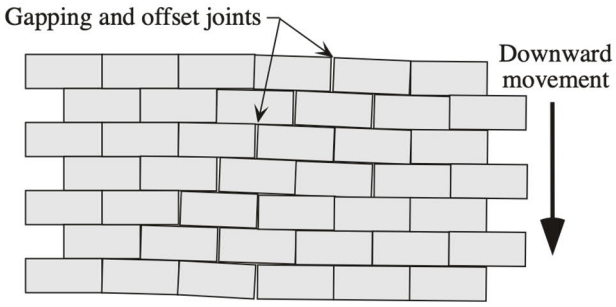
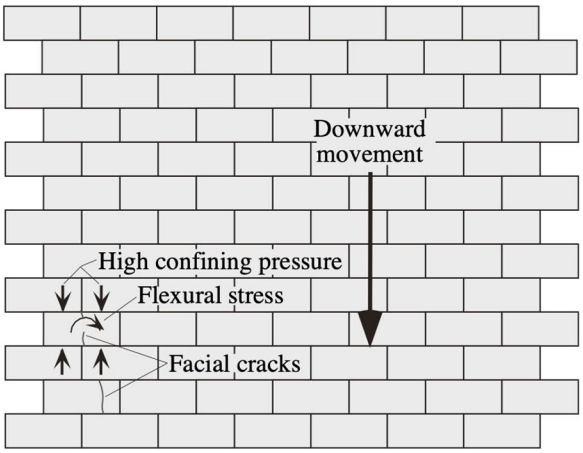
# Settlement - Unit Cracking/Gapping

Keystone modular retaining wall structures can tolerate a certain amount of settlement due to the flexible nature of the system and small individual unit size. Differential settlement limits of 1/100 (1% ) and 1/200 (1/2 %) have been suggested by NCMA and AASHTO respectively for modular block systems. These limits appears to be reasonable for most cases. When greater settlement is anticipated, ground improvement techniques are warranted and possibly the use of slip joints to increase the flexibility of the wall facing system and provide facial stress relief.

Observation of a number of completed structures that have undergone settlement indicates that the wall's tolerance for settlement without cracking is inversely proportional to the wall height. Lower height walls (H < 15') appear to have considerably more facial flexibility than taller walls (H > 15'). This increased flexibility is due to lower confining forces and load transfer taking place on each block which permits small individual movements to occur accommodating the settlement experienced without facial distress. Taller walls place the lower wall units under considerable confining pressure, restricting unit movement and permitting shear and flexural stresses to build up to the point where a block cracks as a means of stress relief.




**Low Wall Settlement**




**Tall Wall Settlement**

Low wall settlement problems are typically observed in residential projects where soils adjacent to houses are uncompacted and the walls settle differentially over a short distance. A wall that traverses an utility trench backfill zone is another typical settlement problem area. Usually gapping or offset joints are visually noted and the settlement is obvious.

Tall wall settlement is not as obvious but occasional facial cracks can be observed in areas of flexural stress concentration, typically in small groupings in the bottom 1/3 of a tall wall. Settlement induced cracks are usually not structurally significant and just a means of facial stress relief for the unreinforced dry-stack concrete facing system.

Cracked units can also be a symptom of a more serious problems so a review by a qualified engineer is always recommended.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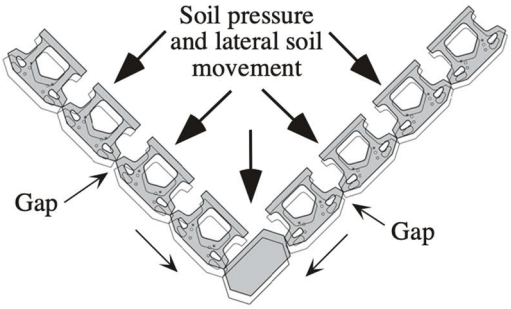
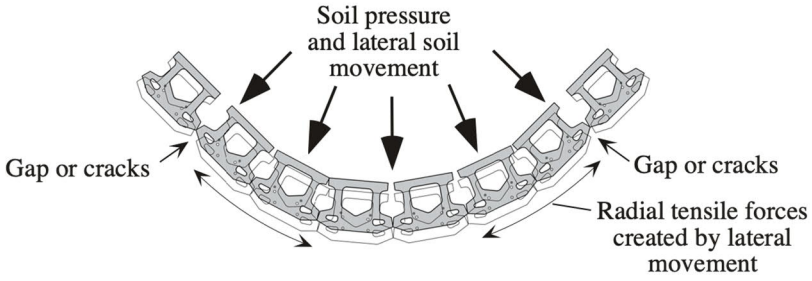
# Corners/Bends—Unit Cracking/Gapping

