Technical Information Sheets

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

12/30/03

Technical Information Sheets

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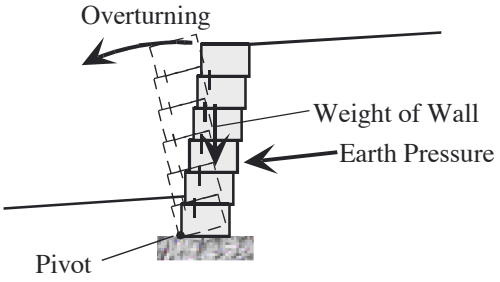
Gravity Walls

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.

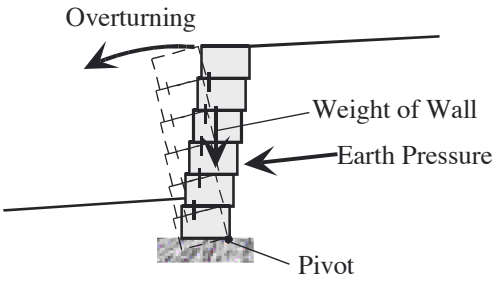
Modular gravity walls rely on weight, depth, wall batter, and inter-unit shear strength to achieve stability. Larger units with more depth provide greater stability and can achieve greater wall heights as indicated in the gravity wall design charts in the Keystone Construction Manual.

The principal mode of failure observed in a modular gravity wall is overturning. 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 for site specific design recommendations. This "rule of thumb" leads to the following rough guidelines:

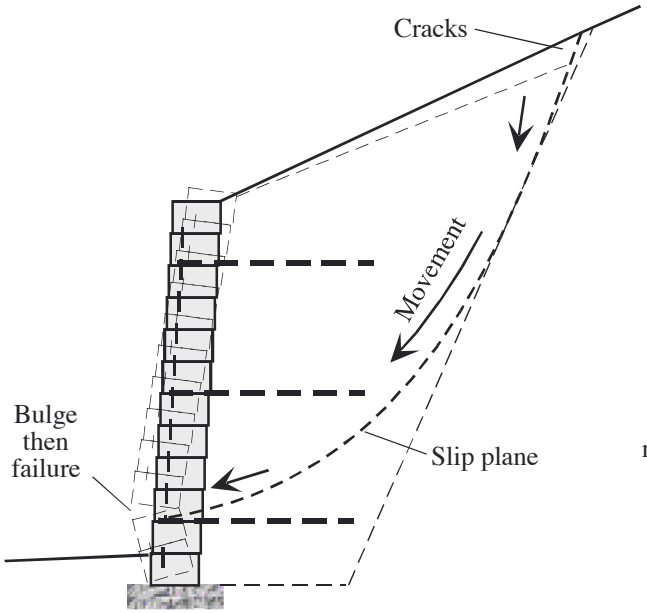
Unit	Depth	Max Height	Approx. Courses
Garden Wall	9"	27"	7 courses
Intermediate	12"	36"	5 courses
Compac Unit	12"	36"	4 courses + cap
Standard Unit	21"	63"	7 courses + cap

Reinforced Soil Walls

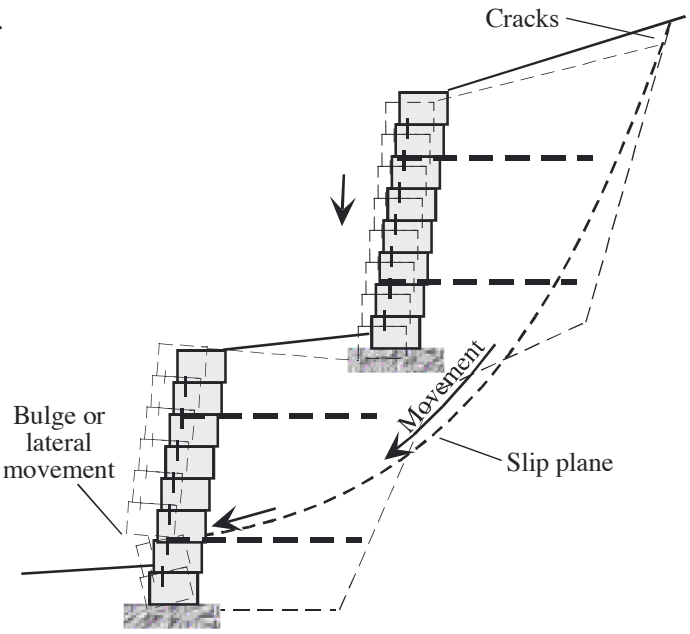
Reinforced soil walls are composite structures which utilize structural Keystone units and geosynthetic soil reinforcement to create a stable mass that can be designed and constructed to much greater heights than simple gravity walls. These structures are commonly referred to as MSE (mechanically stabilized earth) structures or reinforced soil SRW's (segmental retaining walls). Note that only the Keystone "pinned" 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 soil reinforcement length and 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 soil structures.



Internal/External Failure Mechanism



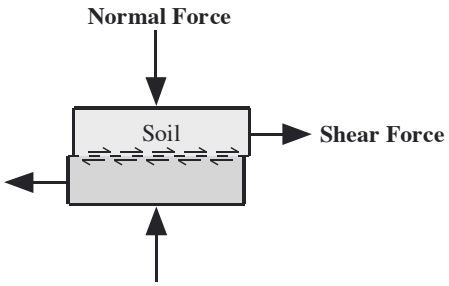
Tiered Wall Failure Mechanism

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.

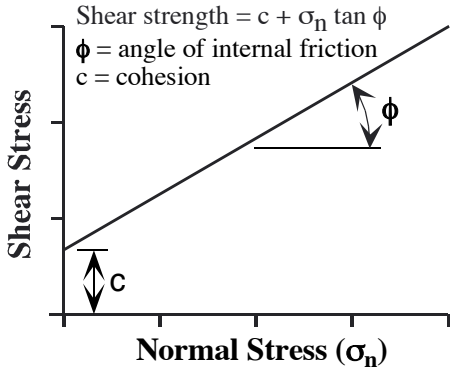
Soil Strength

Soil strength is a complicated geotechnical concept to simplify due to the inherent complexities of different soil types. Frictional strength, cohesive strength, and porewater 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 (ϕ and γ only) is utilized to determine driving and resisting forces. The figures below show the basic soil strength relationship.



Soil Shear Test



Plot of Shear Stress

The values for ϕ and c can be determined by direct shear test for granular soils and by triaxial testing for cohesive soils. Unit weight (γ) 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 design ranges for typical soil types.

Approximate Soil Design Parameter Ranges

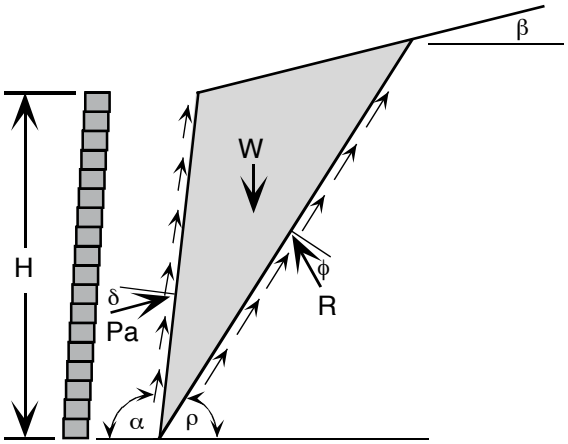
Wall Backfill Classification	Common Description	UNSC Classification	ϕ range	γ range (moist)	Comments
Good	Sand, Gravel, Stone	GW, GP, GM GC, SW, SP	32° - 36°	100 - 135 pcf	Poor grading lowers weight (ie: #57 stone)
Moderate	Silty Sands Clayey Sands	SM, SC	28° - 32°	110 - 130 pcf	Moisture Sensitive
Difficult	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

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.

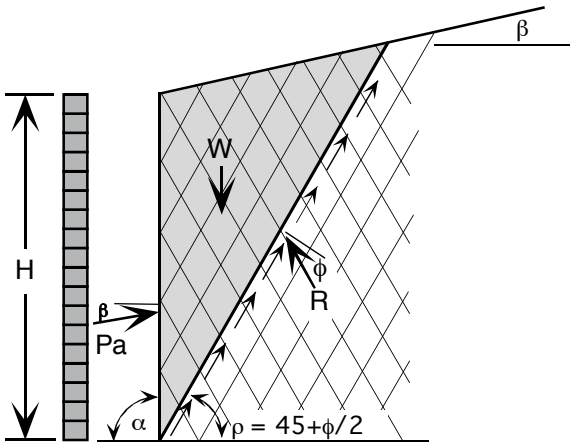
The active earth pressure calculation below requires that the wall structure rotates or yields sufficiently to engage the entire shear strength of the soils involved to create the active earth pressure state. The amount of movement required is highly dependent upon the soils involved.



$$\textbf{Pah} = 1/2\ \gamma\ H^2\ \textbf{Ka}\ \mathrm{Cos}(\delta-(90-\alpha))$$

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

Coulomb Wedge Analysis



$$\textbf{Pah} = 1/2\ \gamma\ H^2\ \textbf{Ka}\ \mathrm{Cos}\ (\beta)$$

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

Rankine "state of stress" Analysis

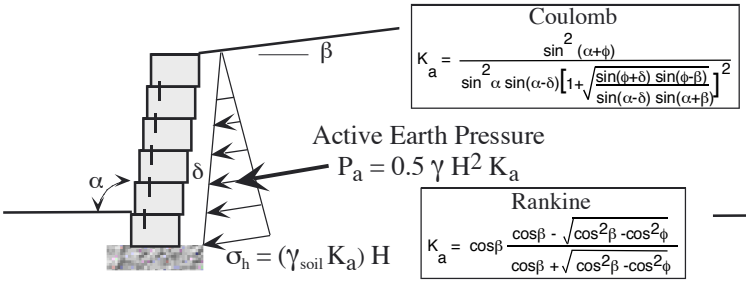
Using identical parameters, Coulomb wedge theory calculates less earth pressure than Rankine theory for a level backslope whereas the values converge under backslope conditions when $\delta = \beta$. Coulomb theory calculates a unique failure angle for every design condition whereas application of Rankine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

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 - Second Edition.*

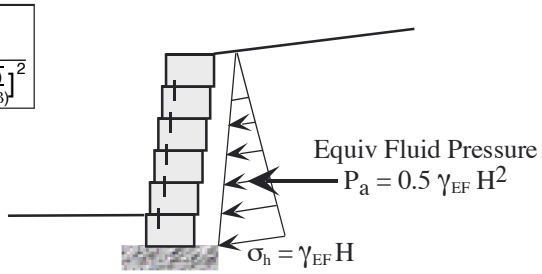
The application of Rankine "state of stress" earth pressure theory and fixed failure plane is favored by the transportation agencies (AASHTO and FHWA) and is described in recent editions of the AASHTO *Standard Specifications for Highway Bridges.*

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



γ_{EF} is assumed fluid pressure weight

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 minimum design loading regardless of structure geometry and assumed soil properties. For example, a heavily battered retaining wall design (10°+) utilizing high assumed soil strengths (ϕ = 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.

