



# Keystone MSE Wall Design and Seismic Applications

- 4/5/16 to 4/7/16

## 1. Product Offerings

1. Extensible - Geosynthetic
2. Inextensible – Steel
3. Research, Development and Testing

## 2. Design Basics and Introduction to Methodologies

1. Differences
2. Comparison

## 3. Introduction to Seismic Design Principles

1. Discussion of the current state of design practices

## 4. Keystone Wall Design Software (Interactive)

1. KeyWall
2. KeyDraw
3. KeySystem I Spreadsheet

- First to market and #1 Structural SRW in the World
- Headquarters in Minneapolis, MN– Worldwide  
Distribution network
- Leader in Engineering Development of SRW systems
  - First segmental retaining wall to market (1986)
  - 30 years of innovation: 180+ Patents / Patents Pending
- Keystone Departments
  - Engineering
  - Marketing
  - Sales
  - Product development and production



**WAIKATO EXPRESSWAY: RANGIRIRI  
FIRTH COMPAC IV w GEOGRID**

- Entry Level Structural Product
- Create small freestanding walls, parapet walls, pilasters, columns, and retaining walls.
- 3 unit system with 7 unique face dimensions.
- Units are packaged together as a system to create a random, natural look.
- Three textured sides on each unit.
  - Used as an exposed end unit or a 90° corner.
- Shouldered pins give multiple setback positions.
  - Create vertical or setback walls



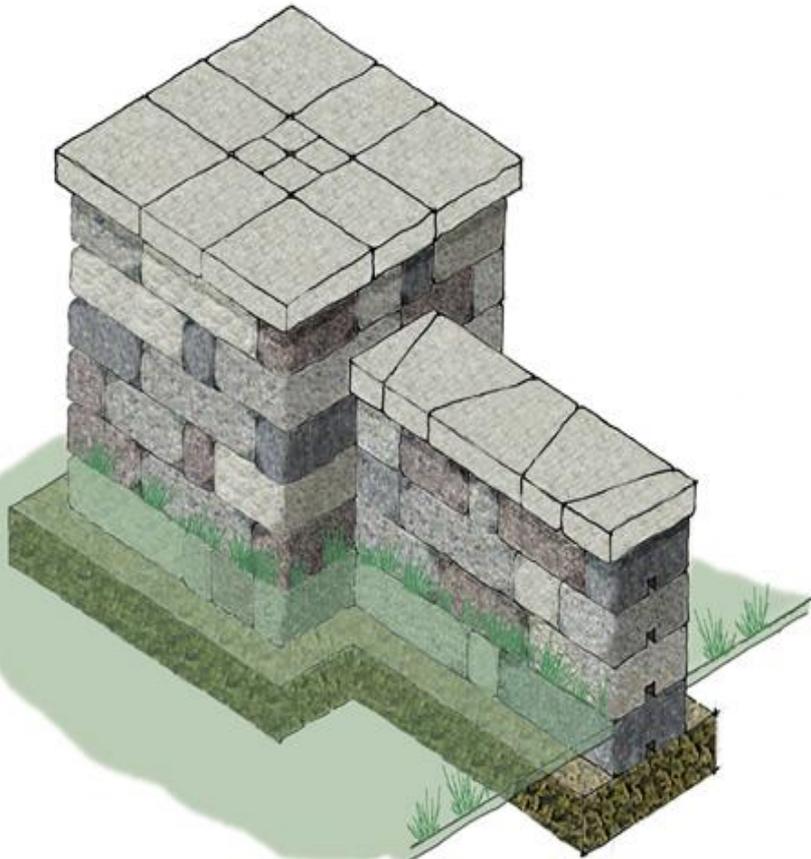
Piece #	1	2	3
Height	150	150	150
Width	400/350	300/250	150/100
Depth	250	250	250
Weight (kg)	27	18	11
Pins	Shouldered Pins		

Note: Unit colors, dimensions, weight, and availability vary by manufacturer.

3 pc. System (1;4;6)

System availability varies by manufacturer. Contact directly for details.

# Versatility – 3 Sides Textured





Unique  
Face  
Texture





# Keystone Standard



1. TRI-PLANE



2. STRAIGHT

Piece #	1	2
Height	200	200
Width	455	455
Depth	533	533
Weight (kg)	52	56
Pins	Straight Pins – 2/unit	

0.091 m<sup>2</sup> per unit

Note: Unit colors, dimensions, weight, and availability vary by manufacturer.

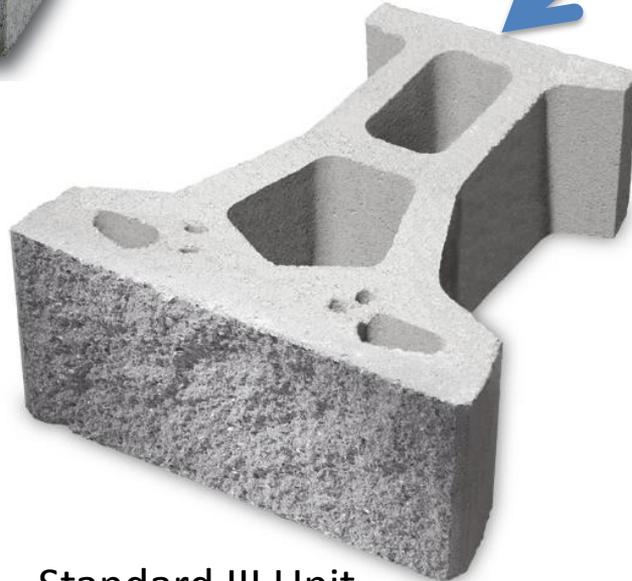
- First Segmental wall unit developed
- Early gravity applications, still best option for gravity wall applications
- Later geogrid introduced for taller walls
- 30 years of unit evolution
- Development of specialty applications for taller gravity walls
- ICC-ES report ESR-2113, ICC-ES is a subsidiary of International Code Council.
  - Evaluation service for independent verification of compliance to the International Building Code (IBC)



Standard I Unit



Standard II Unit



Standard III Unit

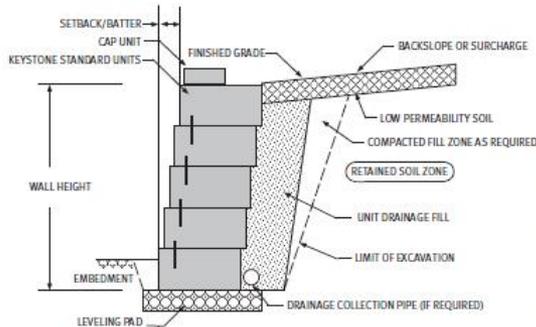
- Wider pin receiving hole for more alignment flexibility
- Vertically aligned cores for ease of core filling
- Individual unit weight decrease of 4.5 to 5.5 kg
- Superior connection strength
- Greater construction flexibility



## Gravity Wall Schematic



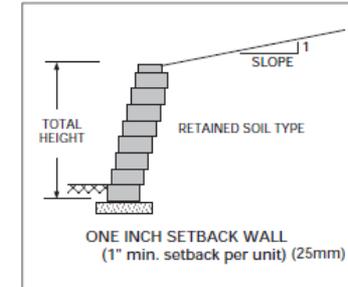
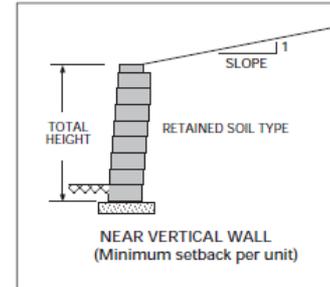
GRAVITY WALL SCHEMATIC



**NOTES:**

Wall Height (H) is the total height from top to bottom.  
 Minimum wall embedment is 6 inches (150mm) or Height/8, whichever is greater for level toe.  
 Subsurface soils must be capable of supporting wall system  
 Unit drainage fill is ¾ inch (20mm) clean crushed stone.  
 Leveling pad is crushed stone base material.  
 All backfill materials are compacted to 95% Standard Proctor Density or 92% Modified Proctor Density.  
 Finished grade must provide positive drainage.

## Maximum Height Gravity Wall Charts



### NEAR VERTICAL - STANDARD UNITS (18")

Soil Type	MAX. HGT.	BACKSLOPE		
		Level	4H:1V	3H:1V
Sand/Gravel	4.33' (1.30m)	3.67' (1.10m)	3.67' (1.10m)	3.00' (0.91m)
Silty Sand	3.67' (1.10m)	3.00' (0.91m)	3.00' (0.91m)	3.00' (0.91m)
Silt/Lean Clay	3.67' (1.19m)	3.00' (0.91m)	3.00' (0.91m)	<1.00' (0.30m)

### SETBACK OPTION - STANDARD UNITS (18")

Soil Type	MAX. HGT.	BACKSLOPE		
		Level	4H:1V	3H:1V
Sand/Gravel	5.67' (1.70m)	5.00' (1.50m)	5.00' (1.50m)	4.33' (1.30m)
Silty Sand	5.00' (1.50m)	4.33' (1.30m)	4.33' (1.30m)	3.67' (1.10m)
Silt/Lean Clay	4.33' (1.30m)	3.67' (1.10m)	3.00' (0.91m)	2.33' (0.71m)

### NEAR VERTICAL - STANDARD UNITS (21")

Soil Type	MAX. HGT.	BACKSLOPE		
		Level	4H:1V	3H:1V
Sand/Gravel	5.00' (1.50m)	4.33' (1.30m)	4.33' (1.30m)	3.67' (1.10m)
Silty Sand	4.33' (1.30m)	3.67' (1.10m)	3.67' (1.10m)	3.00' (0.91m)
Silt/Lean Clay	3.67' (1.10m)	3.67' (1.10m)	3.00' (0.91m)	2.33' (0.71m)

### SETBACK OPTION - STANDARD UNITS (21")

Soil Type	MAX. HGT.	BACKSLOPE		
		Level	4H:1V	3H:1V
Sand/Gravel	7.00' (2.10m)	6.33' (1.90m)	5.67' (1.70m)	5.00' (1.50m)
Silty Sand	5.67' (1.70m)	5.00' (1.50m)	5.00' (1.50m)	4.33' (1.30m)
Silt/Lean Clay	5.00' (1.50m)	4.33' (1.30m)	3.67' (1.10m)	3.00' (0.91m)

NOTES: CALCULATIONS ASSUME A UNIT WEIGHT OF 120 PCF (19kN/m<sup>3</sup>) FOR ALL SOIL TYPES. ASSUMED φ ANGLES FOR EARTH PRESSURE CALCULATIONS ARE: SAND/ GRAVEL=34°, SILTY SAND=30°, AND SANDY SILT/LEAN CLAY=26°. NON CRITICAL STRUCTURES WITH F<sub>s</sub>>1.5. CHARTS ARE PERFORMED USING COULOMBS EARTH PRESSURE ANALYSIS. (NEMA 3RD EDITION) NEAR VERTICAL WALLS UTILIZE 1" BATTER AND 1" SETBACK WALL UTILIZES 8" BATTER. NO SURCHARGES WERE USED IN THE ANALYSIS. SURCHARGES OR SPECIAL LOADING CONDITIONS WILL REDUCE MAXIMUM WALL HEIGHTS. SLIDING CALCULATIONS ASSUME A 6" (150MM) CRUSHED STONE LEVELING PAD AS COMPACTED FOUNDATION MATERIAL. THE INFORMATION PROVIDED IS FOR PRELIMINARY DESIGN USE ONLY. A QUALIFIED PROFESSIONAL SHOULD BE CONSULTED. KEYSTONE ACCEPTS NO LIABILITY FOR THE IMPROPER USE OF THESE TABLES.

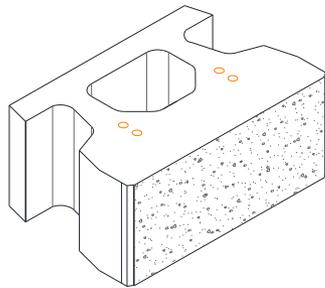
The information contained herein has been compiled by Keystone® Retaining Wall Systems, Inc. and to the best of our knowledge, accurately represents the Keystone product use in the applications which are illustrated. Final determination of the suitability for the use contemplated and its manner of use are the sole responsibility of the user. Structural design and analysis will be performed by a qualified engineer.

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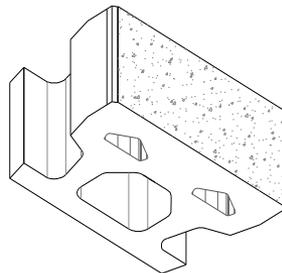
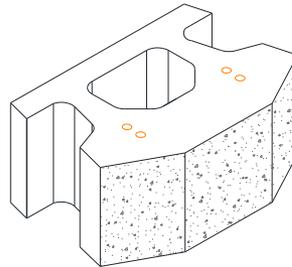
- Single Width Unit Maximum Heights (Good Soil Conditions)
  - Battered (no surcharge) – 2 m
  - Vertical (no surcharge) – 1.5 m
- Interlocked Back to Back Standard Units (Good Soil Conditions)
  - Battered (no surcharge) – 2.8 m
  - Vertical (no surcharge) – 2.2 m
- For most gravity application batter always recommended

# Keystone Compac IV

1. Straight Face Split



2. Tri - Face Split



Bottom View  
Of Compac IV



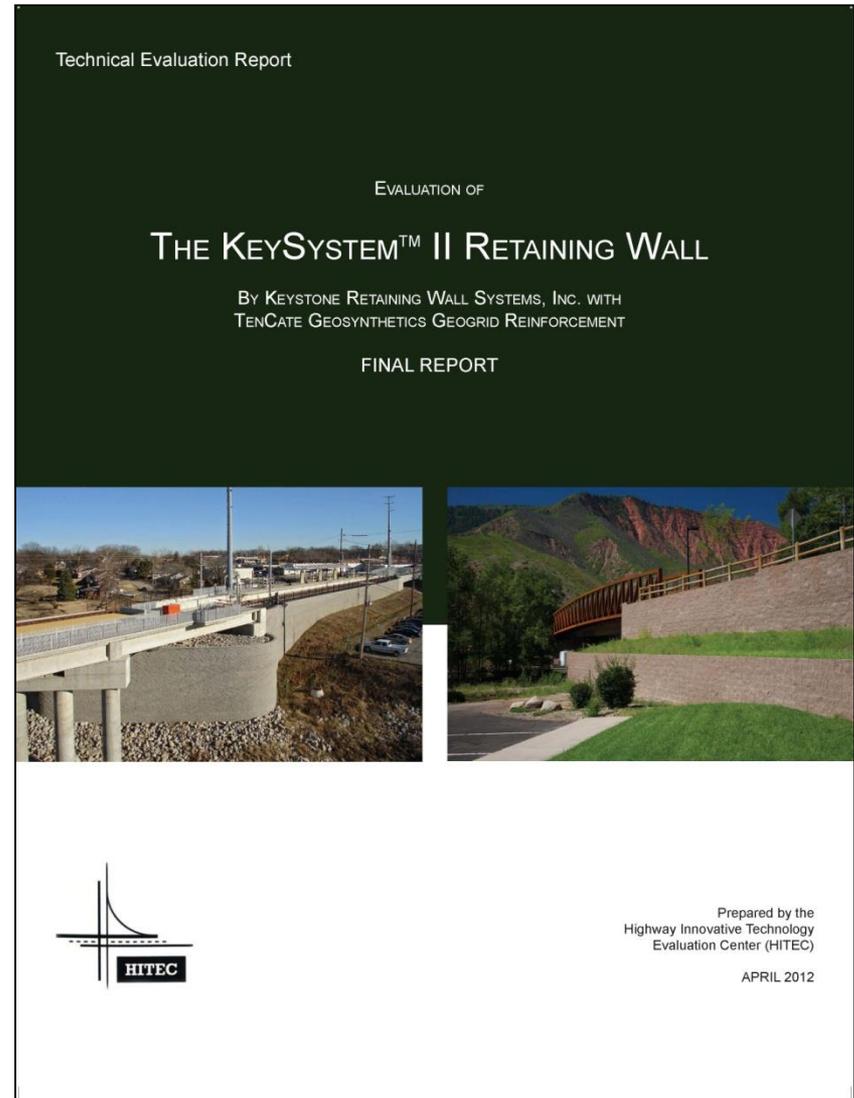
Piece #	1	2
Height	200	200
Width	455	455
Depth	305	305
Weight (kg)	37	34
Pins	Straight Pins -2/unit	