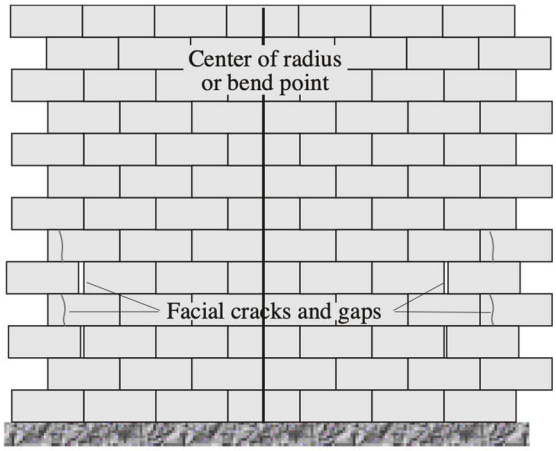
Keystone modular retaining wall structures can tolerate a certain amount of movement due to the small individual unit size and the flexible nature of the system. When corners and tight curves are inserted into an otherwise two dimensional system, a third direction of movement can occur which can cause unit cracking or gapping.



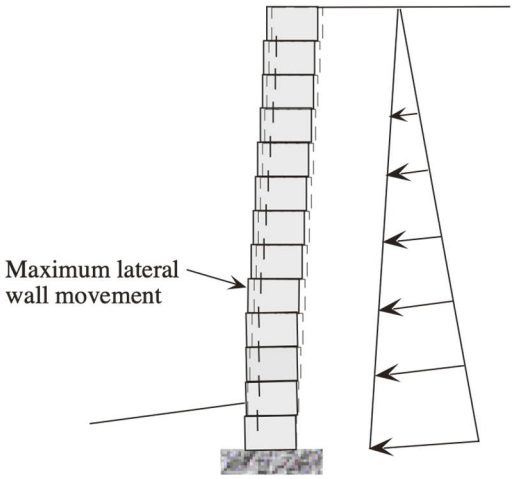

Wall Corner




Wall Radius




**Wall Elevation**




**Wall Section**

Gapping and cracking noted in these situations usually occurs in taller walls with lessor quality backfill and/or poor compaction. The wall backfill strains and deforms laterally under increasing earth fill load resulting in outward facing movement in the bottom third of the wall height. This is typically not a noticeable problem in straight walls but at corner or bends the movement is magnified and can create the gapping and cracking noted due to the buildup of radial tensile forces along the wall face.

Solution: Use high quality granular fill in tight radius or bend areas in taller walls. Backfill entire zone with 3/4" stone to minimize lateral wall movement.