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

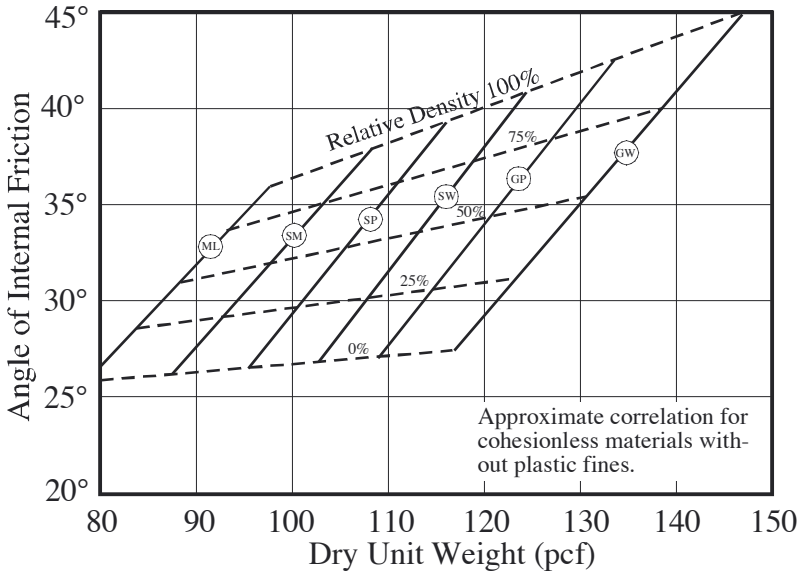
		Rankine-Vertical				Coulomb-1:8 Batter			
Backslope >		Level	4:1	3:1	2:1	Level	4:1	3:1	2:1
ϕ angle	34°	34	37	39	49	25	29	32	39
	30°	40	44	48	65	30	36	40	52
	26°	47	53	59	90	36	45	50	90

Note: Equivalent fluid pressure based on soil weighing 120 pcf. Low ϕ angles and steep backslopes create high pressures and do not permit equation solution to earth pressure.

Compaction 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 performance.



Correlations of Strength Characteristics for Granular Soils
(Ref. from NAVFAC DM 7.01-1986)

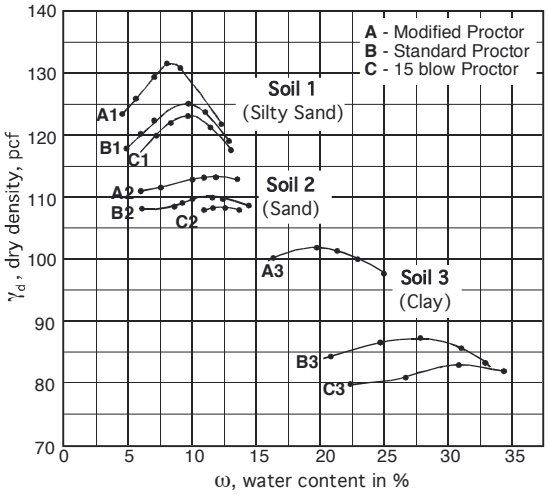
Granular soils are much more tolerant to variations in the placement and compaction process than the finer silts and clays which require close monitoring of moisture content and compaction procedures. The standard soil density specification for the structural fill behind Keystone walls is indicated below. Moisture content is limited to optimum moisture to avoid wall misalignment during construction due to overly saturated soils being compacted behind the wall facing.

Test Criteria	Minimum %	Moisture %	ASTM Method	AASHTO Method
Standard Proctor	95%	+0, -3	D 698	T-99
Modified Proctor	90-92%	+0, -3	D 1557	T-180

Soil Density - Standard vs Modified Proctor

Reinforced soil structures routinely specify that all soils be compacted to 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 specification problems.

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 Data

Type	Max Dry Density,pcf			Optimum 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 Modified Proctor Comparison

Type	Standard	95% Standard	% of Modified
Soil 1 - Silty Sand	125	119	90%
Soil 2 - Sand	110	105	93%
Soil 3 - Clay	88	84	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 in lieu of *Standard Proctor* testing for quality assurance testing of reinforced soil structures:

* 90% - 92% of *Modified Proctor* density is roughly equivalent to the specified 95% *Standard Proctor* density except for fine grained soils (ie: clay) where the difference may be significantly larger.

* *Modified Proctor* testing typically requires a lower optimum moisture content for achieving maximum density which is desirable for Keystone retaining wall construction and performance especially with silts and silty soils.

* 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 90% of Modified Proctor density and optimum moisture content when working with fine grained soils such as clays for best results.

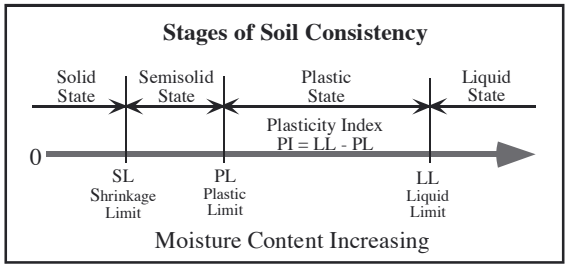
Silt/Clay Soils - Atterberg Limits

Reinforced soil structures perform best when constructed with granular backfill material. Predictable shear strength, low strain and consolidation characteristics, non-plastic behaviour, 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.

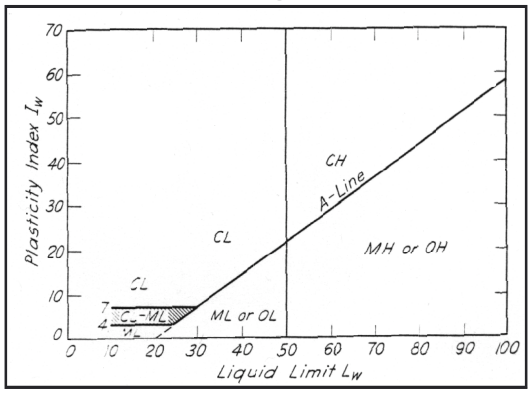
Atterberg limits are a set of index tests performed on fine grained silt/clay soils to determine the relative activity of the soils and their relationship to moisture content. The liquid limit, plastic limit, and shrinkage limits define the relative stages of behavior as indicated below when the soil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

The 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.

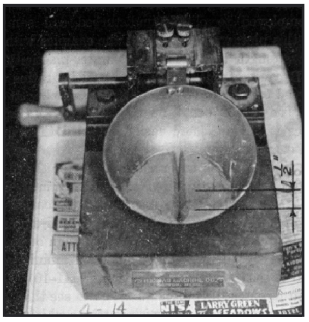
Construction of reinforced soil structures with plastic soils must always proceed carefully due to the potential for wall construction and performance problems and possible creep of the soil. Atterberg limit testing is mandatory for all clay soils and placement and compaction must be carefully monitored.



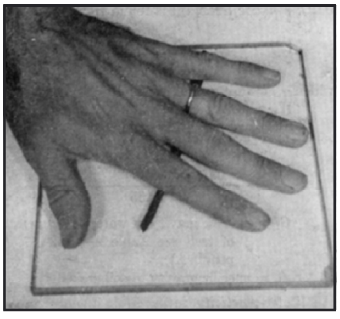
Atterberg Limits

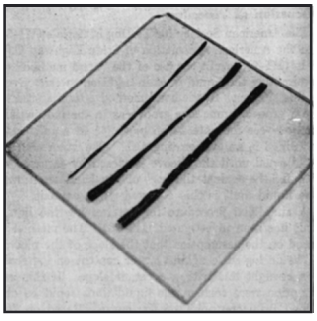


Modified Plasticity Chart



Liquid Limit Test





Plastic Limit Test

Backfill Soil Specification

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 use 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 providea 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%	≤ 20	≤ 40
Unacceptable Silt/Clay	2"	-	≥ 65%	> 20	> 50

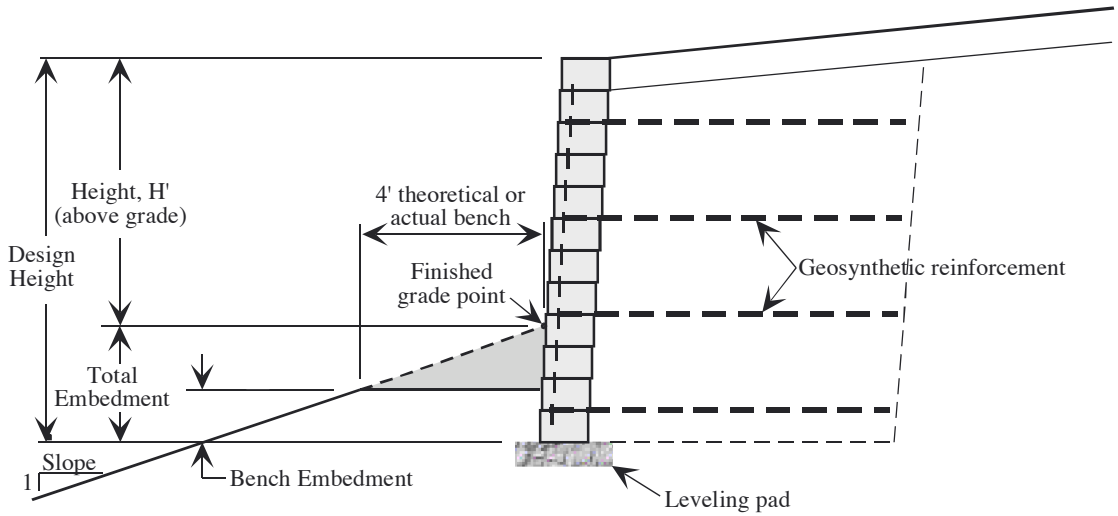
Note: It is easy to consider poor site soils for economic reasons but is not so easy to construct with such soils nor to expect high performance from marginal soils even though a design can be done on paper. The Owner should be advised and make informed choices regarding these issues.

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 theoretical finished grade point is established where the ground in front of the wall intersects the wall face alignment. It is best 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.



Typical Wall Embedment Section

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, or steep toe slopes. Special consideration should always be given to man-made fill slopes which can exhibit poor structural performance.