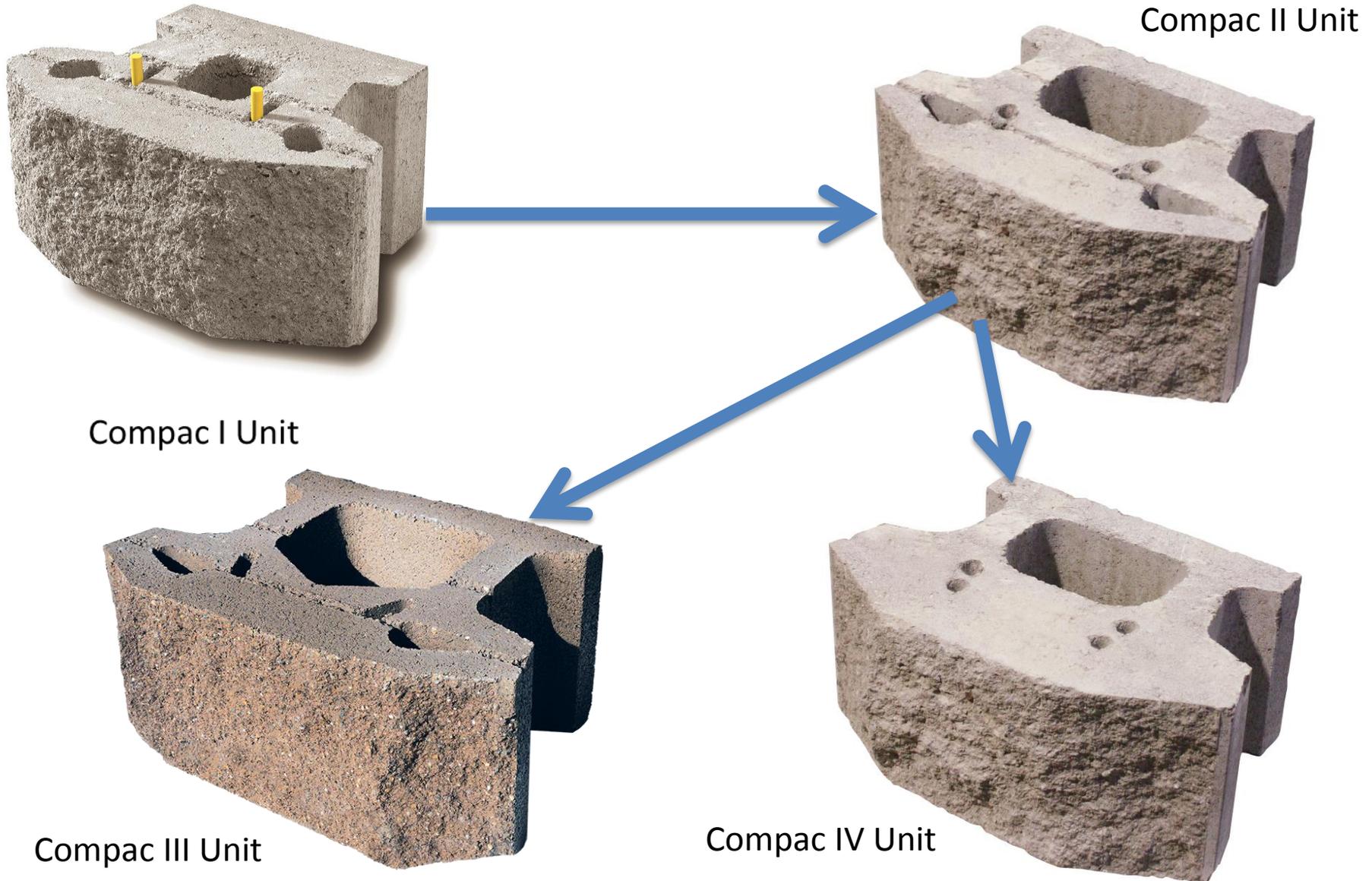
Face Area 0.091 m<sup>2</sup>

Note: Unit colors, dimensions, weight, and availability vary by manufacturer.

- Workhorse of Keystone – probably 75% of walls Nationally & Internationally are with Compac unit designs.
- When utilized with geogrid reinforcement, walls of all height can be designed.
- Highway Innovative Technology Evaluation Center (HITEC)
  - U.S. DOT highway geogrid wall system evaluated for long term connection
- ICC-ES report ESR-2113, ICC-ES is a subsidiary of International Code Council.
  - Evaluation service for independent verification of compliance to the International Building Code (IBC)

- Evaluation published April 2012
- Evaluation based on AASHTO LRFD 5<sup>th</sup> Edition, 2010 and NHI FHWA 2009
- Keystone Compac II Units
- Mirafi Geogrid





- Wider pin receiving hole for more alignment flexibility
- Vertically aligned cores for ease of core filling
- Individual unit weight decrease of 4.5 to 5.5 kg
- Superior connection strength
  - 40% to 100% increase in geogrid connection
- Greater construction flexibility

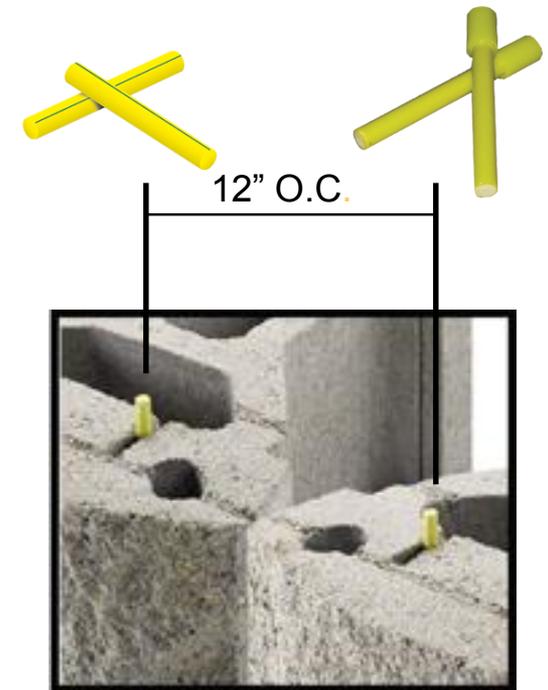




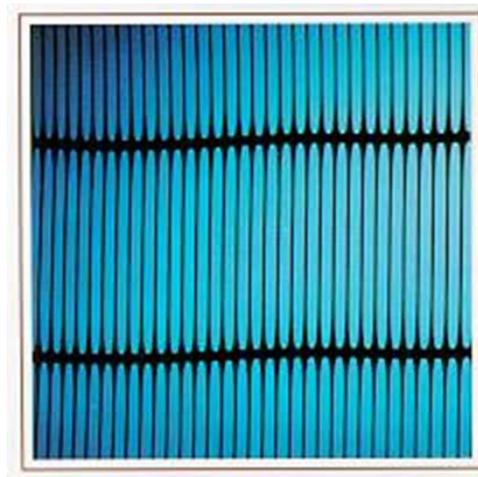
**WAIKATO EXPRESSWAY: RANGIRIRI  
FIRTH COMPAC IV w GEOGRID**

- Universal Facing Unit
  - One facing unit used to construct wall system
  - Universal facing unit allows on-site alignment changes without delays of casting specialty panels
  - Facing units are field cut for pipe penetrations or other obstructions
  - Facing unit can be field cut for slip joints in excessive settlement conditions
  - Face batter adjustable from 1H:8V to 1H:64V
  - Various color and texture options available

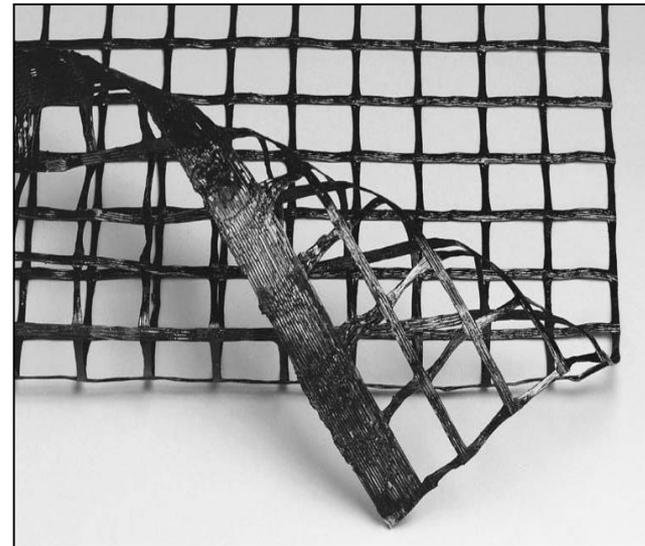
- Keystone Fiberglass Pins
  - Quick & easy alignment for stacking units
  - Provides various degrees of setback
  - Unit shear connector every 12"
  - 20% better connection strength
  - Allows geogrid to be pre-tensioned
  - Ensures grid is attached to wall
  - Non-corrosive
  - High strength 6,400 psi short beam



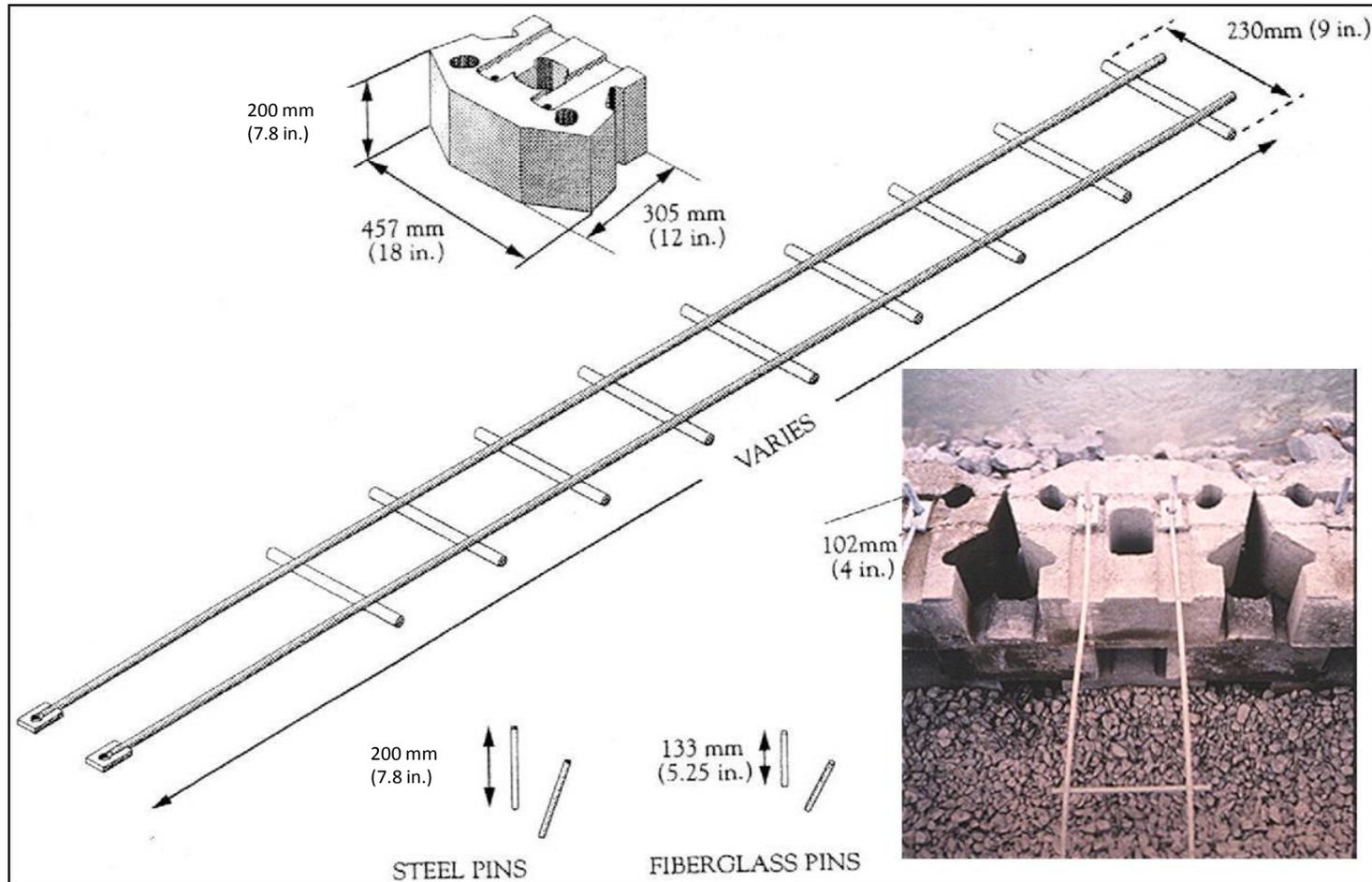
- High Density Polyethylene (HDPE) Geogrids
  - Behaves like plastic
  - Uses lower % of ultimate due to creep
  - Extruded and stretched structure
  - High dimensional stability
  - Newer versions have lower soil interaction values