**Wall Translation Potential**
**Active Earth Pressure Theory**

| Soil Type and Condition | % of Height |
|---|---|
| Cohesionless, dense | 0.1 to 0.2 |
| Cohesionless, loose | 0.2 to 0.4 |
| Cohesive, firm | 1.0 to 2.0 |
| Cohesive, soft | 2.0 to 5.0 |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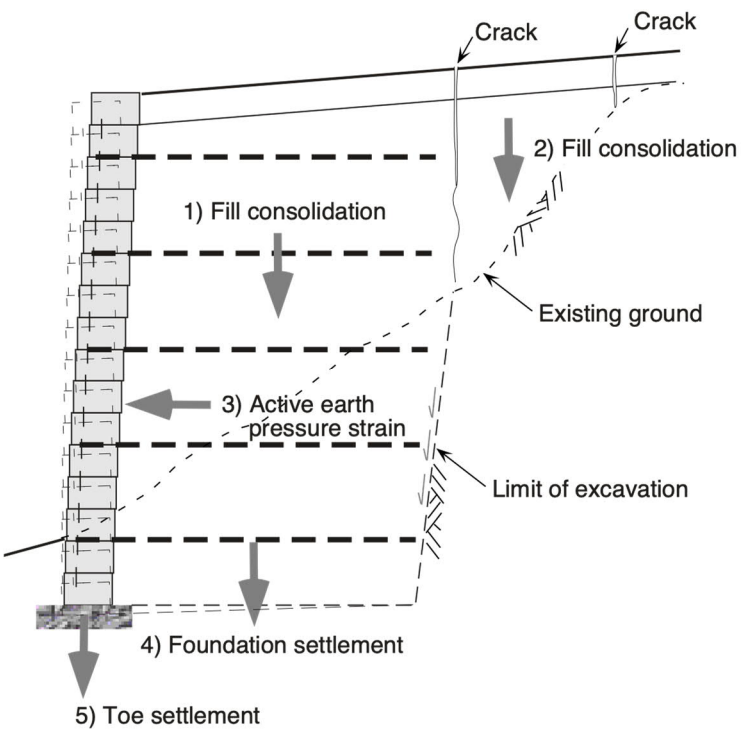
# Backfill Movement and Soil Cracks

Keystone retaining walls are flexible reinforced soil masses which interact with the foundation and retained backfill zones to provide a stable retaining structure. These soil zones have different stress/strain/ consolidation properties which can result in differential movement and strain of the reinforced and retained soil matrix.

Relative movement of the soil masses is typically noted in taller wall structures when small soil cracks occur behind the wall structure near the boundary of soil zones with different strain properties. Experience has shown that this type of soil cracking is most noticeable after very heavy rainstorms where the additional saturated soil weight and seepage pressures involved can cause slight differential movement of the masses. A wall schematic and possible causes are shown below:




**Typical Section**

**Possible Causes of Soil Cracking**

1) Consolidation of reinforced zone backfill - Any settlement of reinforced fill relative to adjacent soils may cause cracking at end of reinforcement. If soils are placed and compacted in dry condition, water can cause secondary consolidation of the reinforced fill and cracking at the end of reinforced zone.

2) Consolidation of retained soil wedge - Similar to Item 1 causing cracking at back of fill wedge relative to existing soils.

3) Lateral wall movement due to active earth pressure state - Lessor quality backfill soils exhibit higher lateral movement to mobilize the active earth pressure state. If the reinforced wall mass strains laterally, the fill must settle accordingly and cracking can occur.

4) Foundation settlement - The foundation soils of many wall structures have not experienced the loading from the new fill which can cause differential settlement between the wall volume and cut slope soil.

5) Toe Settlement - The wall toe may experience more settlement than the wall heel due to lack of overburden or confining pressure resulting in slight lateral wall movement in the upper wall section and tension cracking at the end of reinforcement.

Soil cracks can also be a sign of global instability or continuing settlement which requires an evaluation by a geotechnical engineer. However, most minor soil cracking observed is structurally insignificant to the long term performance of the wall structure but can lead to reflective cracks in pavement sections and/or separation of curbs when of greater magnitude.

Significantly increasing the length of the upper reinforcement levels to help bridge the potential crack zones can be a prudent precaution for projects with flexible pavements extending over all zones. High quality backfill, proper backfill placement and compaction, and firm foundations are the best precautions against soil cracking.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# Masonry Concrete Durability Freeze-Thaw

Concrete may experience a reduction in useful life due to the effects of weathering which includes the effect of freeze-thaw cycles. Freeze-thaw damage to concrete requires a sufficient presence of water in the void spaces of the concrete (critical saturation) to permit high internal pressures to develop from the water freezing and damaging the concrete matrix over time.

Some concrete masonry retaining units have exhibited premature deterioration under saturated freeze-thaw conditions in specific locations which has prompted inclusion of special durability testing requirements in Owner's specifications when required. ASTM published test method C1262 in 1997 which specifically addresses the freeze-thaw testing of concrete masonry units compared to the similar C666 test method for poured concrete. Both tests expose concrete to multiple freeze-thaw cycles in the presence of water.