Recommended Wall Embedment

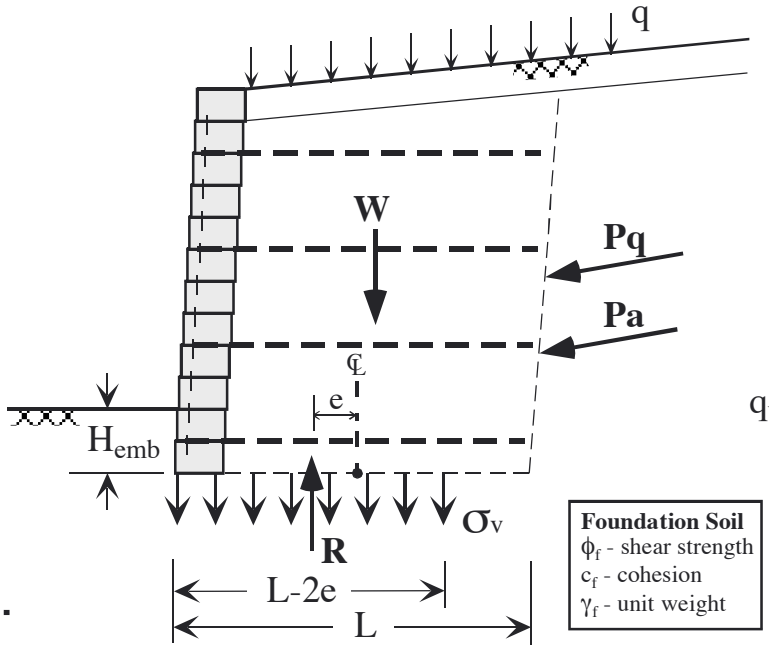
Toeslope Condition	Bench Embedment	Total Embedment
Level	10% H'	10% H'
4H:1V	10% H'	1' +10% H'
3H:1V	10% H'	1.33' + 10% H'
2H:1V	10% H'	2' + 10% H'

(Note: 10% of exposed wall height is good rule of thumb, however, it is possible to reduce embedment to 5% under certain conditions for taller walls where foundation elevations and conditions are clearly established)

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
ϕ_f - shear strength
c_f - cohesion
γ_f - unit weight

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

Bearing Pressure

Vertical Forces, $R = \Sigma W + \Sigma Fv$

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

Bearing Capacity

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

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

Factor of Safety

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

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.

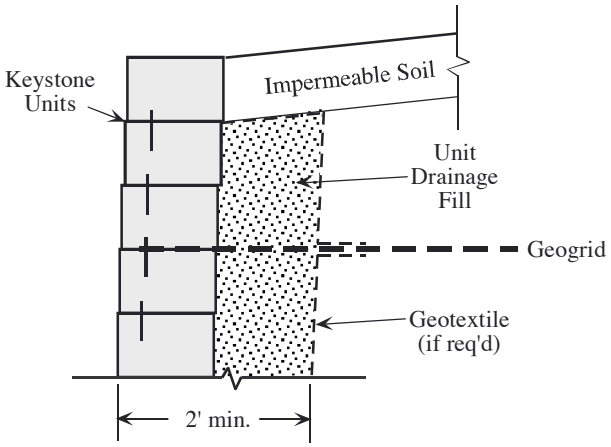
Bearing Capacity Factors (thru Vesic)

ϕ_f	N_c	N_q	N_γ	ϕ_f	N_c	N_q	N_γ
20°	14.83	6.40	5.39	28°	25.80	14.72	16.72
22°	16.88	7.82	7.13	30°	30.14	18.40	22.40
24°	19.32	9.60	9.44	32°	35.49	23.18	30.22
26°	22.25	11.85	12.54	34°	42.16	29.44	41.06

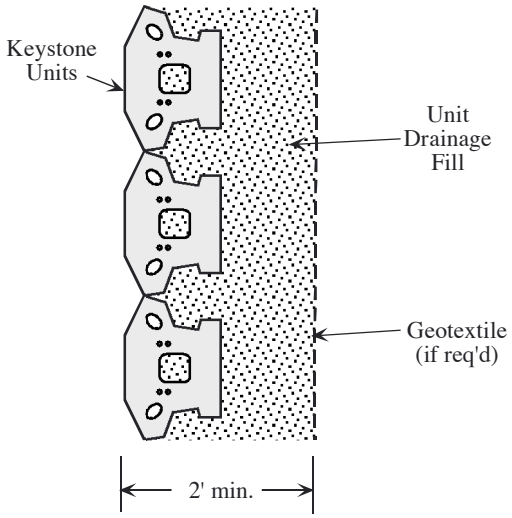
Vesic/Meyerhof Equations : $N_q = e^{\pi \tan\phi} \tan^2(45+\phi/2)$, $N_c = (N_q - 1)\cot(\phi)$, $N_\gamma = 2(N_q + 1)\tan(\phi)$

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 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 in areas where groundwater 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 significantly sized drainage zone.
* Provides a non-frost susceptible zone within and directly behind the wall units minimizing localized effects of freeze-thaw in moist soils behind wall.
* Provides an easily compacted material behind the wall units where compaction effort is difficult without displacing the wall facing units.
* Improves inter-unit shear and geogrid connection strength for units with cores and tapered sides.

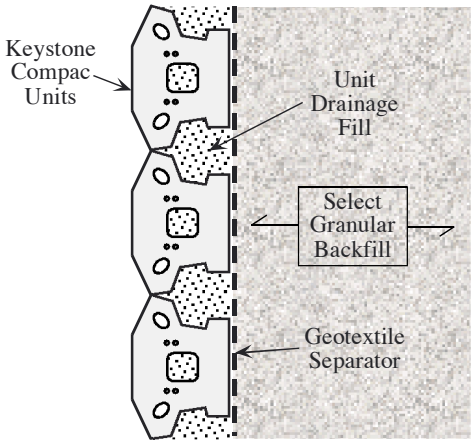
Unit fill drainage material is typically described as a 1/2" - 3/4" clean stone (1" minus or No. 57 stone are common). While many granular materials can be described as "free draining", the following gradation is recommended by Keystone based on experience:

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

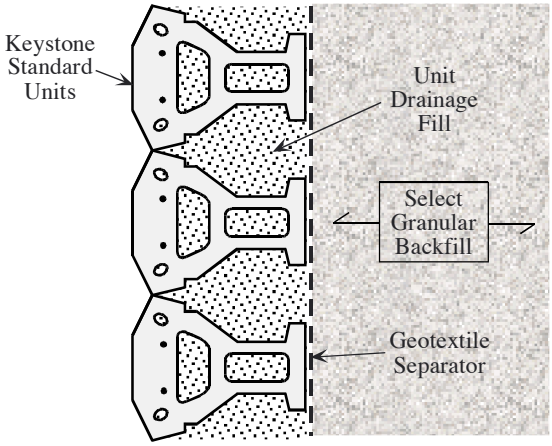
The intent of this specification is to limit the top size to 1" and restrict the sand and silt component to less than 10% to avoid migration of fines and for ease of placement and compaction.

Unit Drainage Fill Options

Placement of unit drainage fill in conjunction with different unit sizes and different backfill drainage and filtration requirements can result in some special combinations that have been utilized successfully in the past. There are some construction alignment considerations with different approaches that must be evaluated by the contractor. Acceptable variations are indicated below:

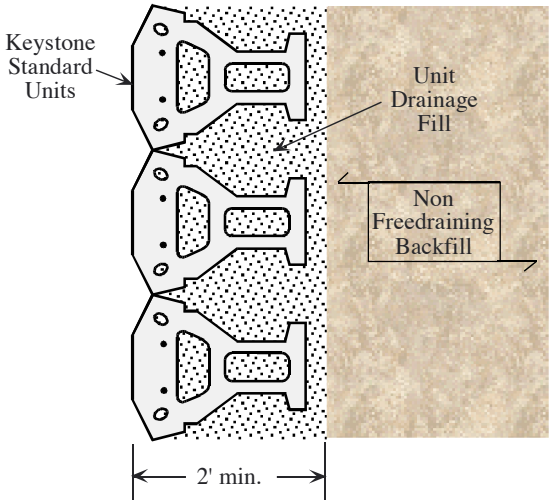


Unit Drainage Fill/Select Backfill

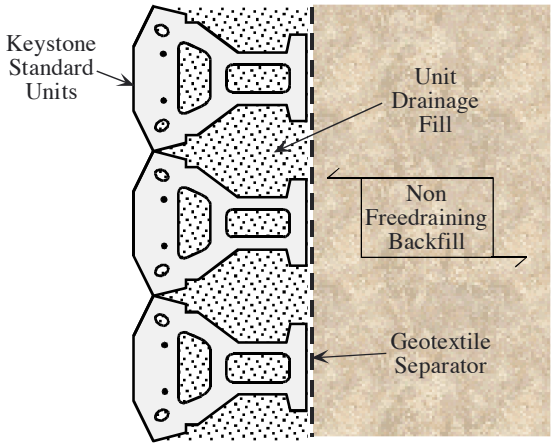


Unit Drainage Fill/Select Backfill

A. Select Backfill - When the reinforced backfill material is a select granular material which drains easily, a geotextile separator may be used to contain the drainage fill within the Keystone unit allowing placement of select backfill first followed by the drainage fill within the units.



Unit Drainage Fill/Non-Select Backfill



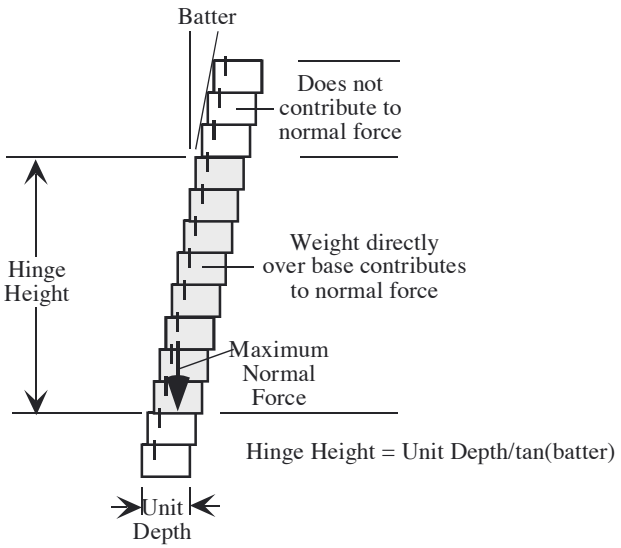
Unit Drainage Fill/Non-Select Backfill

B. Non Select Backfill - Keystone Standard units may be utilized with a geotextile separator against the tail of the units in lieu of the full 24" drainage zone in most applications to improve construction efficiency without significantly reducing drainage capability. The backfill can be placed against the geotextile first followed by the drainage fill within the units.

Hinge Height

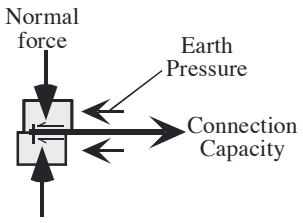
Hinge Height is a concept identified in the NCMA Design Manual for Segmental Retaining Walls which describes the limits of a downward or normal force application in a battered wall structure. The Hinge Height limitation applies primarily to the connection strength evaluation at a specific reinforcement level and the inter-unit shear and sliding calculations. The concept can also be extended to external stability calculations such as sliding and global stability analysis where normal gravity forces must be calculated.