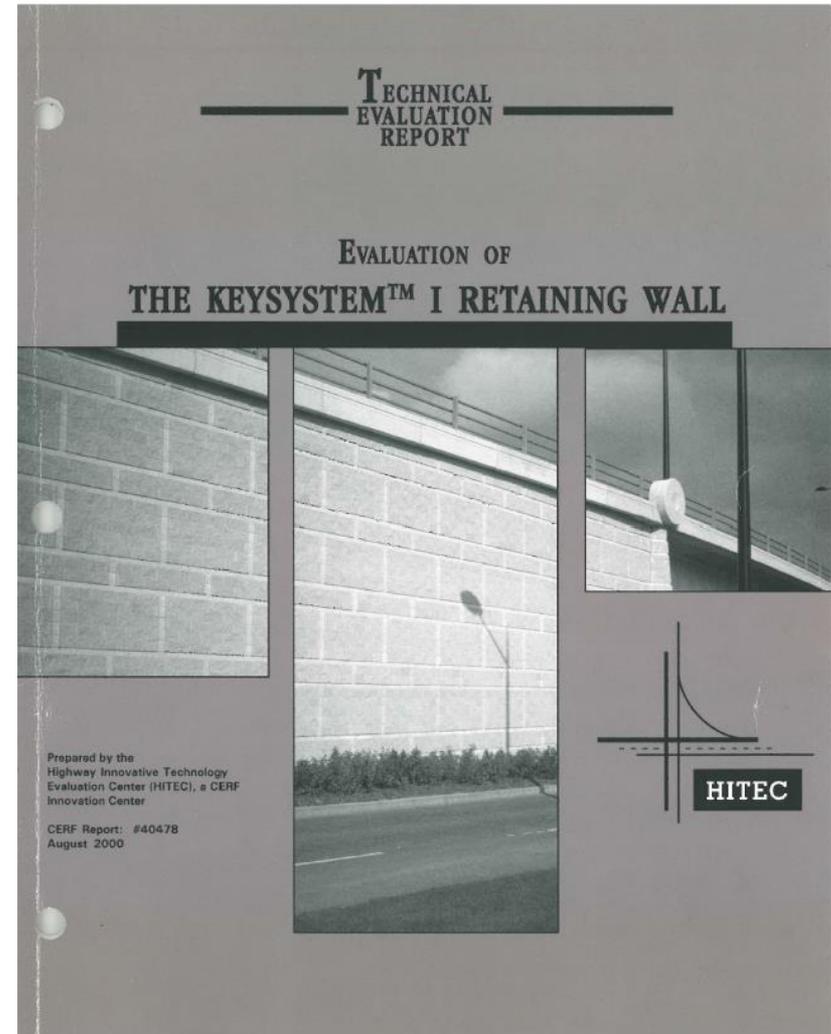
- Polyester (PET) Geogrids
  - High strength uniaxial polyester geogrid
  - Woven and coated with a PVC coating
  - Broad ultimate tensile strength range
  - Excellent durability properties



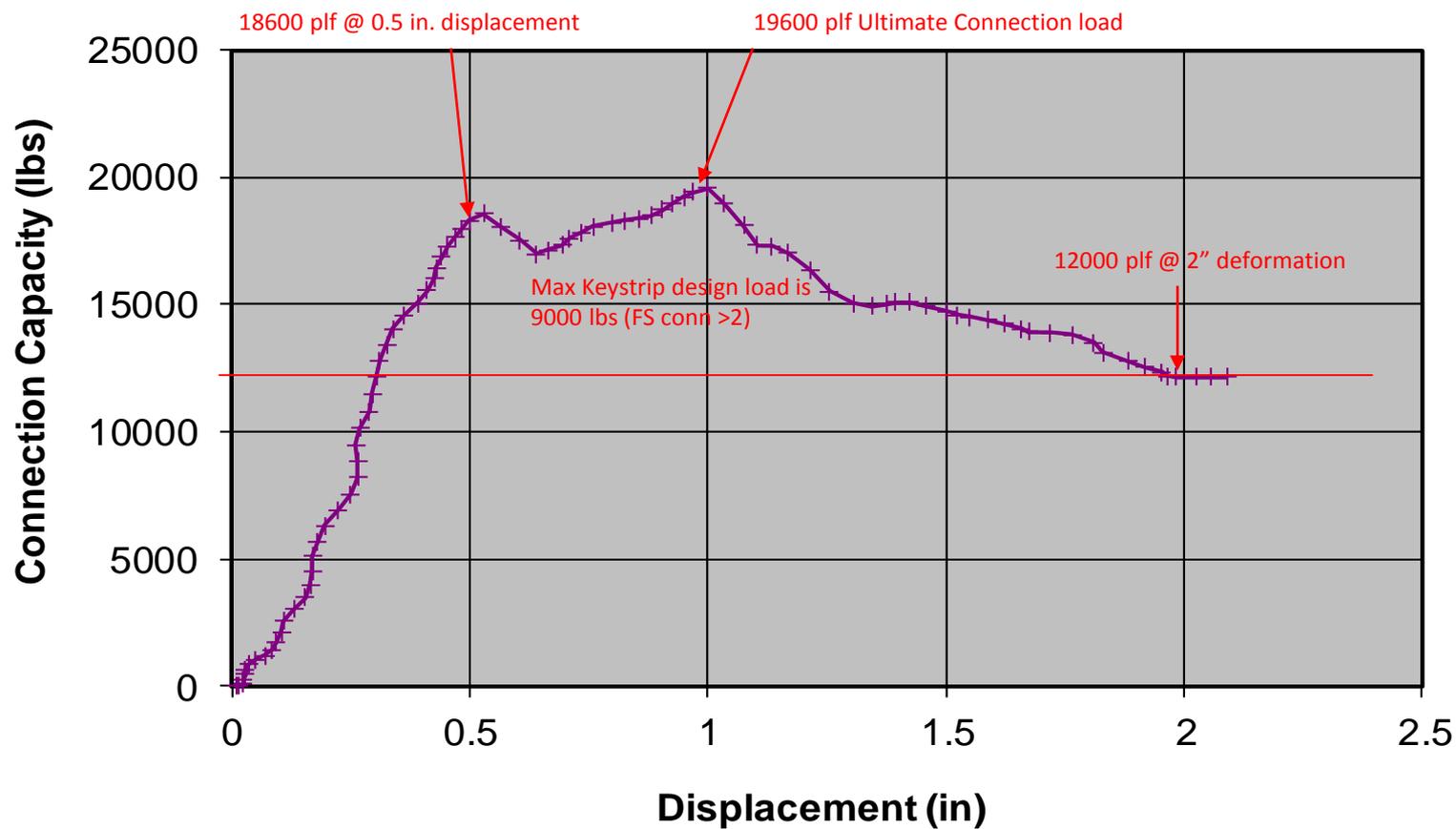
- KeySteel® Soil Reinforcement



- Keysystem I Unit
  - Compressive Strength
    - 4000 psi
  - 82 kN Ultimate reinforcement connection strength @ 15 mm Displ.



## KeySystem I Unit/KeyStrip Connection

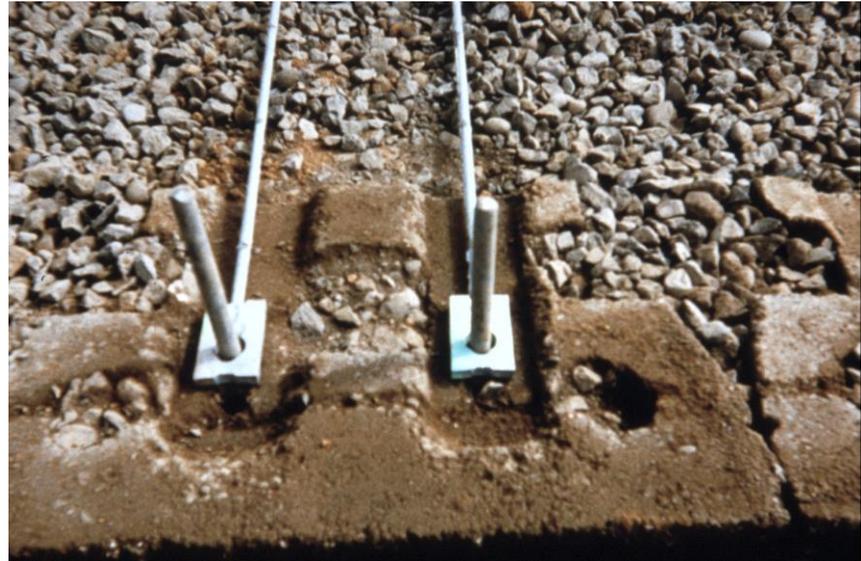


- KeyStrips

- 9mm, 10mm, 11mm Wire
- Crossbars at 300mm & 450mm
- Hot Dipped Galvanized
- 75 to 100 year design life
- Connection capacity limited to 42.4 kN, after factors of safety applied



- W24 – 14.3mm Steel Connection Pins (9/16" diameter)



- 12.7mm Fiberglass Alignment Pins



- Heavy Construction
  - Industrial walls
  - Heavy Highway
  - Railway Design
- Transportation
  - Bridge Abutment
  - Development Roadways
- Seismic
  - Better Seismic Performance under heavy loads than extensible geogrid



Waikato Expressway Construction

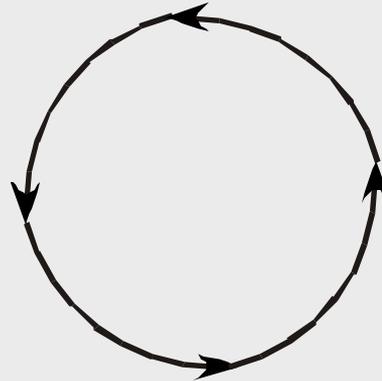


**WAIKATO EXPRESSWAY: CAMBRIDGE TO TAMAHERE  
FIRTH KEYSTONE KEYSTEEL**



**Construction**

**Soils**



**Design**

**Products**

Require attention to four items.

## Soil Types

- Granular Soils - Sand & Gravels
- Fine Grained Soils - Silt & Clays
- Other - Organic, Peat

## Preferred Soil Gradation

<u>Sieve Size</u>	<u>% Passing</u>
2 inch	100-75
3/4 inch	100-75
No. 40	0-60
No. 200	0-35
PI < 15	LL <40

## Design Properties

- $\gamma$ , Moist Unit Weight
- $\phi$ , Effective Shear Strength
- $c$ , Cohesive Strength

## Minimum Compaction Density

- 95% of Standard Proctor
- 92% of Modified Proctor
- Moisture +0%, - 3%

**The art of balancing driving and resisting forces.**

## External Stability

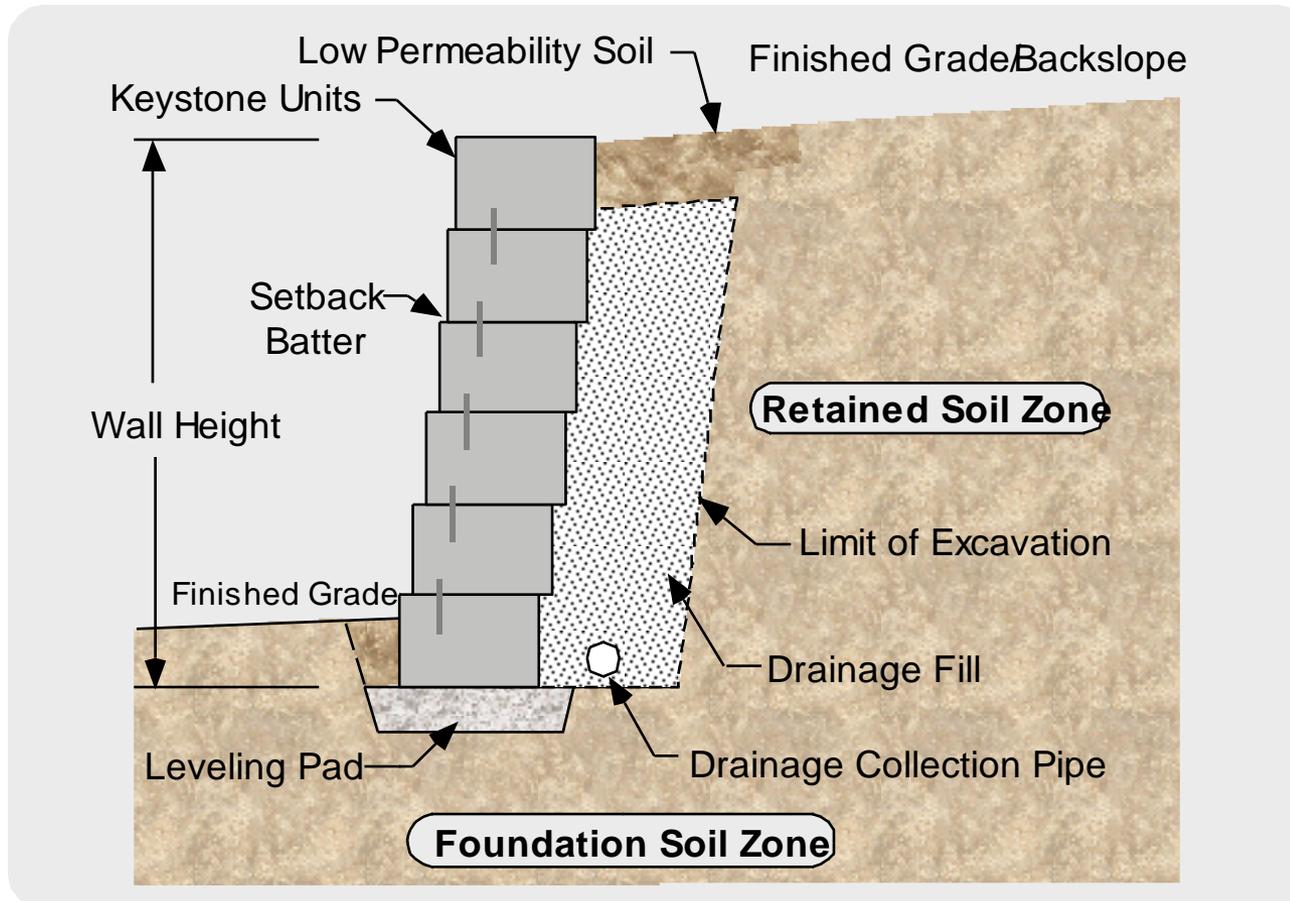
- Base Sliding
- Bearing Capacity & Settlement
- Overturning
- Global Stability