Current reference specifications are summarized below:

**ASTM C1372 - Standard Specification for Segmental Retaining Wall Units**

7.3 When required, sample and test five specimens for freeze-thaw durability in water in accordance with Test Method C1262.

4.2.1 Specimens shall comply with either of the following:

1) the weight loss of each of the five test specimens at the conclusion of 100 cycles shall not exceed 1% of its initial weight: or,

2) the weight loss of each of four of the five test specimens at the conclusion of 150 cycles shall not exceed 1.5% of its initial weight.

**AASHTO - Earth Retaining Systems - Segmental Concrete Facing Blocks**

In areas of repeated freeze-thaw cycles, the facing blocks shall be tested in accordance with ASTM C1262 to demonstrate durability. The facing blocks shall meet the requirements of ASTM C1372, except that acceptance regarding durability under this testing method shall be achieved if the weight loss of each of 4 out of 5 specimens at the conclusion of 150 cycles does not exceed 1% of its initial weight.....Facing blocks directly exposed to spray from deiced pavements shall be sealed after erection with a water resistant coating or be manufactured with a coating or additive to increase freeze-thaw resistance.

Freeze-thaw durability damage requires saturated conditions which is typically only observed along the top of a wall where a continuous snow melt can supply water to the concrete and the saturated freeze-thaw cycle can be repeated numerous times. Saturated concrete can also exist in concrete along waterways or facing roadways where road salt laden water/snow is continuously sprayed against the wall face during the winter.

Some Owners have incorporated salt (saline) into the freeze-thaw testing to create a more aggressive environment for certain roadway applications and can also accelerate the testing by requiring less cycles with saline vs water. However, there is little correlation between accelerated saline testing and in-service performance at the present time and the consistency of the test results between samples and labs leaves much to be desired with saline testing.


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# Vertical Reinforcement Spacing

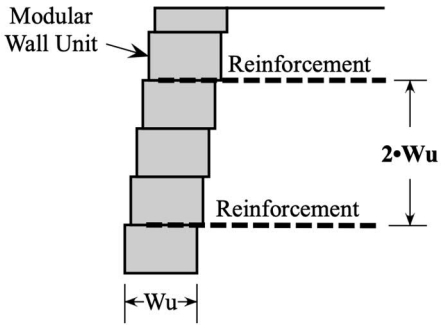
MSE walls are designed and constructed with continuous sheet or discrete soil reinforcement. Geogrid soil reinforcement is most commonly used with segmental wall systems and is placed continuously at each design level (100% horizontal coverage). The vertical spacing of the geogrid reinforcement levels is limited by the size of retaining wall units and facial stability calculations. The vertical spacing can also be limited by specific project design considerations such as calculated tension, connection strength, and pullout factors for static and seismic loadings.

Most published design specifications and common design practice requires that the vertical reinforcement spacing be limited to twice the depth of the unit, Wu, not to exceed 32". This limitation is based on a typical facing stability analysis during construction and extensive industry experience. For the standard 12" deep structural retaining wall unit, a 24" vertical spacing can be maintained and meet all requirements. All major design guidelines address these requirements as noted below:

**NCMA Design Manual for Segmental Retaining Walls, 3rd Edition**
Section 7.2.2: Geogrid Vertical Spacing Requirements

*Vertical spacing of geogrid layers should also be limited to a maximum spacing, regardless of the results of the stability calculations. It is often cost efficient to maximize vertical spacing between geogrid layers as allowed by the stability factors of safety. Even when all internal and facial stability failure modes can be satisfies with greater spacing, However, a maximum vertical spacing between reinforcement layers of 24 in. (610 mm) is suggested to reduce construction stability issues. Some proprietary systems may be capable of supported larger spacing between reinforcement layers without construction issues, however, the reinforcement should not exceed 24" (813 mm). This maximum spacing limits construction issues and also ensures a reinforced soil mass behaves as composite material, as intended by this design methodology. Within these limits, the wall designer should choose an appropriate maximum reinforcement spacing for the proprietary system used.*