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

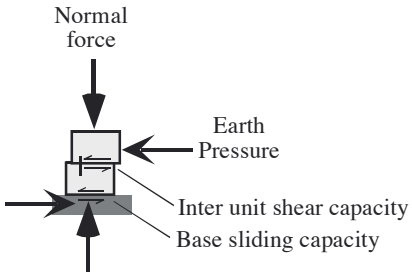


Hinge Height Wall Section

Hinge Height limitation is used for:



1. Connection Strength Evaluation



2. Sliding or Shear Evaluation

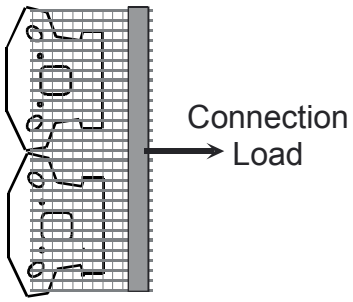
The Hinge Height calculation is not a significant design consideration in vertical or near vertical wall structures. It does becomes a serious design limitation in heavily battered structures with small facing elements where sliding resistance and geosynthetic reinforcement connection capacity are reduced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

Hinge Height Limiting Values

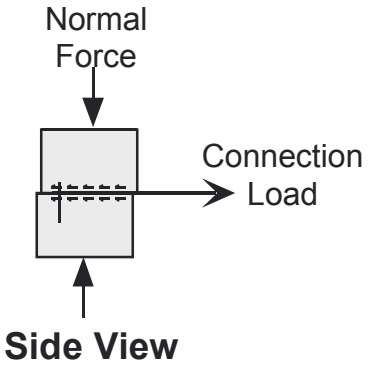
Keystone Unit	Unit Depth	Batter		
		0°	3.6°	7.1°
Standard Unit	1.79'	None	28.5'	14.3'
Compac Unit	1.0 '	None	16.1'	8.0'

Connection Strength

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, 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.

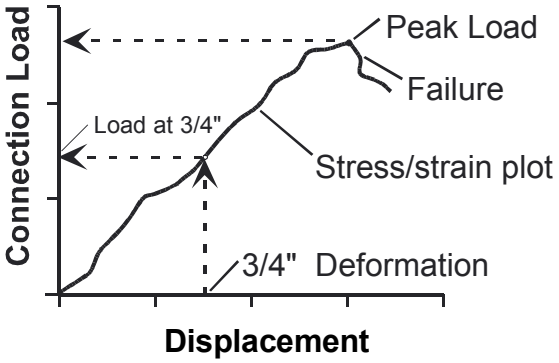


Top View

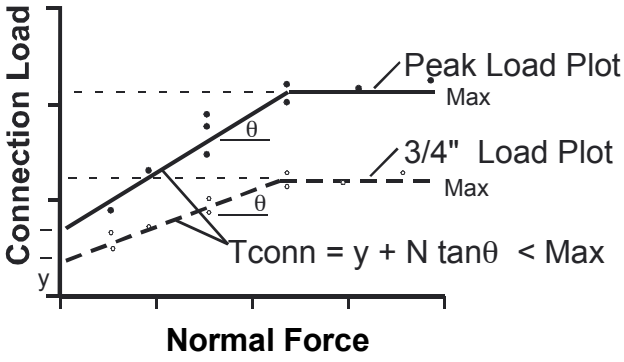


Side View

Connection Test Setup



Load test at one normal force



Connection Load Plots

Full laboratory connection testing has been performed with Keystone Standard and Compac units and geosynthetic soil reinforcement products by Tensar, Mirafi, Stratagrid, Huesker, and Amoco. This data is utilized in each wall design to determine the maximum permissible connection value that can be utilized at each reinforcement level.

The larger Standard unit has considerably more connection capacity than the smaller Compac unit and should be utilized 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 published design standards.

Connection Strength

The proper 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 the 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 some years ago and no longer neglect these more mundane structural calculations. The authors of these "state of the practice" 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 the wall facing system and wall stability during construction loading.

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 is calculated. This connection strength calculation is made at each reinforcement level and a minimum safety factor of 1.5 against rupture must be maintained.

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 or closer vertical spacing of the soil reinforcement to resist or lower the tensile loads in each element in order to maintain acceptable connection safety factors in accordance with published design standards.

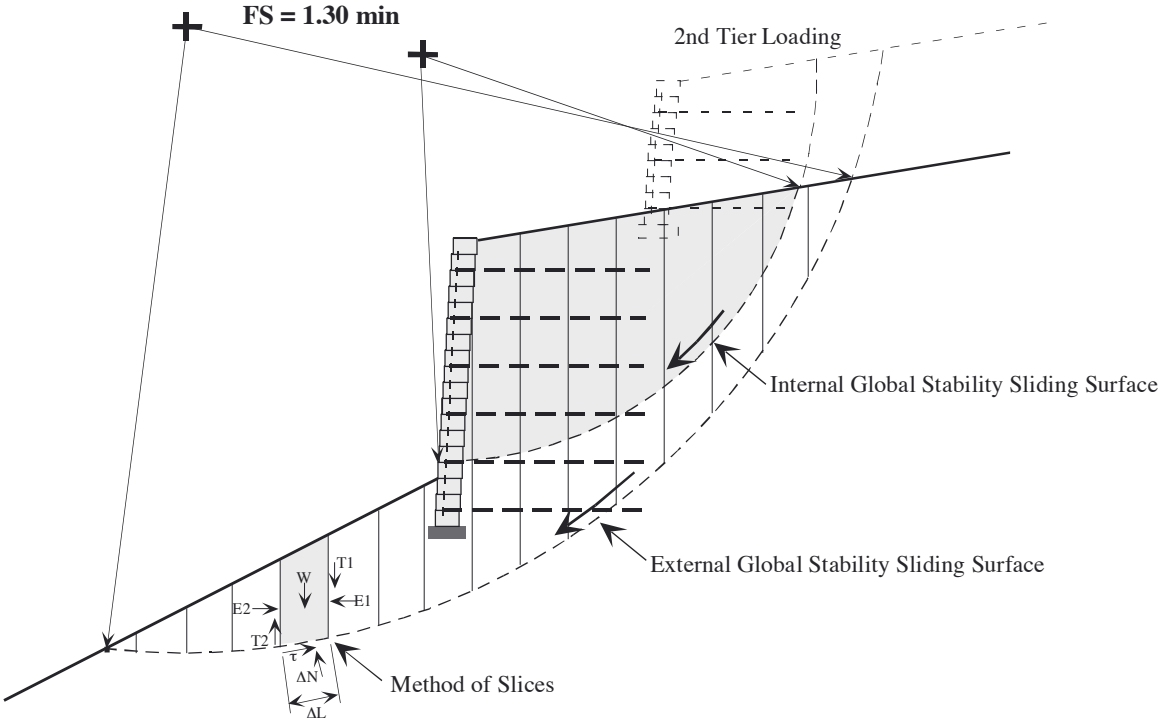
Since the connection strength analysis can be a limiting design factor and require additional soil reinforcement costs to satisfy the required design standards, those not skilled in retaining wall design sometimes ignore, neglect, or down play the connection strength evaluation as a means of reducing cost, increasing competitive position, or otherwise hiding a potential structural limitation in the proposed retaining wall system. This practice is not professional and leads to structure designs that provide less than the required levels of design safety and potential "negligence" claims in the eyes of the legal community when there are performance problems.

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

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, poor soils, and/or tiered wall sections. Global stability analysis looks at a rotational or compound failure mechanism which is significantly different than a simple sliding and overturning analysis.

Global stability analysis provides lower calculated factors of safety than simple sliding and can not be easily "tricked" by artificially low earth pressure calculations for heavily battered walls. 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 and STABL programs 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.

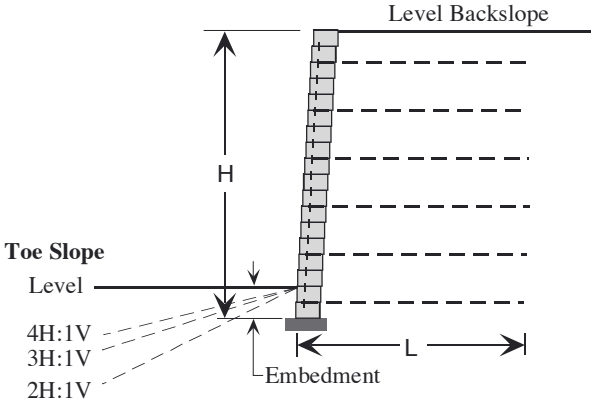
A minimum safety factor of 1.3 is typically required for retaining structures, however, this factor may be increased to 1.5 for critical wall structures such as bridge abutments per AASHTO code.

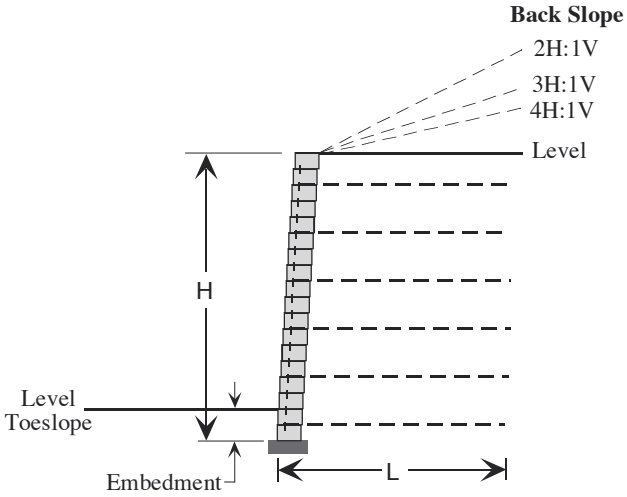
Single Wall - Slope Stability Ratios

The following figures and graphs provide a guide to the relationship between walls and slopes and the L to H ratio required to satisfy basic global stability requirements for simple ϕ only soil strength criteria. Slopes 2H:1V and greater require special attention to soil design parameters.

Assumptions of Stability Analysis

No significant surcharge, γ = 120 pcf, SF> 1.3 min, Bishop.
Vertical reinforcement spacing ~ 2',
Lowest reinforcement ~ 1' from bottom
LTDS of Reinforcement > 1,300 plf min. - upper 10 ft.
> 2,000 plf min - next 10 ft., etc.