## Internal Stability

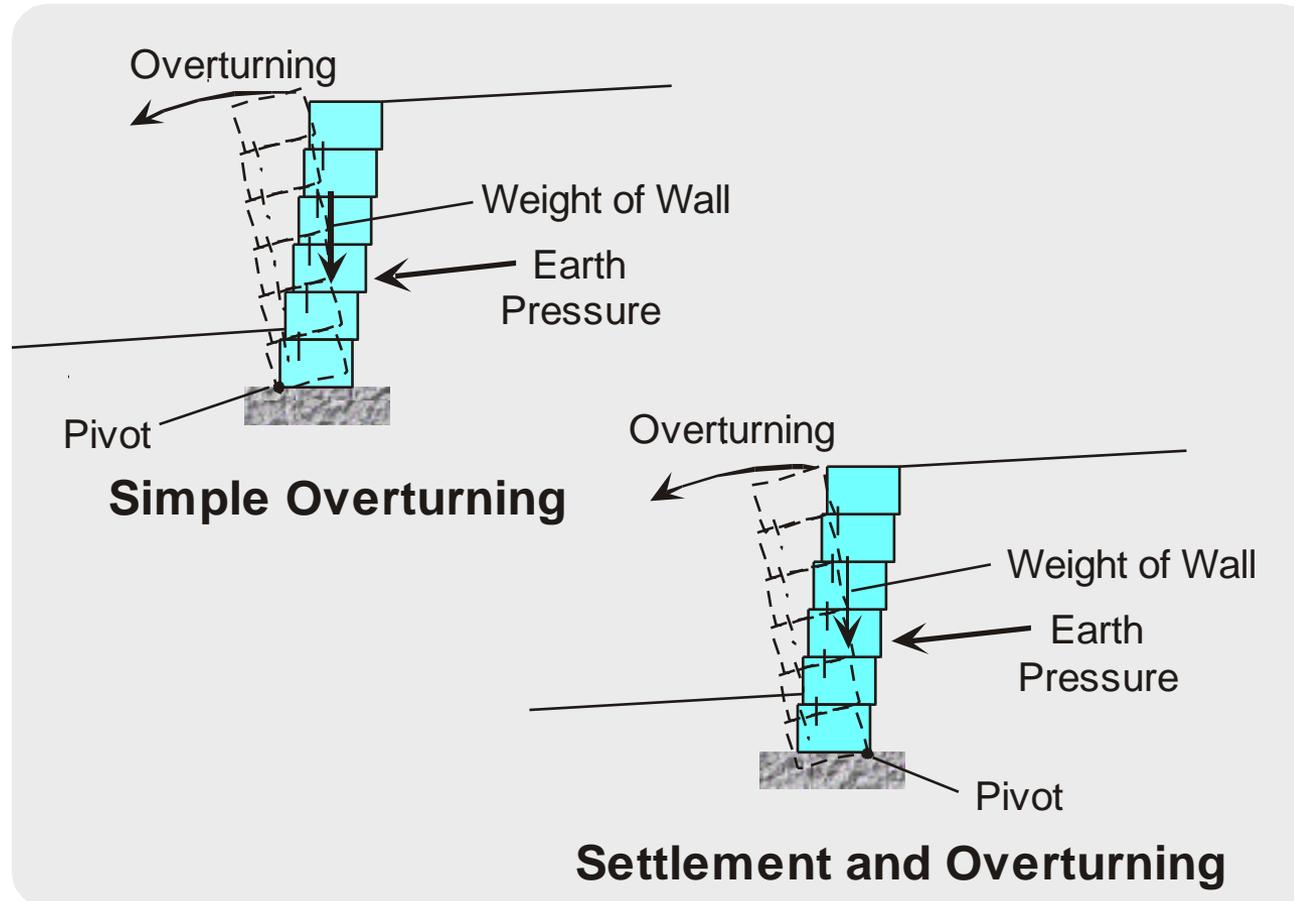
- Reinforcement Tension
- Pullout

## Facing Stability

- Connection
- Shear/Bending
- Overturning @ Top

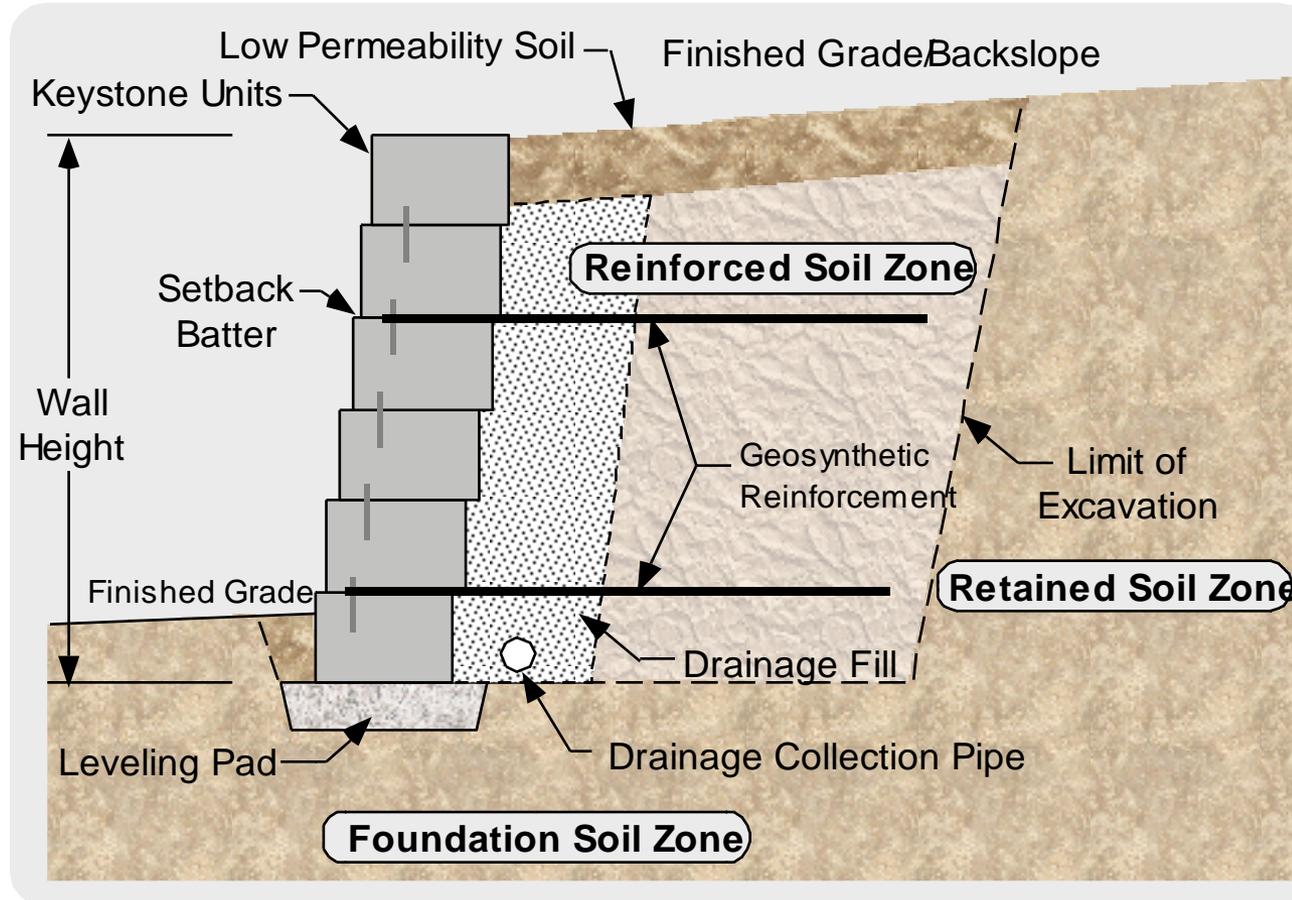


Gravity walls rely on their mass and batter to resist overturning.



Gravity walls typically fall over when built too tall for unit size

The creation of a reinforced mass or geocomposite, made of soil, geosynthetics or steel reinforcement, and concrete facing units, of sufficient size to resist the imposed forces.



Reinforced walls rely heavily on soil strength for the structure.

## Design Manual for SEGMENTAL RETAINING

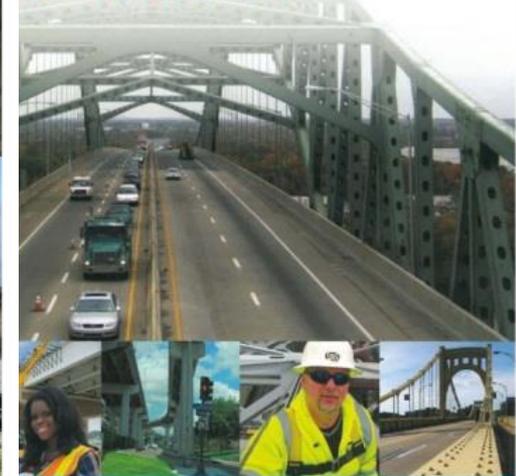
National Concrete Masonry Association  
3rd Edition

## Bridge manual (SP/M/022)

Third edition



## AASHTO BRIDGE DESIGN SPECIFICATIONS



Seventh Edition, 2014  
U.S. Customary Units

Part II: Sections 7—Index

- Rankine and Coulomb design using allowable stress design (ASD) and factor of safety (FS) methods.

$$\frac{R}{P} \geq FS$$

R = Resistance (stabilizing forces)

P = Load (destabilizing forces)

- AASHTO Load Resistance Factor Design (LRFD) and Capacity Demand Ratio (CDR)

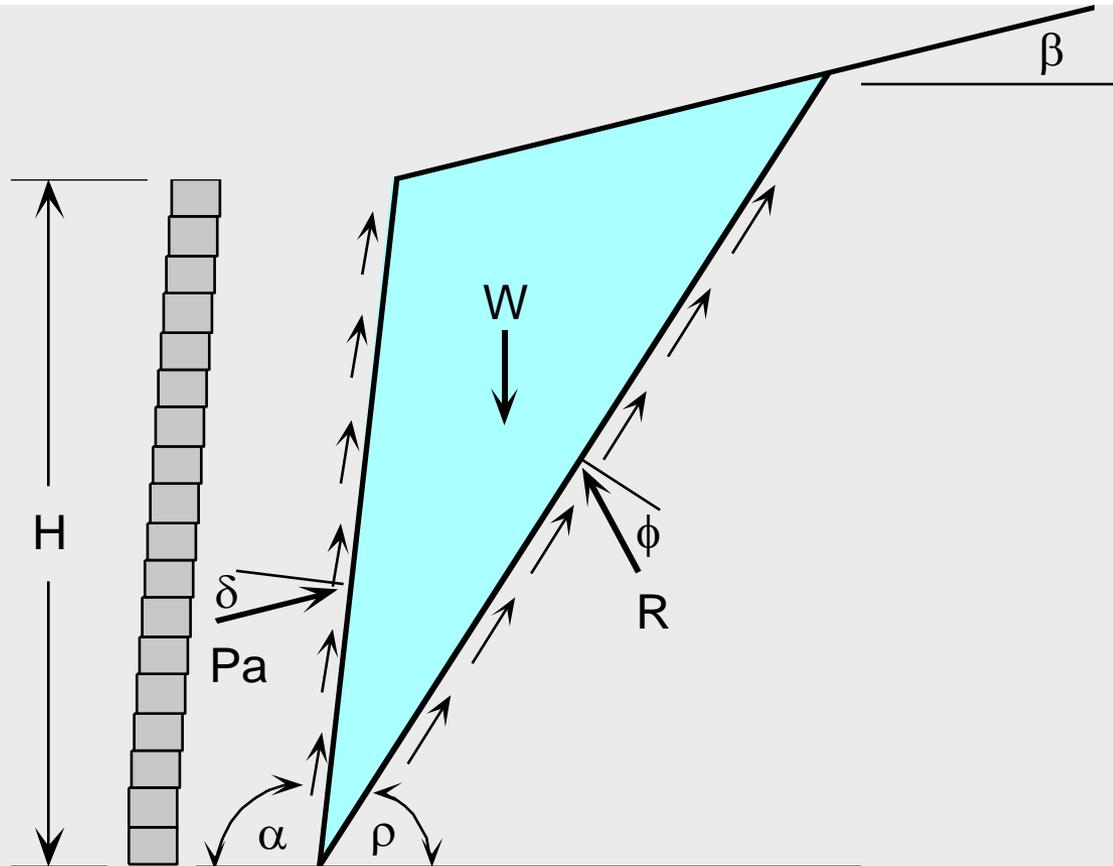
$$CDR = \frac{\phi R}{(\gamma_1 * P_1) + (\gamma_2 * P_2)} > 1.0$$

$\phi$  = resistance factor

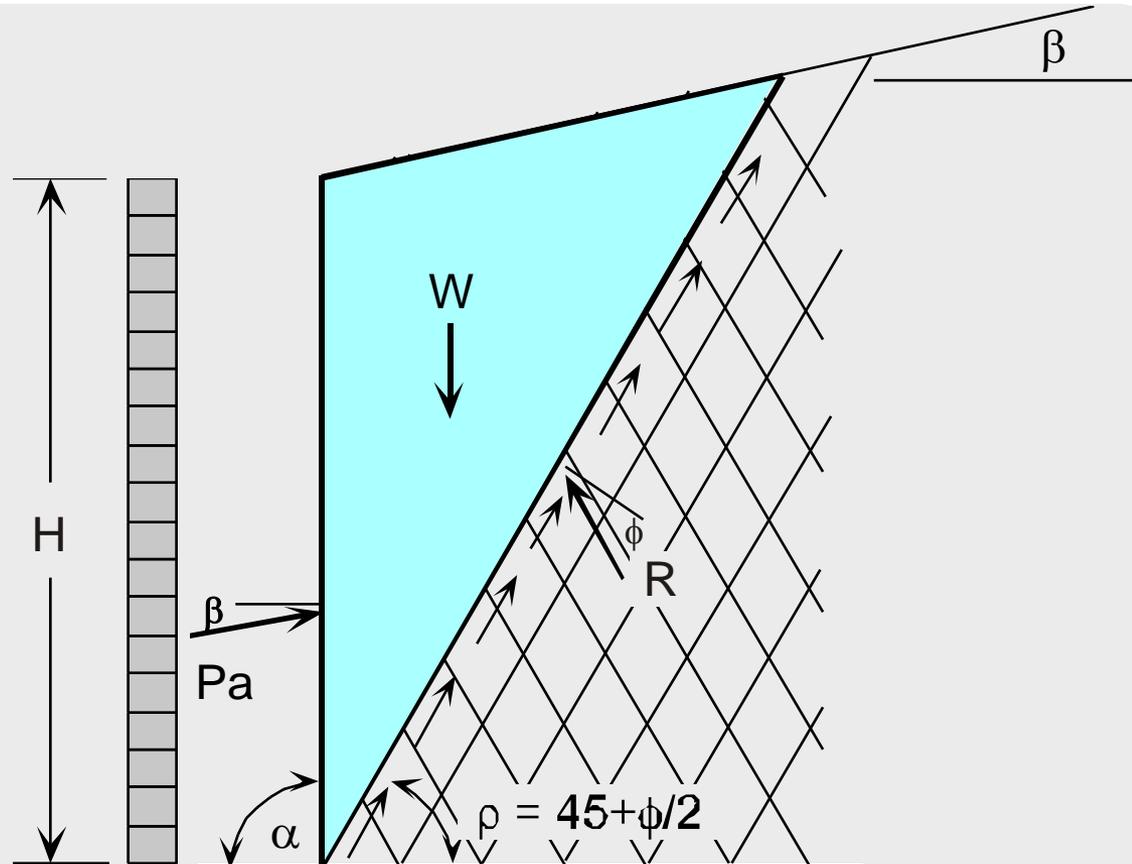
R = resistance (stabilizing forces)