*For modular blocks that are less than or equal to 10 in. (254 mm) in depth, it is recommended that the maximum vertical spacing of the reinforcement layers be no more than twice the depth of the unit. For example, the maximum vertical spacing for a 9 in. (229 mm) deep modular block unit would be 18 in. (457 mm). Regardless of the spacing of the reinforcement, compaction of the reinforced/retained soil zone must never exceed 8 in. (203 mm) in thickness.*




**Maximum Spacing Section**

**Examples:**

For a 8" tall x 9" deep unit, the vertical spacing is limited to 2 x 9" = 18" which would be two block courses or 16" vertical spacing.

For a 6" tall x 10" deep unit, the vertical spacing is limited to 2 x 10" = 20" which would be three block courses or 18" vertical spacing.

**AASHTO LRFD Bridge Design Specifications, 2016, Section 11.10.2.3.1**
*...For segmental concrete facing blocks, facing stability calculations shall include an evaluation of the maximum vertical spacing between reinforcement layers, the maximum allowable facing height above the uppermost reinforcement layer, inter-unit shear capacity, and resistance of the facing to bulging. The maximum spacing between reinforcement layers shall be limited to twice the width, Wu, illustrated in Figure 11.1.6.4.4.b-1, of the segmental concrete facing block or 2.7 ft, whichever is less.....*

**FHWA NHI-10-024, Section 4.4.7.d**
*For walls constructed with modular blocks, the maximum vertical spacing of reinforcement should be limited to two times the block depth (front fact to back face) or 32 in. (810 mm), whichever is less, to insure construction and long-term stability...*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# Keystone Retaining Wall Precautions and Maintenance

The Keystone Segmental Retaining Wall (SRW) is constructed of concrete masonry units, geogrid soil reinforcement fabric, and compacted structural backfill. The structure's performance is sensitive to any post construction activities that may damage components, increase loading conditions, and/or reduce overall stability. The following list is intended to provide guidelines for the proper care of a Keystone wall.

1. The area behind the wall that contains geogrid soil reinforcement fabric (reinforced zone) is the primary structural component of the wall system. Do not, under any circumstances, excavate through, drill through, or otherwise damage this reinforcement fabric without written approval of the design engineer of record.
2. The drain line at the base of the wall (if required and installed) was stubbed out to daylight or for final connection to storm drainage systems by others. Please insure that all connections are made to proper drain outlets and that any drains outlets to daylight are not buried.
3. The Keystone SRW wall is normally constructed over a crushed stone base at a nominal depth. No digging or excavation shall be done within 3 feet horizontally from bottom face of wall or to such depth that would compromise the integrity of the wall foundation.
4. All water must be diverted away from the base of wall to avoid erosion and undermining of the wall foundation after installation. This includes temporary site grading during construction and final site grading.
5. Landscape watering and surface drainage above the wall should be designed in consultation with the Civil and Geotechnical engineer and performed in such away to avoid standing water, water cascading over the wall, and infiltration (saturation) of the reinforced soil zone.
6. Do not increase the height of an existing wall as originally designed and constructed with more block units without the written approval of the design engineer of record.
7. Do not add a slope or increase the steepness of a back slope beyond what was considered in the original grading plan and wall design without written approval of the design engineer of record.
8. Do not add additional surcharges within a lateral distance of twice the overall height of the structure(s) without written approval of the design engineer of record or unless considered in the original wall design. This would include sound walls, landscaping walls, swimming pools, buildings, garages, etc.
9. Do not operate heavy equipment, within four feet of top of wall face. The surcharge from equipment weight can push the upper wall units out resulting in unacceptable misalignment after construction.
10. Segmental retaining walls are flexible structures (vs. rigid as in CMU or concrete walls) and are subject to some post construction settlement and movement. All structures (i.e. sidewalks, pavements, curbs, trash enclosures, utility lines, etc.) should be designed to handle some ground movement and not be connected directly to the wall facing units or other structures.
11. The retaining walls should be periodically inspected to verify that drainage measures are functioning properly, erosion has not occurred along the top, ends, or bottom and any unanticipated movement or deflection of the wall system has been evaluated by a qualified engineer.
12. The wall facing should be inspected annually for growth of weeds, bushes, or trees than may occur in the facing system. Any excessive growth should be removed and treated to stop continued growth and possible damage to facing over time.