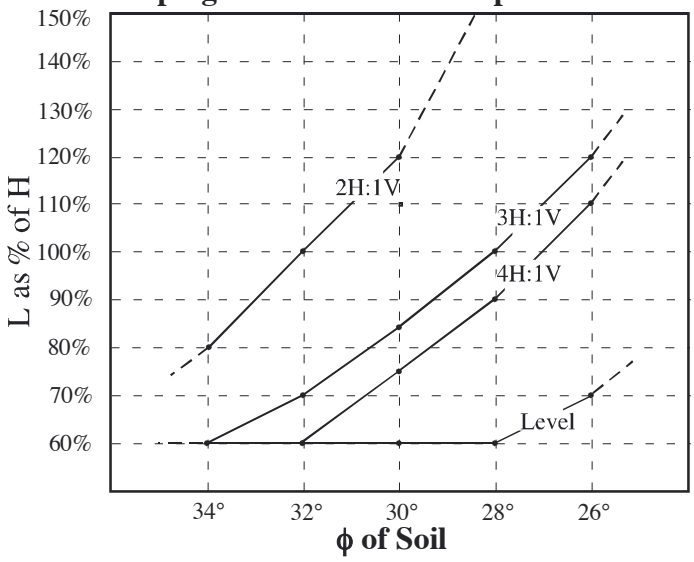
Min. Embedment for Toeslope

Level	10% H
4H:1V	1.0' + 10% H
3H:1V	1.3' + 10% H
2H:1V	2.0' + 10% H

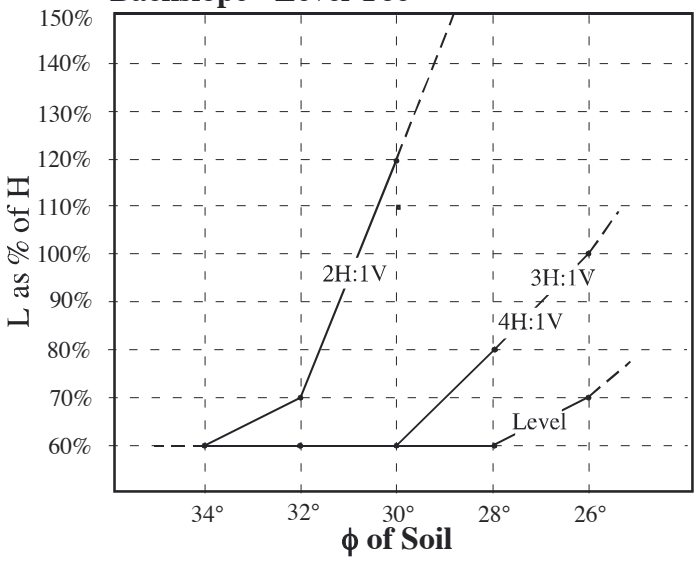
Min. Embedment for Backslope

Level	10% H
4H:1V	10% H
3H:1V	10% H
2H: 1V	10% H

Sloping Toe - Level Backslope



Backslope - Level Toe

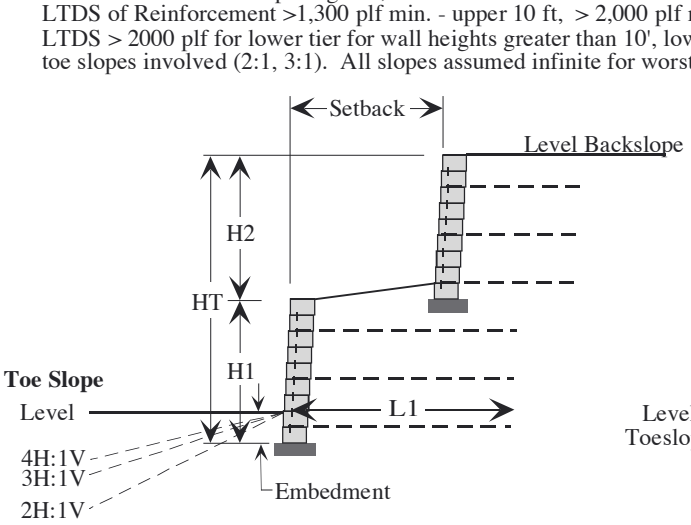


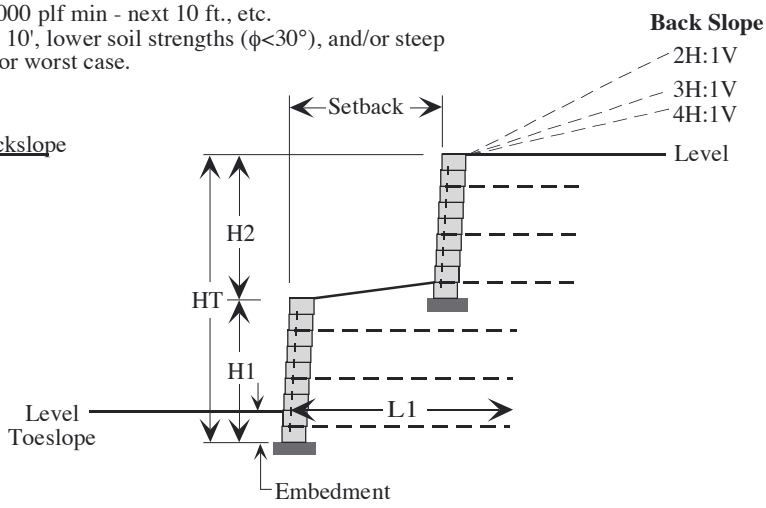
Tiered Wall - Slope Stability Ratios

The following figures and graphs provide a guide to the relationship between tiered walls and slopes and the L1 to HT ratio required to satisfy basic global stability requirements for simple ϕ only soil strength criteria. Slopes 2H:1V and greater require special attention to soil design parameters.

Assumptions of Stability Analysis

H1 ~ H2 ~ Setback. Note: Closer spacing is better for global stability, worse for stress.
No significant surcharge, γ = 120 pcf, SF>1.3 min - Bishop , Top of lower wall ~ Bottom of upper wall
Vertical reinforcement spacing ~ 2', Lowest reinforcement ~ 1' from bottom
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.



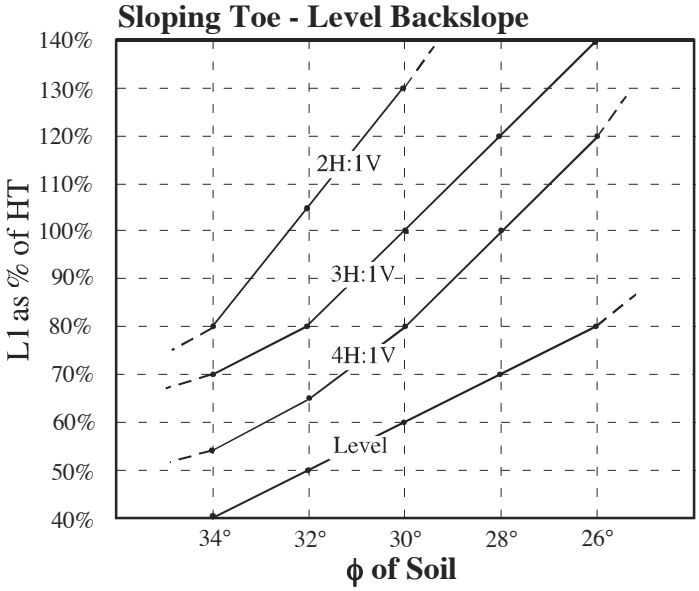


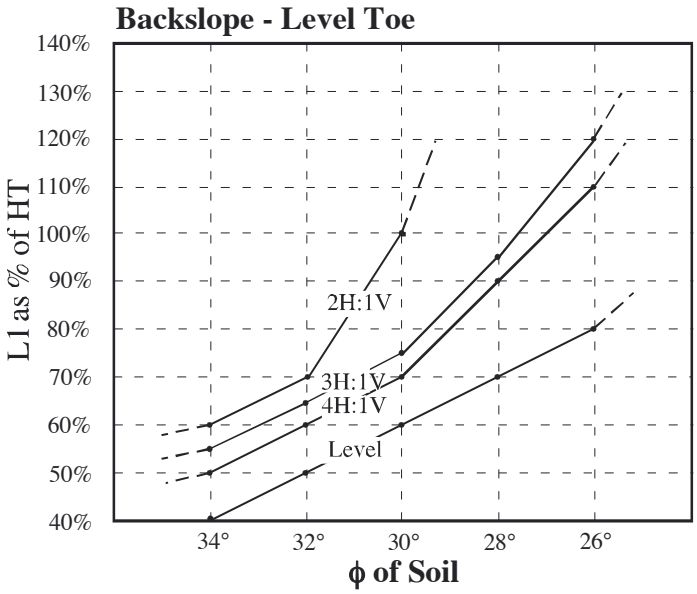
Min. Embedment for Toeslope

Level	10% HT
4H:1V	1.0' + 10% HT
3H:1V	1.3' + 10% HT
2H:1V	2.0' + 10% HT

Min. Embedment for Backslope

Level	10% HT
4H:1V	10% HT
3H:1V	10% HT
2H: 1V	10% HT



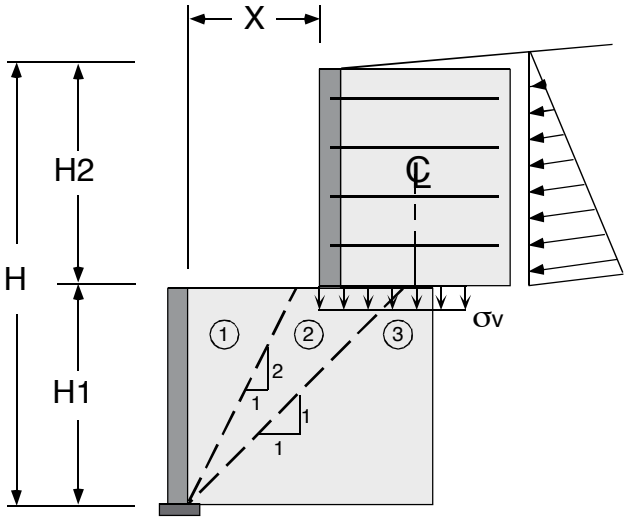


Tiered Wall - Internal Analysis

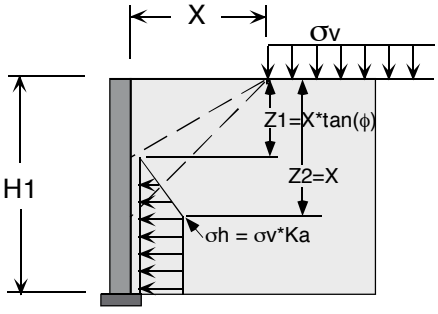
The external stability analysis for tiered or terraced wall structures is primarily accomplished by global stability analysis software used in conjunction with wall design software. Global stability analysis should also check for internal failure planes passing through the lower wall, insuring that the reinforcement is long and strong enough, but determining the actual 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 special software. Approximation techniques can be utilized but may be unduly conservative due to the obvious 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



Load Approximation

Zone Analysis

① When the upper wall setback falls within Zone 1 (X < H1/2) or X < 4' minimum, the upper wall fully surcharges the lower wall and the lower wall should be designed accordingly keeping in mind that connection strength is affected by splitting the walls apart a short distance.