$\gamma$  = load factor for a certain load type

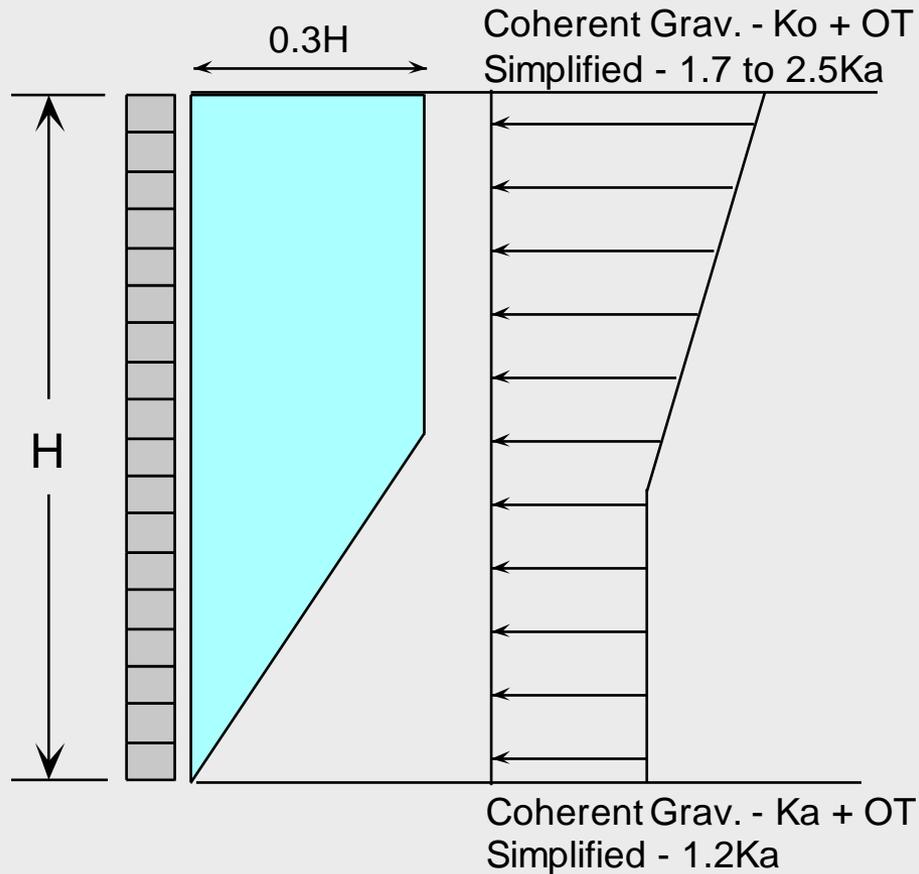
P = Load of a certain type (destabilizing force)



Coulomb (1776) Theory

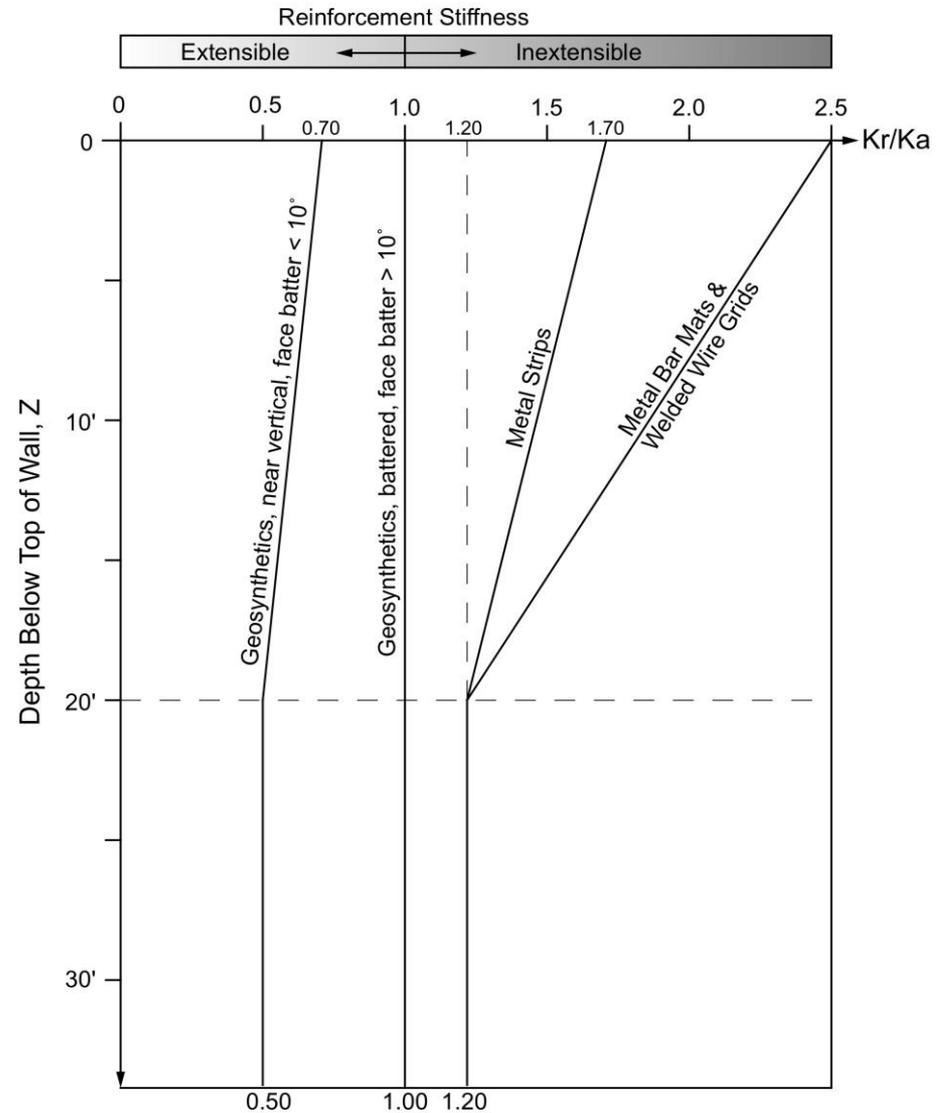


Rankine (1857) Theory



## Inextensible Theory

\*Unofficial Document  
 Currently in AASHTO sub-committee,  
 awaiting review and approval.



## Coulomb

$$P_a = 1/2 \gamma H^2 K_a$$

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

## Rankine

$$P_a = 1/2 \gamma H^2 K_a$$

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

### For level surcharge and infinite slope conditions

- **Coulomb** - Coulomb failure plane varies as a function of the wall geometry and friction angles for both the soils and the soil wall interface.

$$\tan(\rho - \varphi) = \frac{-\tan(\varphi - \beta) + \sqrt{\tan(\varphi - \beta) [\tan(\varphi - \beta) + \cot(\varphi + l)] \{1 + \tan(\delta - 1) \cot(\varphi + l)\}}}{1 + \tan(\delta - l) [\tan(\varphi - \beta) + \cot(\varphi - l)]}$$

where:

$\varphi$  = angle of internal friction

$l$  = batter of wall measured from vertical ( $\alpha - 90^\circ$ )

$\beta$  = slope angle above the wall

$\delta$  = angle of friction at back of wall

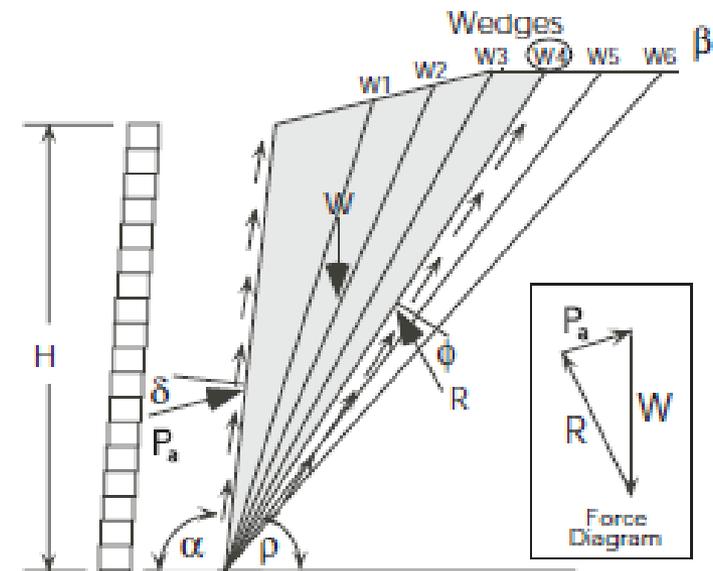
- **Rankine** – Where  $\rho$  is fixed and measured from horizontal under all design scenarios, which is only technically correct for level surcharge applications and minimal wall batter.

$$\rho = 45^\circ + \frac{\varphi}{2}$$

- In theory, the Rankine failure plane varies under backslope conditions. However, it is customary to fix the failure plane at the equation above in earth reinforcement design, thus best representing the curved failure surface and locus of maximum stress points for a reinforced soil mass

- Major difference between Rankine and Coulomb
  - Coulomb model and equations account for friction between the back of the wall and the soil mass as well as wall batter.
  - Rankine equations more conservatively assume no wall friction at the soil-wall interface and a vertical wall structure which greatly simplifies the mathematics of the problem.
  - The friction at the back of the wall face and at the back of the reinforced zone for external stability computations, provides an additional resisting force component that helps support the unstable wedge of soil.
  - Because of the additional resisting forces, lateral earth pressure in Coulomb is generally less than Rankine method.

- The limitations of closed form solutions, such as Coulomb and Rankine, is that only simple level and infinite sloping surcharges with uniform loadings can be analyzed.
- It is necessary to look at a “trial wedge” or “approximation method” when attempting to analyze broken back slope or other slope/load combinations.



- AASHTO and NCMA suggest an approximation method for broken-back slope conditions that defines equivalent design slopes for the external analysis.
  - However, the internal analysis is not well defined for unusual slopes and loading conditions and the designer is expected to use engineering judgement with the simplified methods.
- Keywall “Trial wedge” analysis used is consistent with the fundamental assumptions of the applicable coulomb and Rankine theories by setting  $\delta=\beta$ .
  - “Trial wedge” results match the equation solutions for the level and infinite slope conditions, but will determine the “correct” internal and external values for broken slope conditions and offset live and dead loads.

\*Note AASHTO LRFD use the AASHTO “Simplified” method for calculating internal pressure and the trial wedge for calculating external loading conditions. MSEW utilizes Trial Wedge.

- Which method do I use?
  - Each methodology is fundamentally different
  - Understand the design methodology for a particular project
  - Project Specific
    - Public / DOT – AASHTO / Rankine
    - Schools – Coulomb
    - Private – Rankine
  - The most important issue is that the designer understand and be comfortable with a design methodology, its limitations and follow the methodology in its entirety.

## Advantages / Disadvantages

### Coulomb

- Provides lowest calculated earth pressure by taking all beneficial components into account
  - Wall Batter
  - Wall Friction
- Reinforcement lengths significantly longer at the top of wall than the bottom due to flatter failure plane
- Reduced earth pressure may permit vertical spacing of reinforcement in lower walls that exceed the wall facing's stability during construction

### Rankine

- No assumption has to be made with regard to friction between the wall structure and retained soil mass.
- Simpler formula and failure plane definitions
- Due to the higher earth pressure coefficient, stronger reinforcement may be necessary at the bottom of wall.



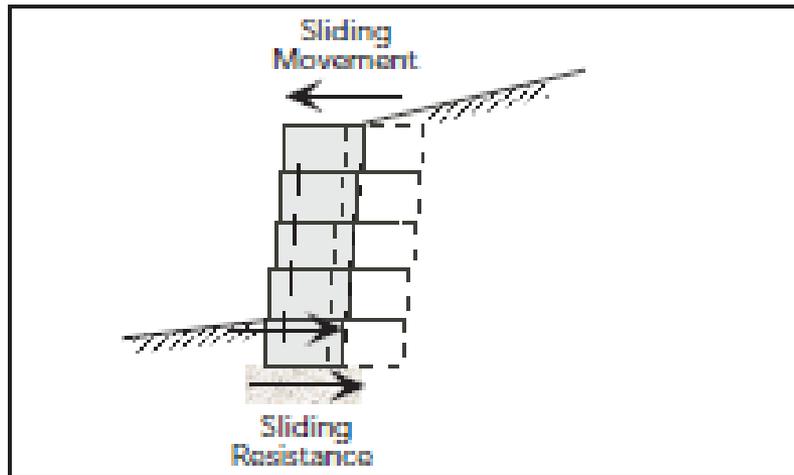


Figure 4:8 Gravity Wall Sliding

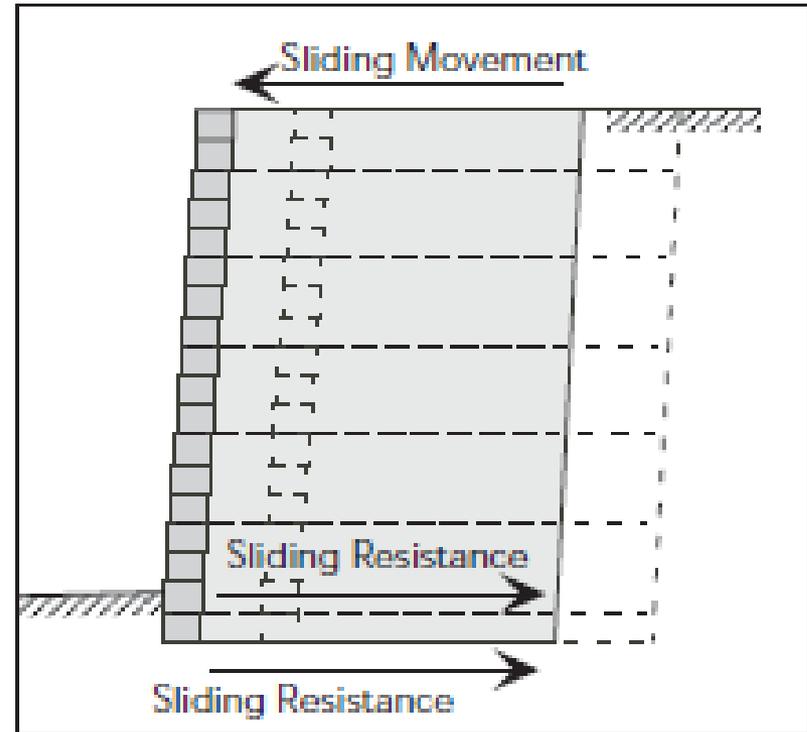


Figure 4:9 Reinforced Wall Sliding

## Coulomb

- Driving Forces

$$(P_a + P_q) \cos \delta$$

- Resisting Forces

$$(W_f + W_1 + W_2) \tan \phi$$

weaker soil (reinforced or foundation) as the resisting force

## Rankine - AASHTO

- Driving Forces

$$(P_a + P_q) \cos \beta$$

- Resisting Forces

$$(W_f + W_1 + W_2 + P_{av} + P_{qv}) \tan \phi$$

weaker soil (reinforced or foundation) as the resisting force

Note: Live load does not contribute to resisting forces.

\*Rankine includes vertical earth load components. NCMA 3<sup>rd</sup> Editions now permits the inclusion of vertical earth load components at the designers option.