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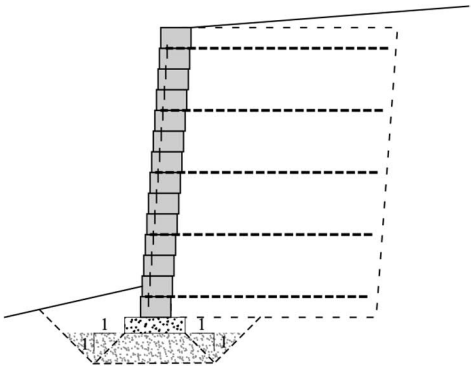
# Foundation Improvements

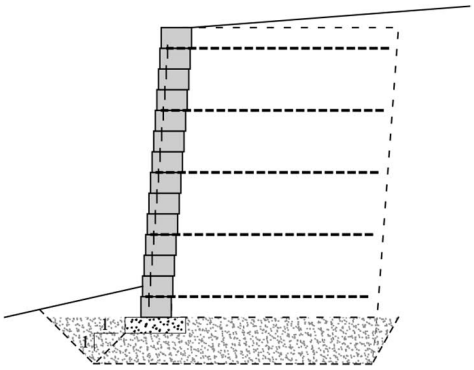
Keystone segmental walls are somewhat flexible and can tolerate movement better than rigid structures thus soil bearing capacity is of lessor concern. However, poor foundation soils can cause excessive wall movement and potential loss of bearing capacity and must be corrected prior to wall construction.

The following diagrams depict typical foundation improvement schemes in increasing complexity. Other methods such as surcharging foundation soils and/or utilizing lightweight backfill and geofoam may help reduce pressure and improve foundations. The geotechnical engineer should be consulted for foundation improvement recommendations any time there is a concern.



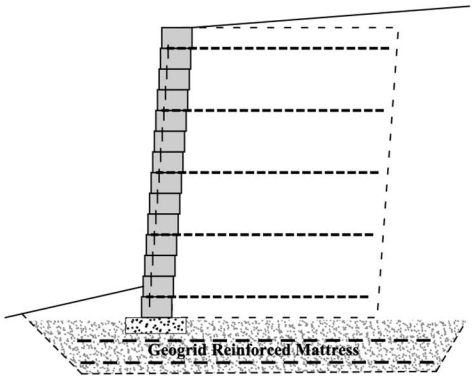
## Local Bearing Capacity

Remove and replace foundation material as directed by engineer to properly support facing column against local settlement or punching failure similar to a building strip footing



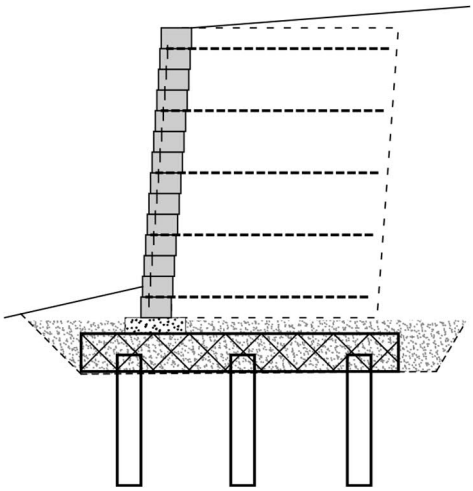
## Gross Bearing Capacity

Remove and replace foundation material under reinforced zone as directed by engineer to properly support entire reinforced mass and improve overall bearing capacity.




## Gross Bearing or Sloping Toe Instability

Install reinforced base under wall to increase stiffness of foundation improvement relative to underlying soils and improve global stability.



## Poor Foundation Bearing Capacity

Install stone columns or geopiers to increase bearing capacity for foundation soils. Install piling and slab to support reinforced wall structure when poor soils or water table does not permit conventional improvements.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