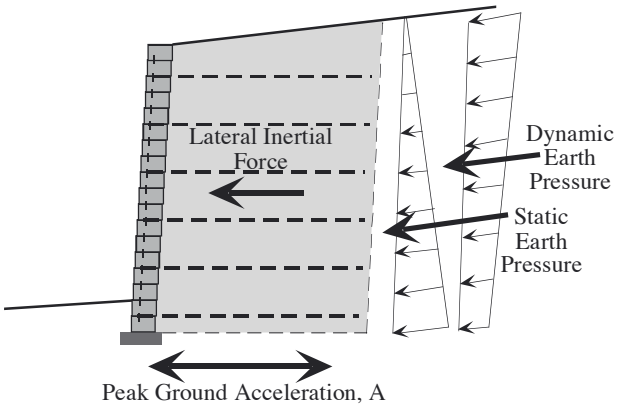
② When the upper wall setback falls within Zone 2 (H1/2 < X < H1), the upper wall surcharges the lower portion of the lower wall as indicated and the reinforcement design of the lower wall should account for the additional surcharge.

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 only be checked for overall/global stability.

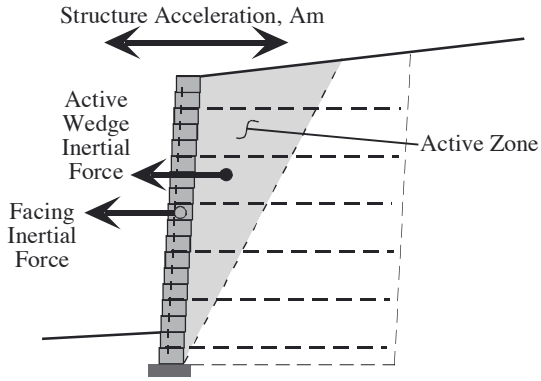
Seismic Design

Keystone retaining wall structures have proven to be earthquake resistant due to the system's 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. Most codes are silent on retaining wall seismic design criteria or methods and the issue is left to owners and engineers on a project by project basis.

The only published seismic design standards are contained in the AASHTO Standard Specifications for Highway Bridges which describe a psuedo-static method of analysis based on the Mononobe-Okabe application of conventional earth pressure theory. A schematic of psuedo-static analysis considerations is shown below as it pertains to soil reinforced structures.



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.

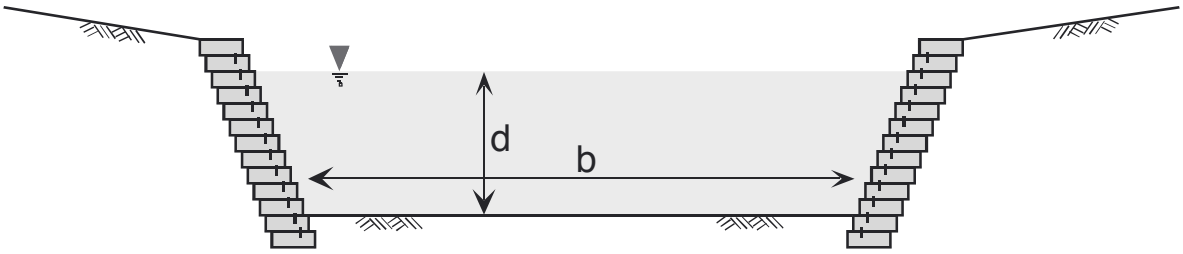
Sliding, overturning, and bearing pressure are analyzed in the conventional manner including the additional driving components of dynamic earth pressure and structure inertial force. Peak bearing pressure and eccentricity can also be checked but there is no particular acceptance criteria for these items. Soil liquefaction 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.

Water Flow - Manning's Number

Keystone walls are increasingly being 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 crossection and Manning's "n" values are provided below. Since only the tri-planer split-faced units were tested, we believe that straight split-faced units would provided slightly lower values due to less facial relief if required.



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

Where: V = velocity (feet per second)
n = Manning roughness coefficient
R = hydraulic radius (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 - Utah State University 1991

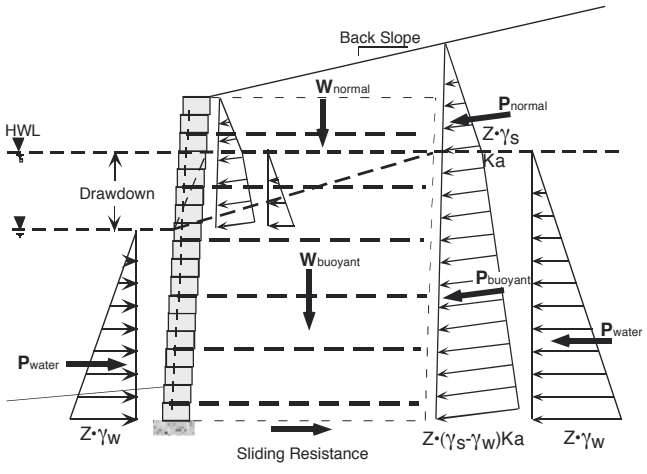
Rapid Drawdown Analysis

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 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, the 1996 AASHTO code says "For structures along rivers and canals, a minimum differential hydrostatic pressure equal to 3 feet of water shall be considered for the design....". The Army Corps of Engineers' Retaining and Flood Wall Design Manual suggest that a sliding safety factor of 1.33 may be appropriate for unusual or water to the top of wall conditions as well as lower factors for bearing capacity, etc, during the design event.

It is important to properly address internal and external stability of a reinforced soil structure for the 3' drawdown condition. Internally, the hydrostatic pressure differential can create additional loading on the reinforcement and connections while decreasing normal load in the pullout resistance calculations. Externally, high water reduces the effective weight of the mass (buoyancy) and sliding resistance is decreased. Global stability and foundation stability must also be reviewed against the varying water conditions.



Typical Design Section

Drawdown Design Guidelines

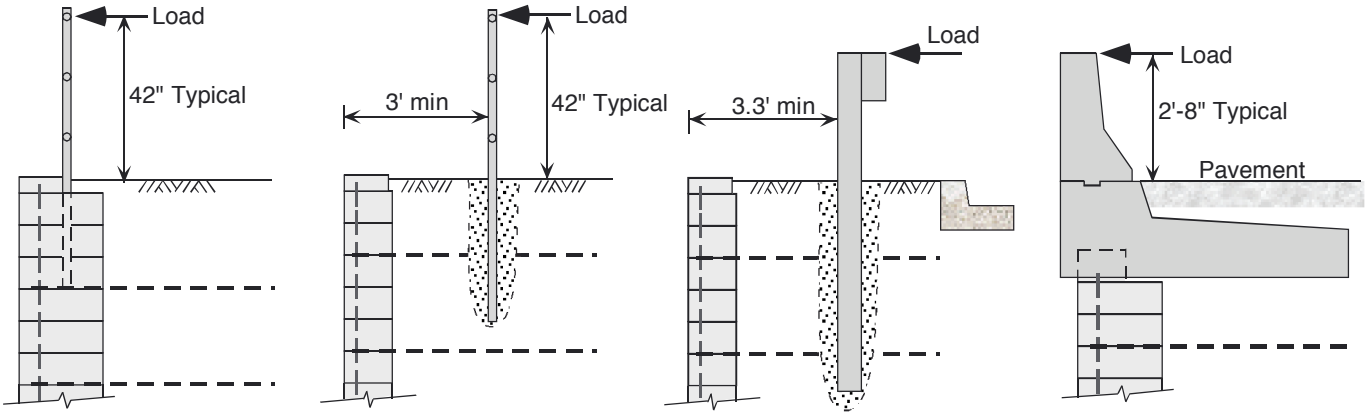
* 3' drawdown condition is a typical design requirement.

* Drawdown evaluation utilizes lower safety factor for combined loading analysis. (COE Guideline SF> 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 only.

* Liberal use of free draining backfill material minimizes internal drawdown conditions and associated pressure differentials.

Railing and Barrier Requirements



Railing-Direct Mount **Railing - Offset** **Guardrail** **Traffic Barrier**

Introduction

Railing, guardrail, 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 railings 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.

Guard 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. Some excerpts from design codes may be useful in defining the general intent of barriers:

Guardrail (UBC)

Guardrail is a system of building components located near the open sides of elevated walking surfaces for the purpose of minimizing the possibility of an accidental fall from the walking surface to the lower level.

Railing/Guard Requirements (BOCA)

Where retaining walls with differences in grade level on either side of the wall in excess of 4 feet (1219 mm) are located closer than 2 feet (610mm) to a walk, path, parking lot or driveway on the high side, such retaining walls shall be provided with guards that are constructed in accordance with Section 1021.0 or other approved protective measures.

Railings (AASHTO)

Railings shall be provided along the edges of structures for protection of traffic and pedestrians.

Summary

The railing/barrier issue can be a logistical and structural problem with modular wall systems due to the inability of the small wall units to resist concentrated loads and the need for lateral space at the top of wall to install most barrier systems. Proper planning and design is required.

The design loadings can be quite significant as indicated below:

UBC Railing and Guardrail Loadings

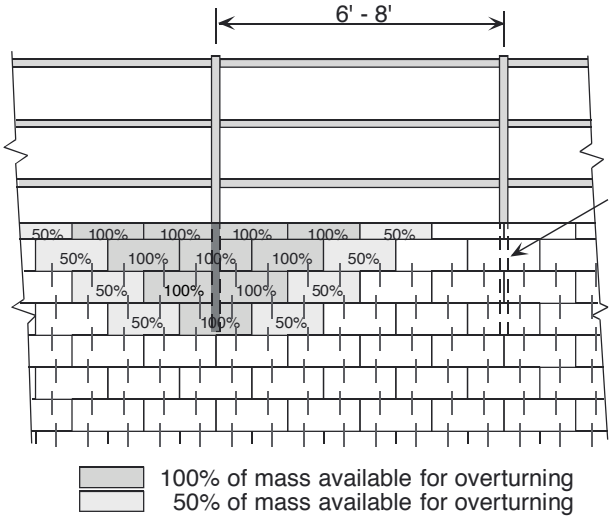
* Other than exit facilities	20 plf
* Exit facilities serving an occupant load greater than 50.	50 plf
* Minimum point loading	200 lbs
* Vehicle Barrier	6000 lbs

AASHTO Railing Loadings

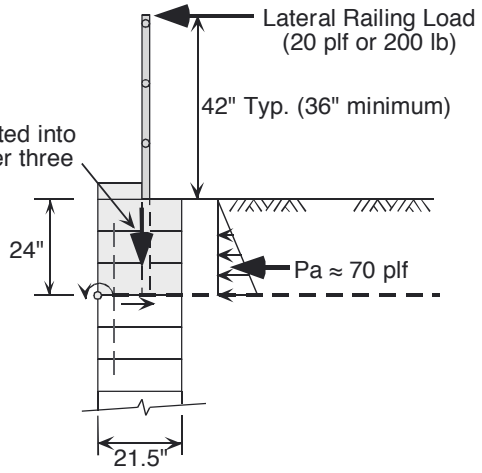
* Pedestrian Railing (W)	50 plf
* Traffic Barrier (P)	10,000 lbs

Typical Railing Design

Direct Mount - 20 plf -Standard Units



Typical Railing Elevation



Typical Railing Design 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 UBC for more critical conditions and by AASHTO for highway projects requires that the top of wall analysis be treated differently.