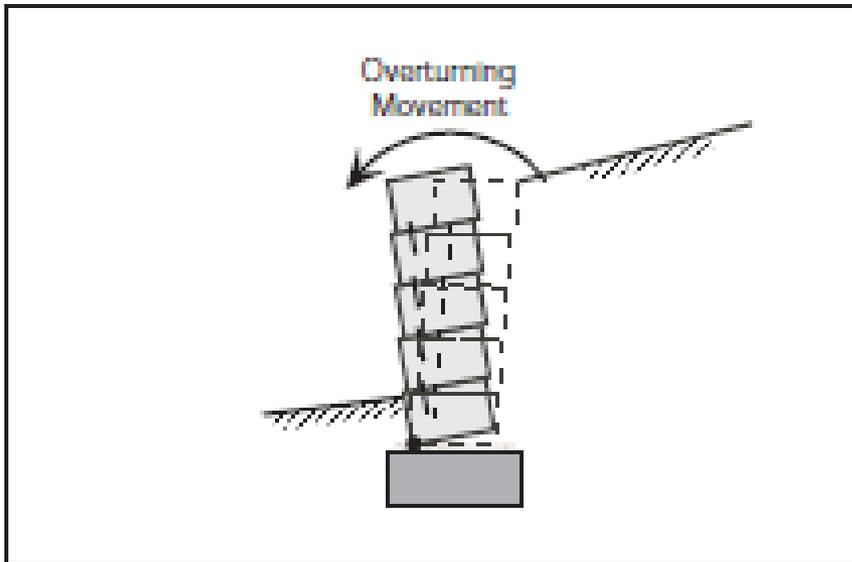


Figure 4:11 Gravity Wall Overturning

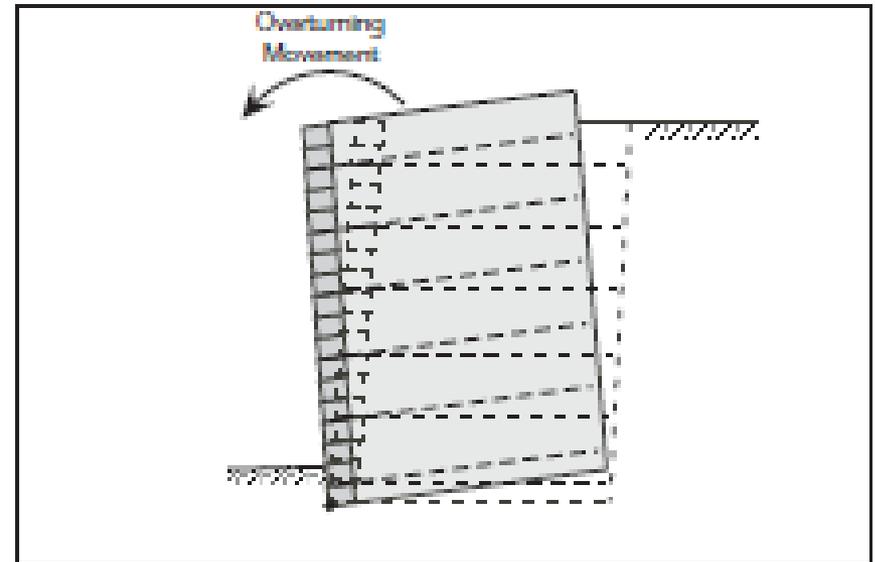


Figure 4:12 Reinforced Wall Overturning

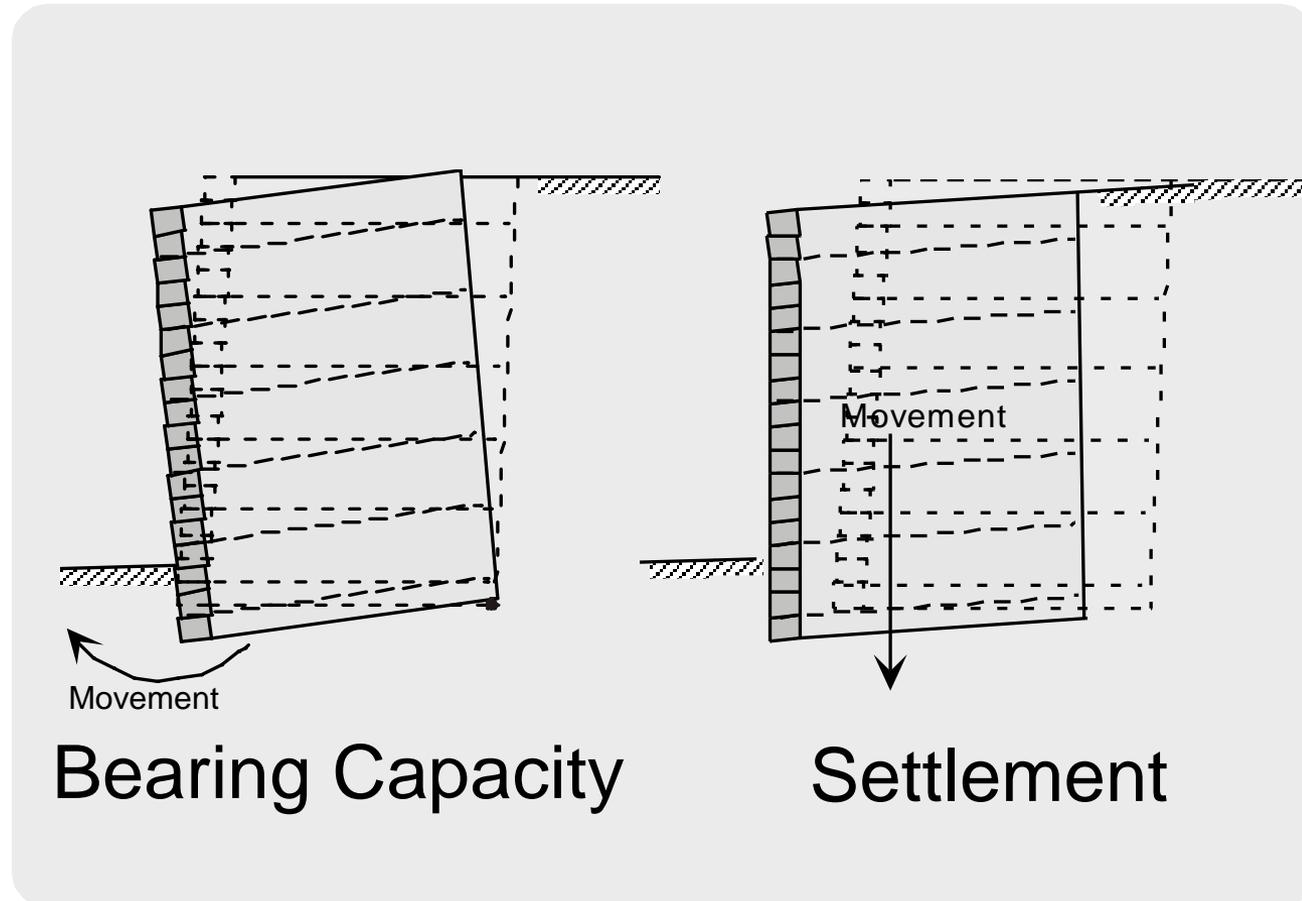
## Coulomb

- Driving Moments  
 $(P_a + P_q) \cos \delta$ , are the driving forces at their respective moment arms of  $H/3$  or  $HS/3$  and  $H/2$  or  $HS/2$  up from the toe
- Resisting Forces  
 $(W_f, W_1, W_2)$  at their respective moment arm from the toe to each center of gravity as the resisting moment.

## Rankine - AASHTO

- Driving Forces  
 $(P_a + P_q) \cos \beta$ , are the driving forces at their respective moment arms of  $H/3$  or  $HS/3$  and  $H/2$  or  $HS/2$  up from the toe
- Resisting Forces  
 $(W_f, W_1, W_2, P_{av}, P_{qv})$  at their respective moment arm from the toe to each center of gravity as the resisting moment

Note: Live load does not contribute to resisting forces. The live load surcharge is included as a driving force and not as a stabilizing force. Only permanent forces within the wall are included as stabilizing forces.

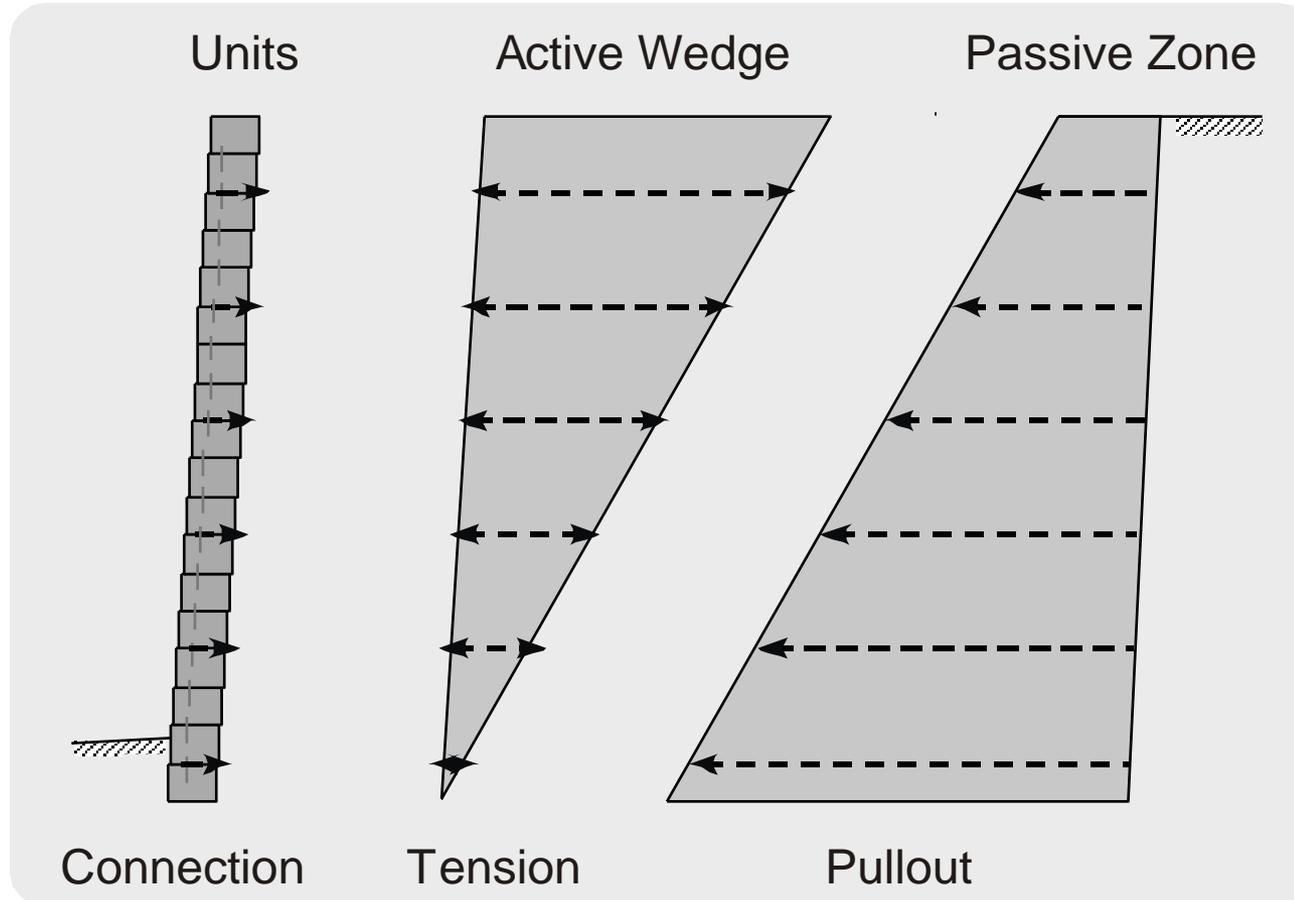


Bearing and Settlement  
are geotechnical issues

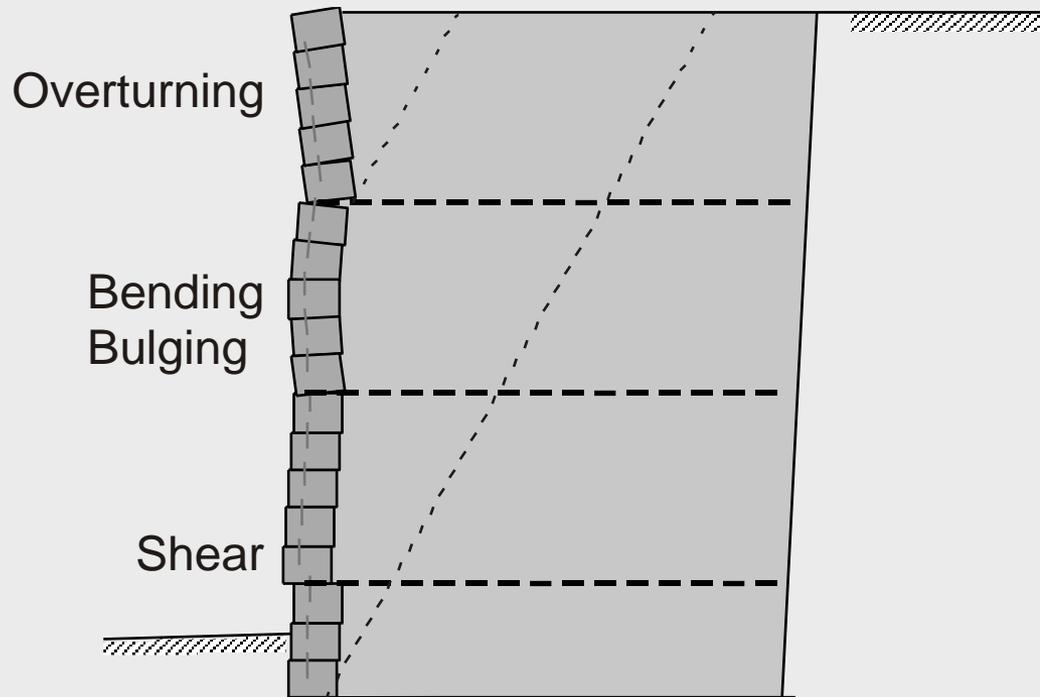
- No calculation differences
- Differences in the Factor of Safety
  - F.S.  $\geq$  2.0 NCMA
  - F.S.  $\geq$  2.5 AASHTO ASD
  - CDR  $>$  1.0 AASHTO LRFD (For Bearing be careful as a reduction factor of 0.65 is applied, which under ASD would be a 2.0 factor of safety.)
- Settlement, particularly differential settlement should be evaluated by a qualified engineer.
- Maximum allowable differential settlement for reinforced soil systems
  - 1% NCMA
  - ½% FHWA

Internal stability is the ability of the reinforced mass to maintain its structure and resist the applied loads without deforming or failing.