Driving Moments **(20plf 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

SFot = 1835/1194 = **1.53** ≥ 1.50 minimum, **OK**

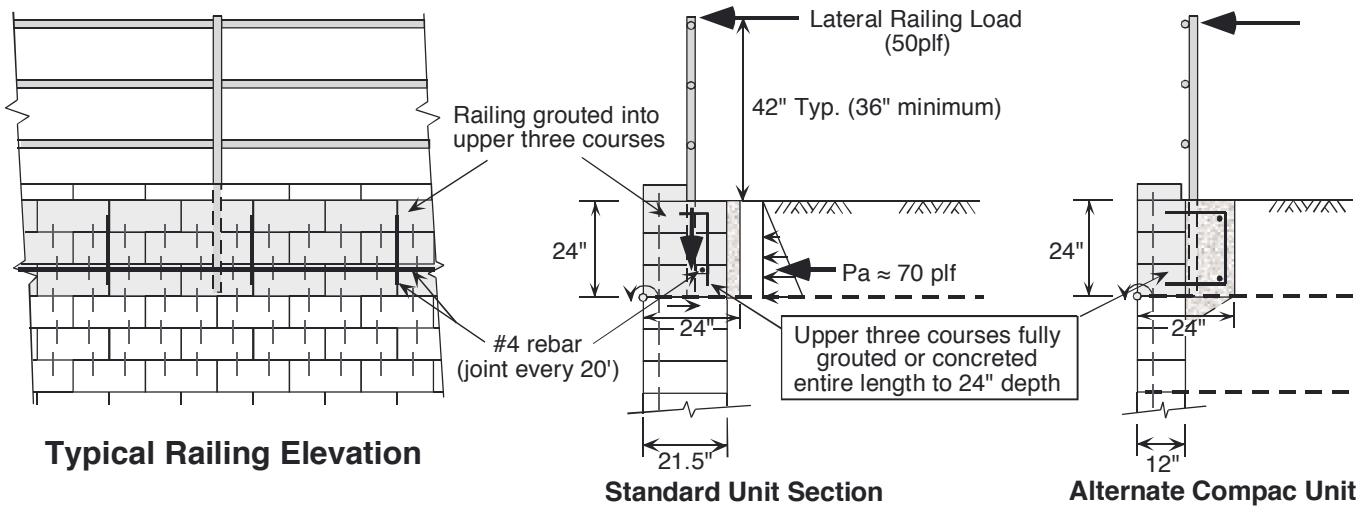
Design Note:

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.

Typical Railing Design

Direct Mount - 50 plf -Standard/Compac Units



Typical Railing Elevation

Standard Unit Section

Alternate Compac Unit

Typical Railing Design 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 a 50 plf AASHTO/UBC lateral design loading without additional structure.

Railing Analysis

Shear resistance of Standard units (>1000 plf) and Compac 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.

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

SFot = 575/322 = **1.78** < 1.50 minimum, **OK**

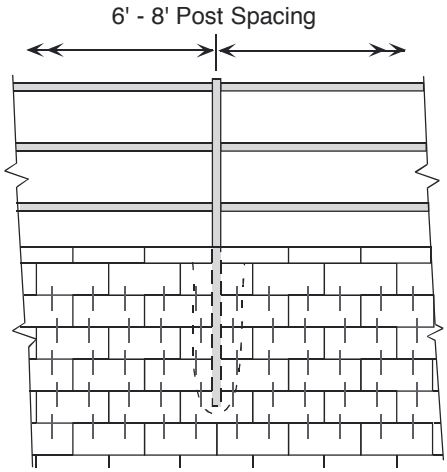
Design Note:

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

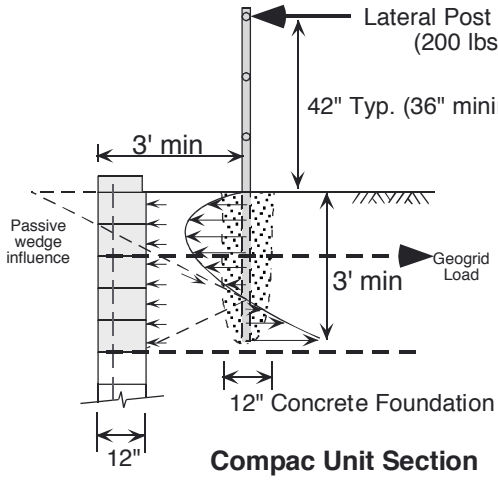
Keystone Compac units require additional reinforcement and concrete to provide the overturning mass necessary to resist design loadings. Compac unit designs should consider offset railings as a simpler and more economical alternative.

Typical Railing Design

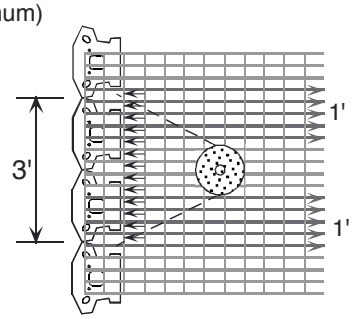
Offset Installation - 20 plf - Compac Unit



Typical Railing Elevation



Compac Unit Section



Plan View

Typical Railing Design Section

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 (ie: Strata 200 - 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	200 lbs x 5.5'/1.0'± ratio	= 1100 lbs
Soil	0.5 x 120 x $(2.33)^2$ x 0.3 x 3'	= 294 lbs
	Total	= 1394 lbs

Resisting Strength @ Post

Geogrid 2' x (LTDS @ 1200 plf)	= 2400 lbs
Geogrid 2' x (Connection @ 960 plf)	= 1980 lbs
Passive Soil Shear	
2' x 1.5' x 1' x 120 pcf x Tan 30°	= 207 lbs

SF = (1980+207)/1394 = **1.56** < 1.50 minimum, **OK**

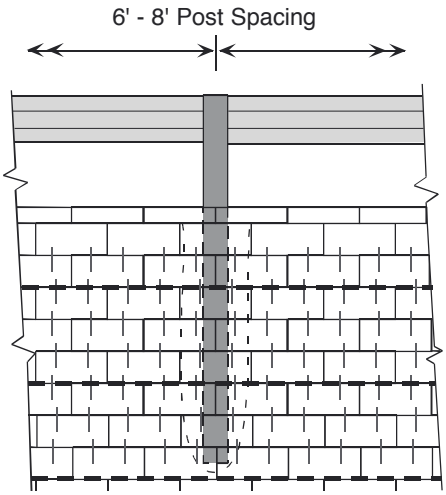
Note: Only thrust is calculated above. Moments could also be calculated but the concept is to have the reinforcement placed high enough to absorb the thrust without permitting the wall to be pushed out.

Design Note:

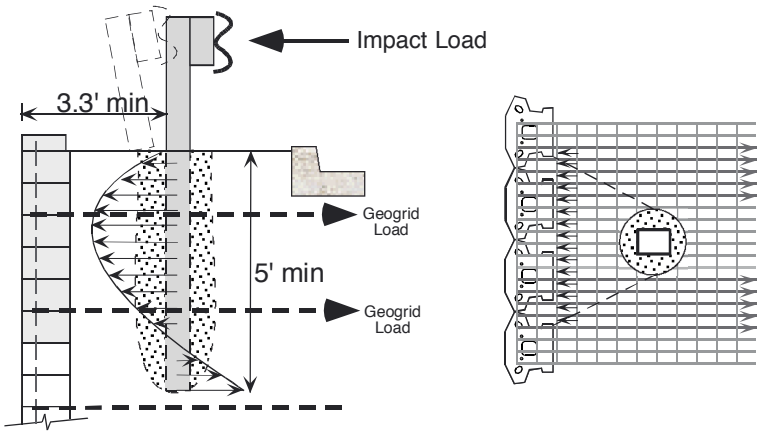
The design shown is the minimum requirement. Larger loadings will require greater offset, greater depth, and/or additional reinforcement to provide local design stability.

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.

Typical Guardrail Detail



Typical Guardrail Elevation



Compac Unit Section **Plan View**

Typical Guardrail Section

Introduction

Guardrails or flexible post and beam barriers are not "designed" in the conventional sense as a guardrail is not expected to resist traffic impact loadings and remain serviceable like other barriers. The guardrail 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 guardrail 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 1m (3.3') minimum (can be up to 1.5m (5') with more flexible rail systems) which can be a problem when not properly accounted for on project site design plans.

Guardrail Analysis

The analysis of a Keystone wall structure with a guardrail 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 design criteria appears to be a reasonable consideration:

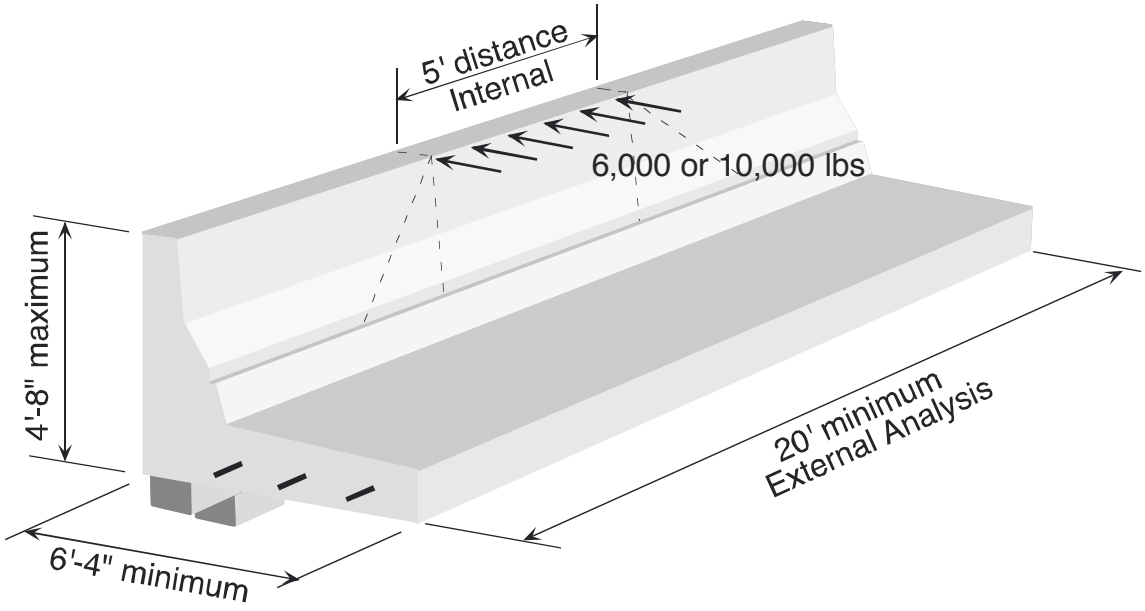
AASHTO '97

Flexible post and beam barriers, when used, shall be placed a minimum distance of 1.0 m (3.3') from the wall face, driven 1.5 m (5') below grade and spaced to miss the reinforcements.... The upper two rows of reinforcement shall be designed for an additional horizontal load of 4.4 kN per lineal meter of wall (300 plf).