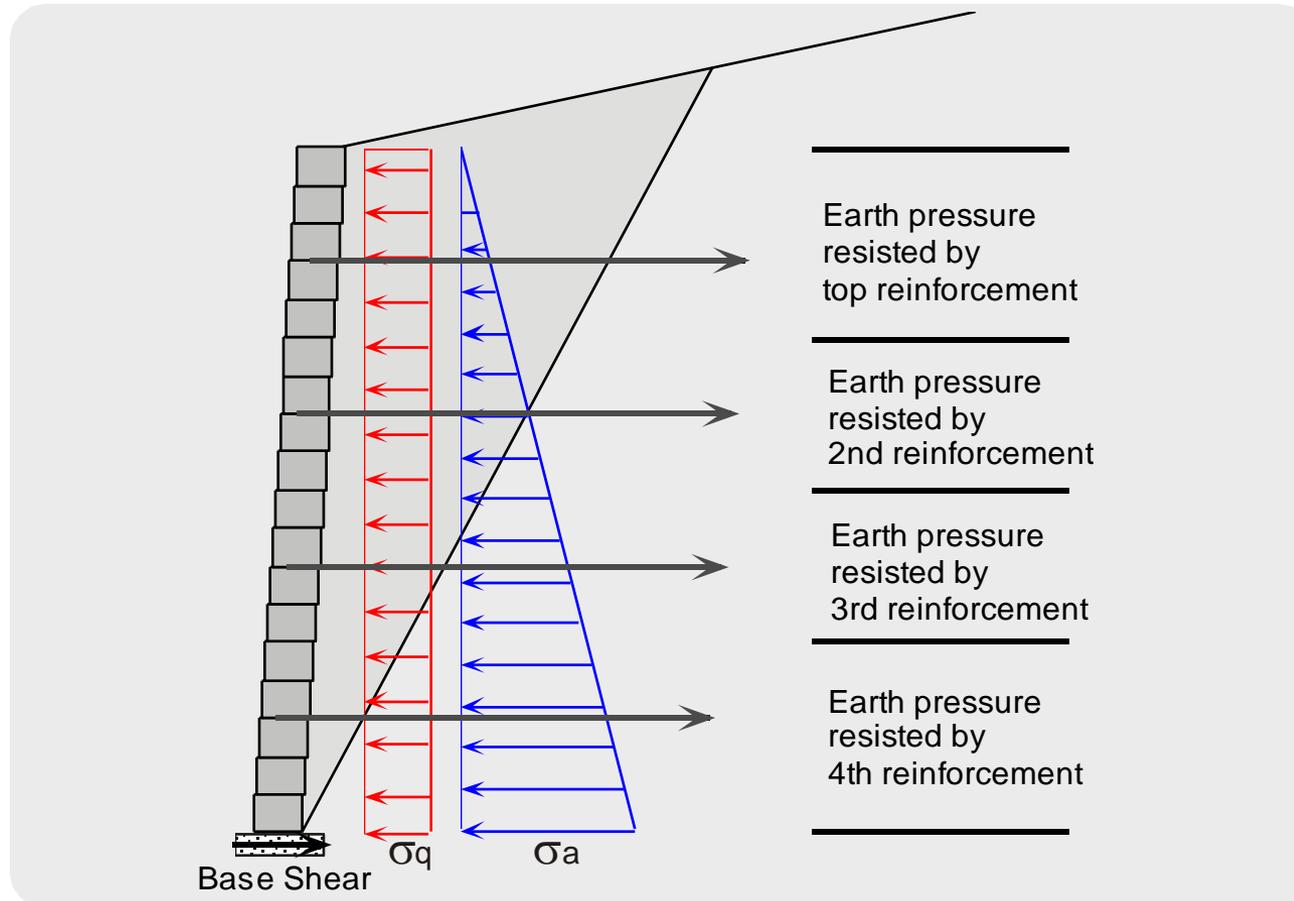
- In soil reinforced wall system, it is the tensile and pullout capacity of the reinforcing elements and inter-unit shear/connection capacity that holds the potential wedge of soil in place.
- The retained soil mass, or structure, is composed of the Keystone units at the face combined with reinforcing elements extending back beyond the Coulomb or Rankine failure plane.



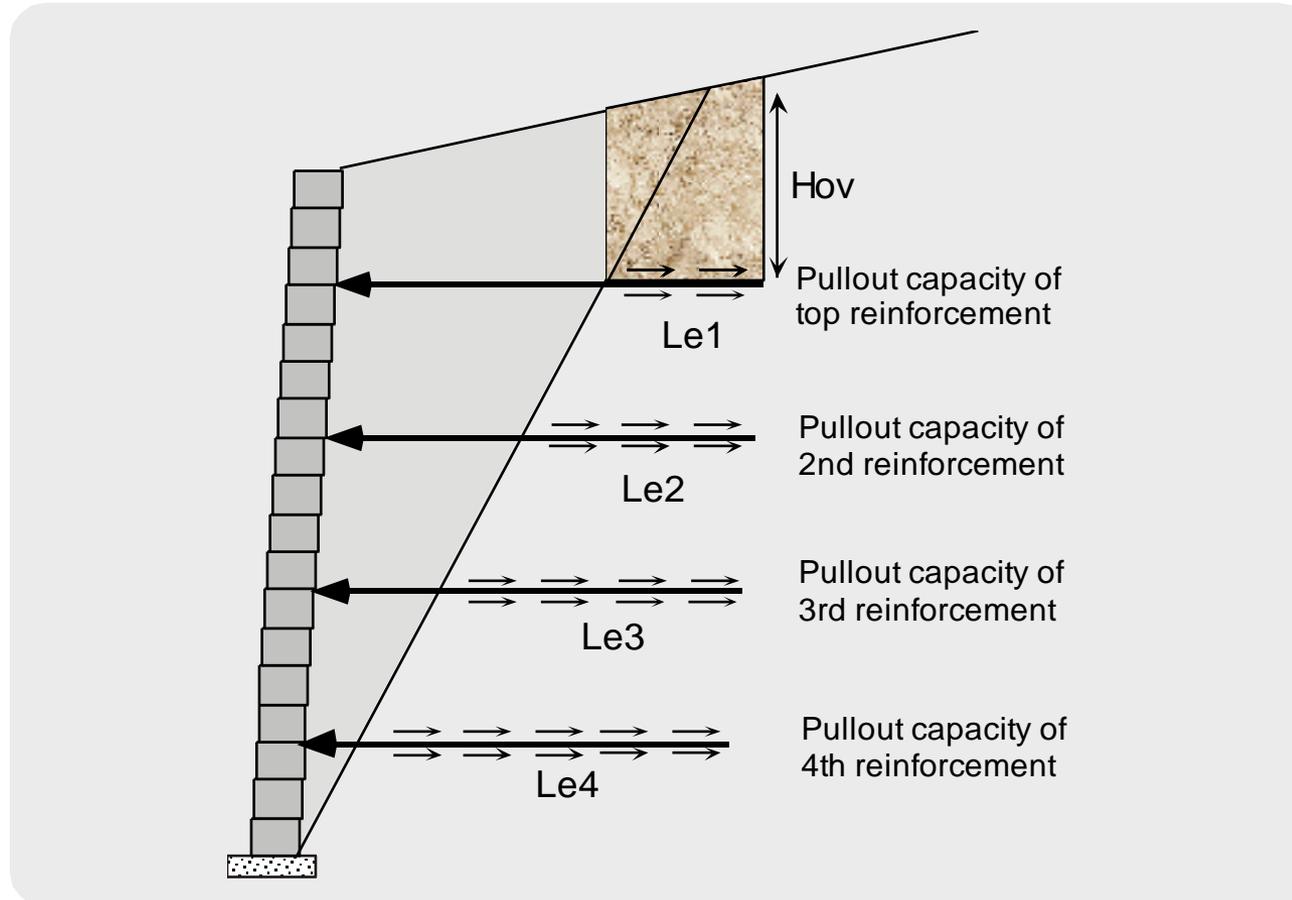
Internal Stability  
Tension, Connection and Pullout



Local Stability  
Overturning, Bending, Shear



$$\text{Geogrid Load} = (\sigma_q + \sigma_a) \cdot \text{Tributary Area} \leq T_{al}$$



$$\text{Pullout} = (2 Le)(\gamma H_{ov})(\text{Tan } \phi C_i)$$

$$\text{FS} = \text{Pullout} / \text{Geogrid Load}$$

There are no differences in the internal formulas between Coulomb and Rankine methods. The only difference is ASD vs. AASHTO LRFD and the application of load and resistance factors in the internal calculations.

## **The Elements of Internal Design are to ensure:**

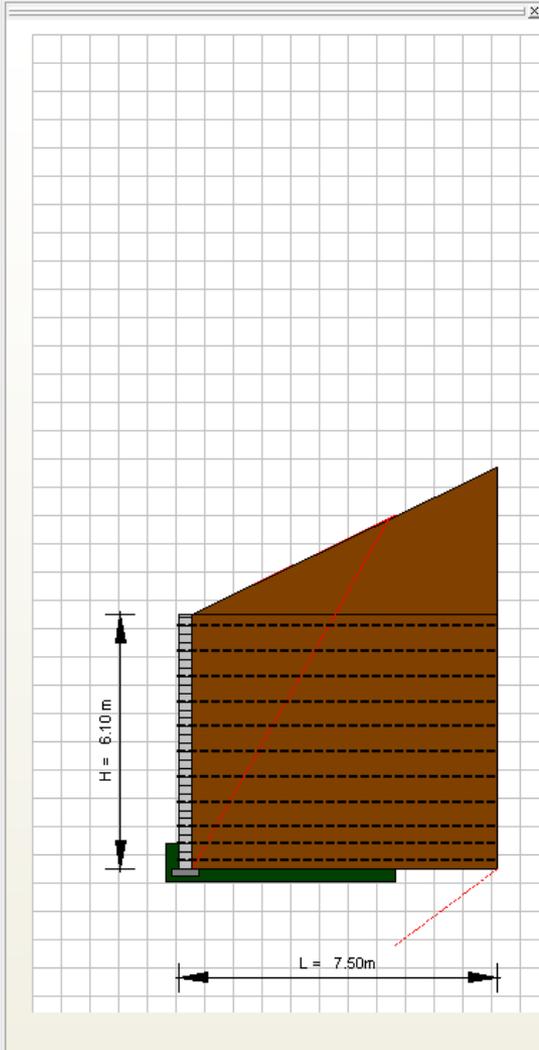
1. The tensile elements do not exceed their working stress or factored resistance limits.
2. The tensile element have adequate connection capacity to the Keystone units.
3. The tensile elements have adequate anchorage beyond the potential failure plane to hold the wedge of soil in place.
4. There is not a potential surface where the mass can shear internally.
5. The facing is stable against potential shear, bulging and overturning.

- Design Scenario
  - Wall Height 20'
  - Retained and Found. soil zone parameters  $28^\circ$ ,  $\gamma=19 \text{ kN/m}^3$
  - Reinforced Zone Foundation  $32^\circ$ ,  $\gamma=19.6 \text{ kN/m}^3$
  - Infinite backslope 2h:1v, ( $26^\circ$ )
  - Level Toe Slope
  - Near Vertical Wall batter
  - Compac II, Mirafi Geogrids

General | Geometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS

Case 1 / Rankine  
New Case

Navigation icons: left, right, up, down, zoom in (+), zoom out (-), and a list icon (L). Buttons: New Design, Print Long, Print Short.



Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
11	5.80	7.50	1.43	3XT	18.66	10.05	N/A	>10
10	5.20	7.50	4.30	3XT	18.66	12.35	N/A	>10
9	4.60	7.50	7.17	3XT	18.66	14.65	N/A	>10
8	4.00	7.50	10.04	3XT	18.66	16.95	N/A	>10
7	3.40	7.50	12.91	3XT	18.66	19.24	N/A	>10
6	2.80	7.50	15.78	3XT	18.66	20.42	N/A	>10
5	2.20	7.50	18.65	3XT	18.66	21.54	N/A	>10
4	1.60	7.50	21.52	7XT	31.46	24.45	N/A	>10
3	1.00	7.50	20.12	7XT	31.46	25.46	N/A	>10
2	0.60	7.50	17.53	7XT	31.46	26.14	N/A	>10
1	0.20	7.50	18.81	7XT	31.46	26.81	N/A	>10

FS Sliding	1.50	e	0.25/0.25	FS Bearing	6.53/6.53	FS Bending	3.34
FS OT	4.10	Bearing	196.00/196.00			FS Shear	2.91

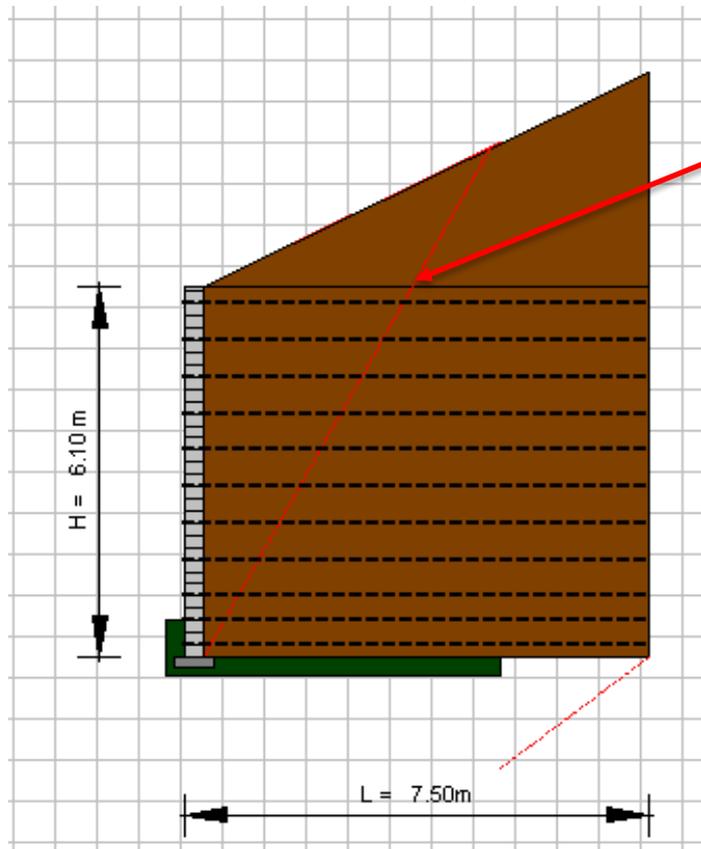
General | Geometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS | ICS