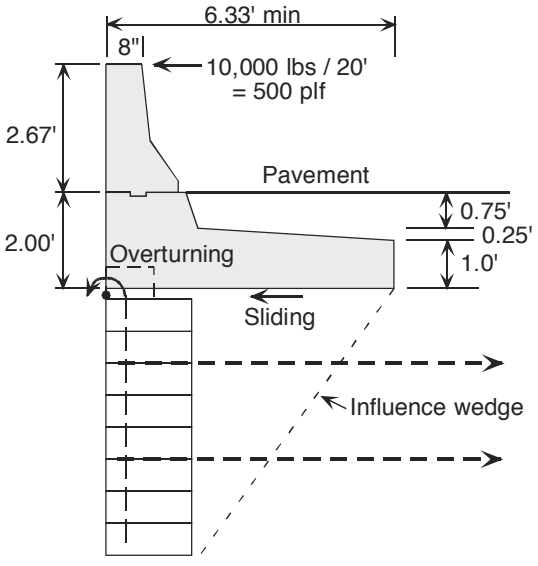
Design Note

Two levels of reinforcement are required in the upper four feet 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 a drive point is used to cut through the reinforcement.

Typical Traffic Barrier Design



Typical Barrier Section



Typical Design Section

Barrier imparts additional 500 plf to upper reinforcement levels. Note: some AASHTO interpretations require 2000 plf to be added to the upper reinforcement levels regardless of slab configuration.

"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 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 psuedo-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).

The analysis of the upper two reinforcement levels for this additional loading may consider reduced safety factors 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.

Quality Assurance Provisions

Design Provisions

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

	ϕ	c	γ	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 be confirmed by the site geotechnical engineer prior to wall construction.

2. The walls are designed to support the following maximum surcharge loadings:

Live Load -	250 psf	Wall 1, 2
Backslope -	3H:1V max	Wall 3
Seismic -	A = 0.20g	All Walls
Hydrostatic -	3' drawdown	Not applicable

   The wall design maintains a minimum factor of safety of 1.5 on all elements of the static wall design unless otherwise noted in the calculations. Global stability, when evaluated, maintains a minimum factor of safety of 1.3 unless otherwise noted.

3. The wall foundation soils at each wall location shall be capable of safely supporting 3000 psf without failure or excessive settlement. Local bearing capacity shall be confirmed by the site engineer .

Construction 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 backslope 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.

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. The precision of setting the first course is proportional to the amount of time spent on this function. Subsequent 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, the misalignment becomes increasingly noticeable to all involved and becomes a very difficult problem to correct at a later time without dismantling 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 or 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

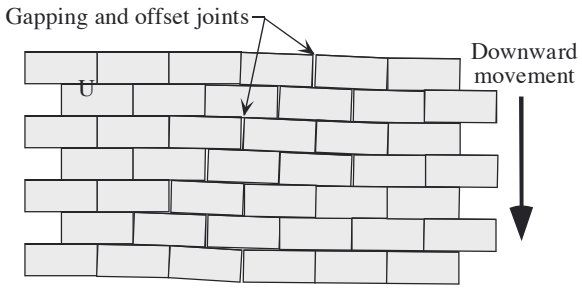
Vertical misalignment is not as obvious as horizontal misalignment but contributes equally to the problems noted in taller walls. Every course of wall units should be checked for levelness with minor shimming done every course or two to insure that the units remain true to the design batter. Walls can 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.

Shimming should be limited to a maximum of 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.

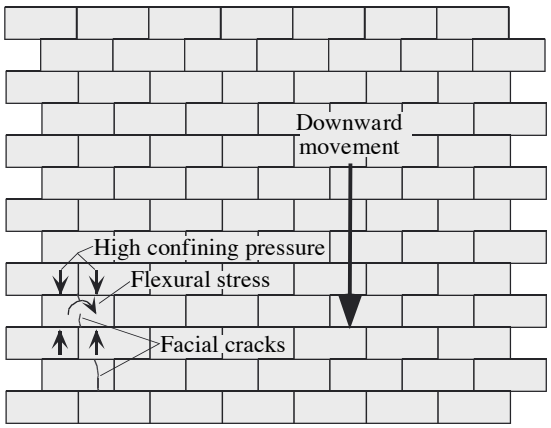
Unit Cracking/Gapping - Settlement

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 or 1% and 1/200 or 1/2 % have been suggested by NCMA and FHWA 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

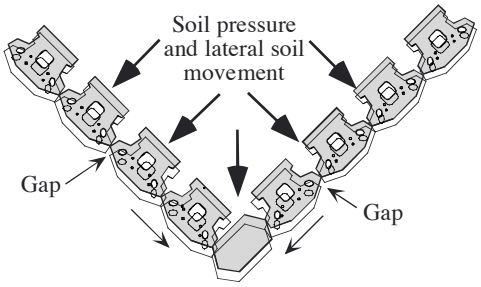
Low wall settlement problems are typically observed in residential projects where soils adjacent to houses are uncompacted and the walls settle differentially over a short distance. Usually gapping or offset joints are visually noted and the settlement is obvious.

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 facing system.

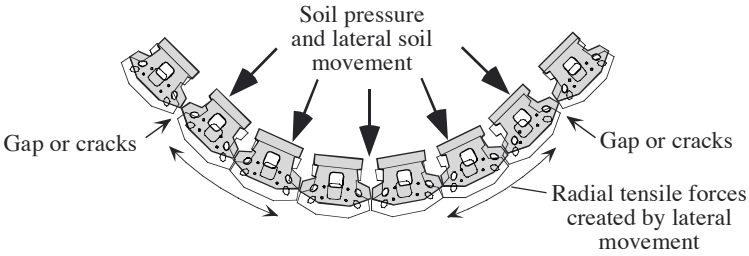
Cracked units can also be a symptom of other types of more serious problems so a review by a qualified engineer is always recommended.

Unit Cracking/Gapping - Corners/Bends

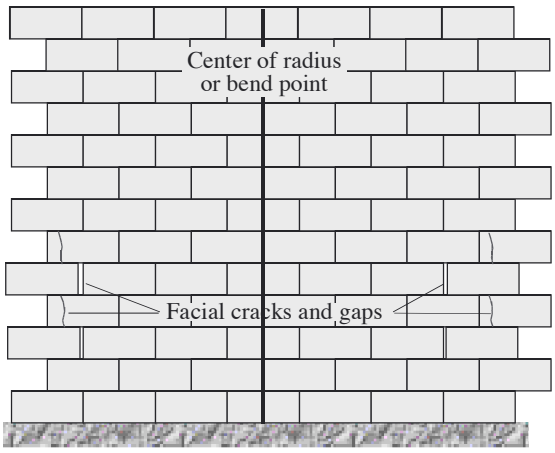
Keystone modular retaining wall structures can tolerate a certain amount of movement due to the flexible nature of the system and small individual unit size. When corners and tight curves are inserted into an otherwise two dimensional system, a third direction of movement can occur which can cause unit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



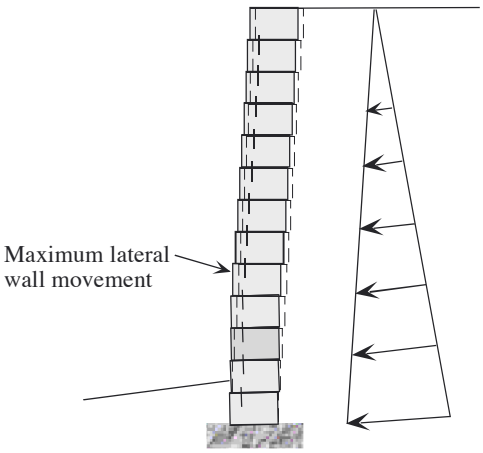
Wall Corner



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.

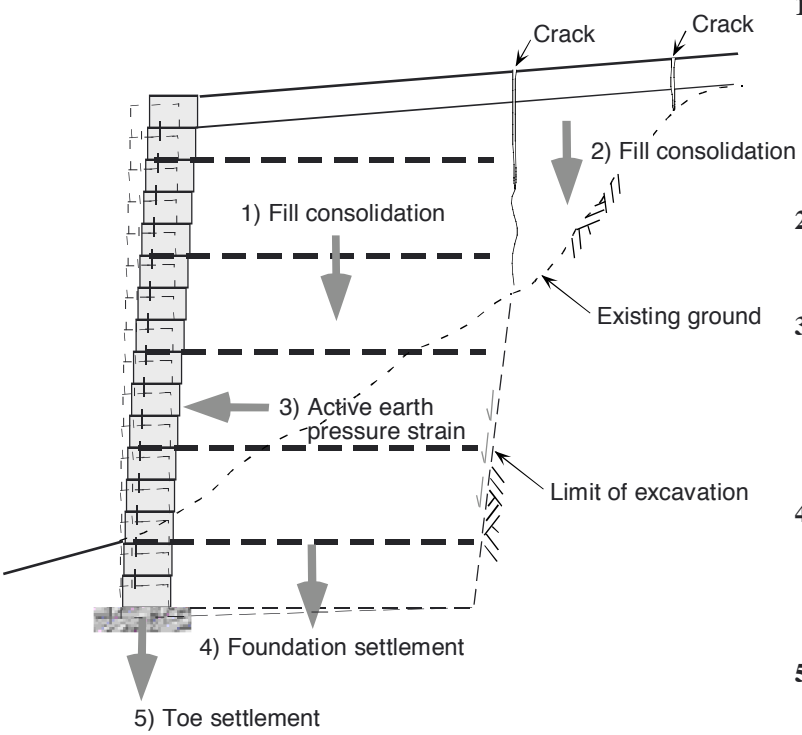
Wall Translation Potential
Active Earth Pressure Theory

Soil Type and Conditon	% 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

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.

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.

The most current reference specifications through 2000 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 2000 - Section 7 - 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 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. Testing in water is recommended in accordance with ASTM C1262 or C666 at the number of cycles and weight loss required by the Owner.