Case 2 / NCMA 3rd Ed  
New Case

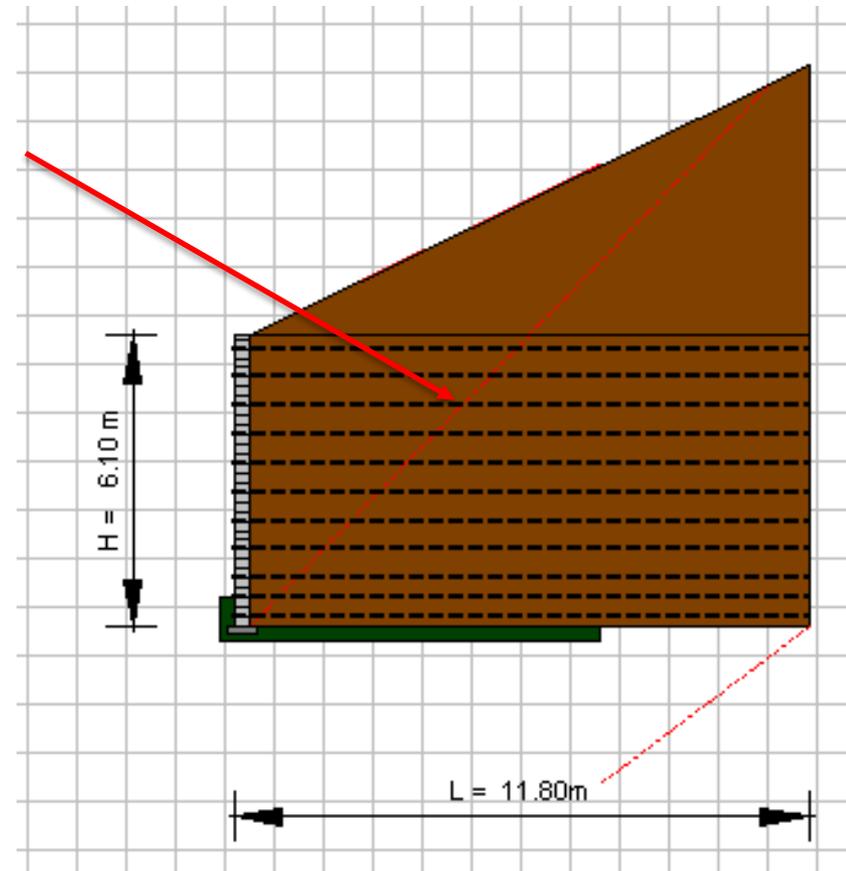
Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
11	5.80	11.80	1.47	3XT	18.66	10.05	N/A	>10
10	5.20	11.80	4.40	3XT	18.66	12.35	N/A	>10
9	4.60	11.80	7.34	3XT	18.66	14.65	N/A	>10
8	4.00	11.80	10.27	3XT	18.66	16.95	N/A	>10
7	3.40	11.80	13.20	3XT	18.66	19.24	N/A	>10
6	2.80	11.80	16.14	3XT	18.66	20.42	N/A	>10
5	2.20	11.80	19.07	7XT	31.46	23.44	N/A	>10
4	1.60	11.80	22.01	7XT	31.46	24.45	N/A	>10
3	1.00	11.80	20.58	7XT	31.46	25.46	N/A	>10
2	0.60	11.80	17.93	7XT	31.46	26.14	N/A	>10
1	0.20	11.80	19.23	7XT	31.46	26.81	N/A	>10

FS Sliding	1.50	e	0.75/0.75	FS Bearing	9.18/9.18	FS Bending	3.26
FS OT	4.73	Bearing	196.54/196.54			FS Shear	2.85

## Rankine



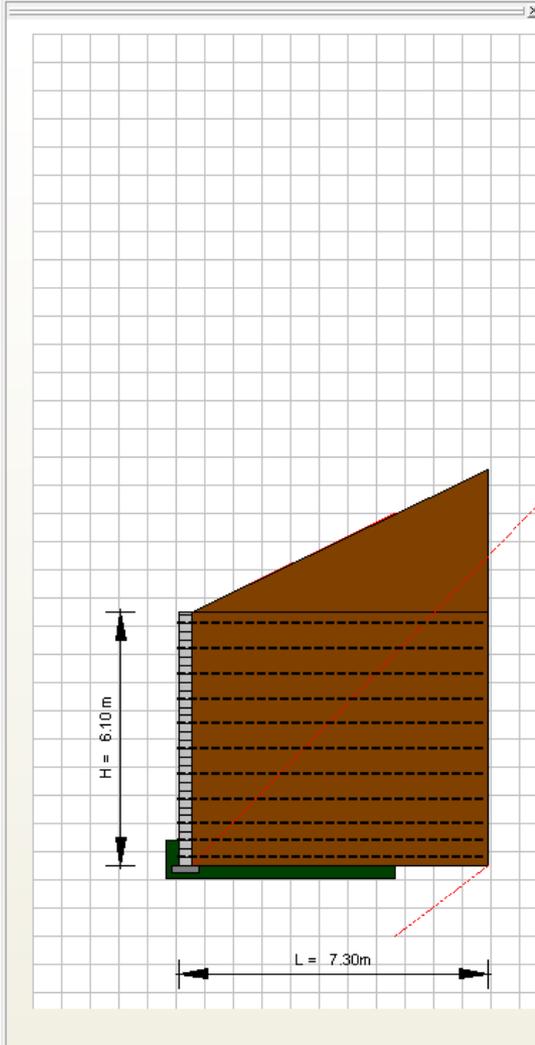
## Coulomb



General | Geometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS | ICS

Case 2 / NCMA 3rd Ed  
New Case

New Design | Print Long | Print Short



Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
11	5.80	7.30	1.47	3XT	18.66	10.05	N/A	>10
10	5.20	7.30	4.40	3XT	18.66	12.35	N/A	>10
9	4.60	7.30	7.34	3XT	18.66	14.65	N/A	>10
8	4.00	7.30	10.27	3XT	18.66	16.95	N/A	>10
7	3.40	7.30	13.20	3XT	18.66	19.24	N/A	>10
6	2.80	7.30	16.14	3XT	18.66	20.42	N/A	>10
5	2.20	7.30	19.07	7XT	31.46	23.44	N/A	>10
4	1.60	7.30	22.01	7XT	31.46	24.45	N/A	>10
3	1.00	7.30	20.58	7XT	31.46	25.46	N/A	>10
2	0.60	7.30	17.93	7XT	31.46	26.14	N/A	>10
1	0.20	7.30	19.23	7XT	31.46	26.81	N/A	>10

FS Sliding	1.52	e	0.43/0.43	FS Bearing	5.68/5.68	FS Bending	3.26
FS OT	3.91	Bearing	209.67/209.67			FS Shear	2.85

## Rankine

## Coulomb

General | Gometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS | Case 1 / Rankine New Case

General | Gometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS | ICS | Case 2 / NCMA 3rd Ed New Case

Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
10	5.60	5.70	1.99	3XT	18.66	10.82	N/A	>10
9	5.00	5.70	4.11	3XT	18.66	13.12	N/A	>10
8	4.40	5.70	6.35	3XT	18.66	15.41	N/A	>10
7	3.80	5.70	8.60	3XT	18.66	17.71	N/A	>10
6	3.20	5.70	10.84	3XT	18.66	19.67	N/A	>10
5	2.60	5.70	13.08	3XT	18.66	20.79	N/A	>10
4	2.00	5.70	15.32	3XT	18.66	21.91	N/A	>10
3	1.40	5.70	17.57	3XT	18.66	23.04	N/A	>10
2	0.80	5.70	19.81	7XT	31.46	25.80	N/A	>10
1	0.20	5.70	18.22	7XT	31.46	26.81	N/A	>10

Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
10	5.60	8.65	2.15	3XT	18.66	10.82	N/A	>10
9	5.00	8.65	4.44	3XT	18.66	13.12	N/A	>10
8	4.40	8.65	6.86	3XT	18.66	15.41	N/A	>10
7	3.80	8.65	9.27	3XT	18.66	17.71	N/A	>10
6	3.20	8.65	11.69	3XT	18.66	19.67	N/A	>10
5	2.60	8.65	14.11	3XT	18.66	20.79	N/A	>10
4	2.00	8.65	16.53	3XT	18.66	21.91	N/A	>10
3	1.40	8.65	18.95	7XT	31.46	24.79	N/A	>10
2	0.80	8.65	21.37	7XT	31.46	25.80	N/A	>10
1	0.20	8.65	19.66	7XT	31.46	26.81	N/A	>10

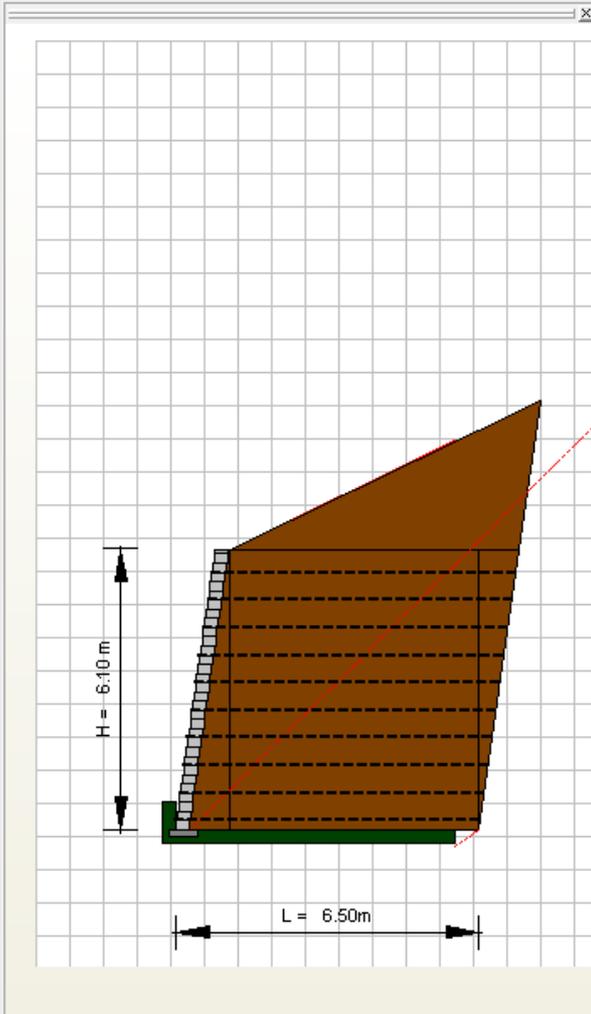
FS Sliding	1.51	e	0.00/0.00	FS Bearing	6.37/6.37	FS Bending	3.45
FS OT	3.76	Bearing	168.33/168.33	FS Shear	3.45	FS OT	4.30

FS Sliding	1.50	e	0.26/0.26	FS Bearing	8.61/8.61	FS Bending	3.20
FS OT	4.30	Bearing	169.71/169.71	FS Shear	3.19	FS OT	4.30

General | Geometry | Soils | Factors of Safety | Reinforcement | Seismic | RESULTS | ICS

Case 2 / NCMA 3rd Ed  
New Case

New Design | Print Long | Print Short



Layer	Height	Length	Tension	Reinforcing	Tal	Tcl	Tsc	FS PO
10	5.60	6.50	2.15	3XT	18.66	10.82	N/A	>10
9	5.00	6.50	4.44	3XT	18.66	13.12	N/A	>10
8	4.40	6.50	6.86	3XT	18.66	15.41	N/A	>10
7	3.80	6.50	9.27	3XT	18.66	17.71	N/A	>10
6	3.20	6.50	11.69	3XT	18.66	19.67	N/A	>10
5	2.60	6.50	14.11	3XT	18.66	20.79	N/A	>10
4	2.00	6.50	16.53	3XT	18.66	21.91	N/A	>10
3	1.40	6.50	18.95	7XT	31.46	24.79	N/A	>10
2	0.80	6.50	21.37	7XT	31.46	25.80	N/A	>10
1	0.20	6.50	19.66	7XT	31.46	26.81	N/A	>10

FS Sliding 1.52  
FS OT 3.98

e Bearing 0.00/0.00  
170.74/170.74

FS Bearing 7.03/7.03

FS Bending 3.20  
FS Shear 3.19



**WAIKATO EXPRESSWAY: CAMBRIDGE TO TAMAHERE  
FIRTH KEYSTONE KEYSTEEL**

"While all investigators have concluded that the dynamic lateral pressures developed during earthquakes exceed the static pressures on earth retaining structures, a survey of a number of engineering companies highway departments and port authorities in California shows that .... it is general practice to make no special allowance for increased lateral pressures on retaining walls .... due to earthquake effects. This also appears to be the case in many other countries."

"It should be noted that the factor of safety provided in the design of walls for static pressures may be adequate to prevent damage or detrimental movements during many earthquakes. (...) Thus where backfill and foundation soils remain stable, it is only in areas where very strong ground motions might be expected, for walls with sloping backfills or heavy surcharge pressures and for structures which are very sensitive to wall movements, that special seismic design provisions for lateral pressure effects may be necessary."

- Coulomb
- Rankine
- Mononobe-Okabe

## Coulomb

$$P_a = 1/2 \gamma H^2 K_a$$

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

## Rankine

$$P_a = 1/2 \gamma H^2 K_a$$

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

Note: Backslope can not be greater than phi angle

$$E_{AE} = \frac{1}{2} \gamma H^2 (1 - k_v) K_{AE}$$

where the seismic active pressure coefficient  $K_{AE}$  is

$$K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos \theta \cos^2 \beta \cos(\delta + \beta + \theta)} \times \left[ 1 + \sqrt{\frac{\sin(\phi + \delta) \sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta) \cos(i - \beta)}} \right]^{-2}$$

and where

$\gamma$  = unit weight of soil (kcf)

$H$  = height of soil face (ft.)

$\phi$  = angle of friction of soil ( $^\circ$ )

$\theta$  =  $\arctan(k_h / (1 - k_v))$  ( $^\circ$ )

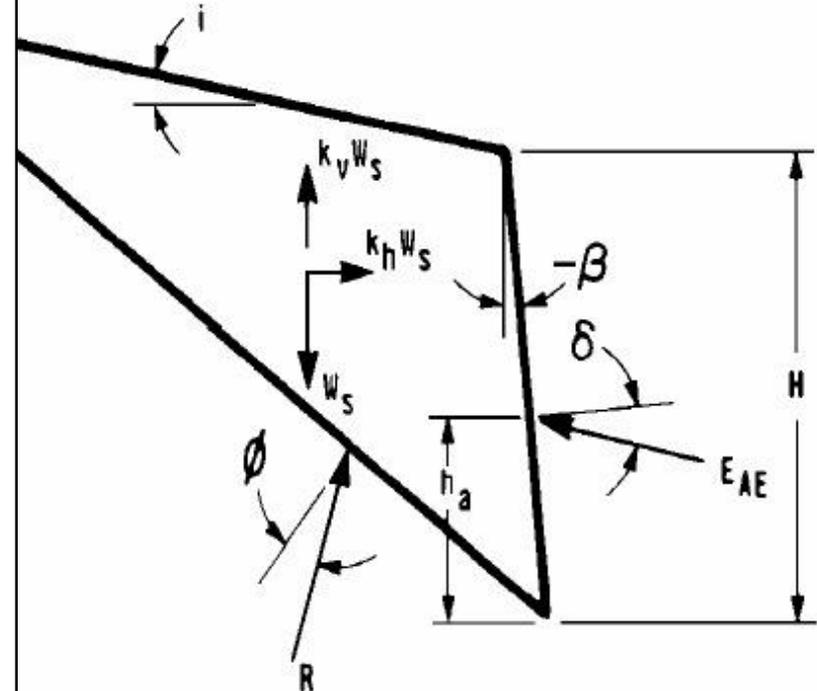
$\delta$  = angle of friction between soil and abutment ( $^\circ$ )

$k_h$  = horizontal acceleration coefficient (dim.)

$k_v$  = vertical acceleration coefficient (dim.)

$i$  = backfill slope angle ( $^\circ$ )

$\beta$  = slope of wall to the vertical, negative as shown ( $^\circ$ )



- Backslope MUST be  $< \phi$  of retained zone
- Let's look at the critical portion of the equation

$$\sin(\varphi - \theta - i)$$

$\varphi$  = angle of friction of soil

$$\theta = \text{arc tan}\left(\frac{Kh}{1-Kv}\right)$$

$i$  = backfill slope angle.

- Example

$\varphi$  = angle of friction of soil =  $28^\circ$

$\theta = \arctan\left(\frac{K_h}{1-K_v}\right)$  where,  $K_h=0.20g$ ,  $K_v=0$

$\theta = 11.3^\circ$

$i$  = backfill slope angle = 3h:1v =  $18^\circ$

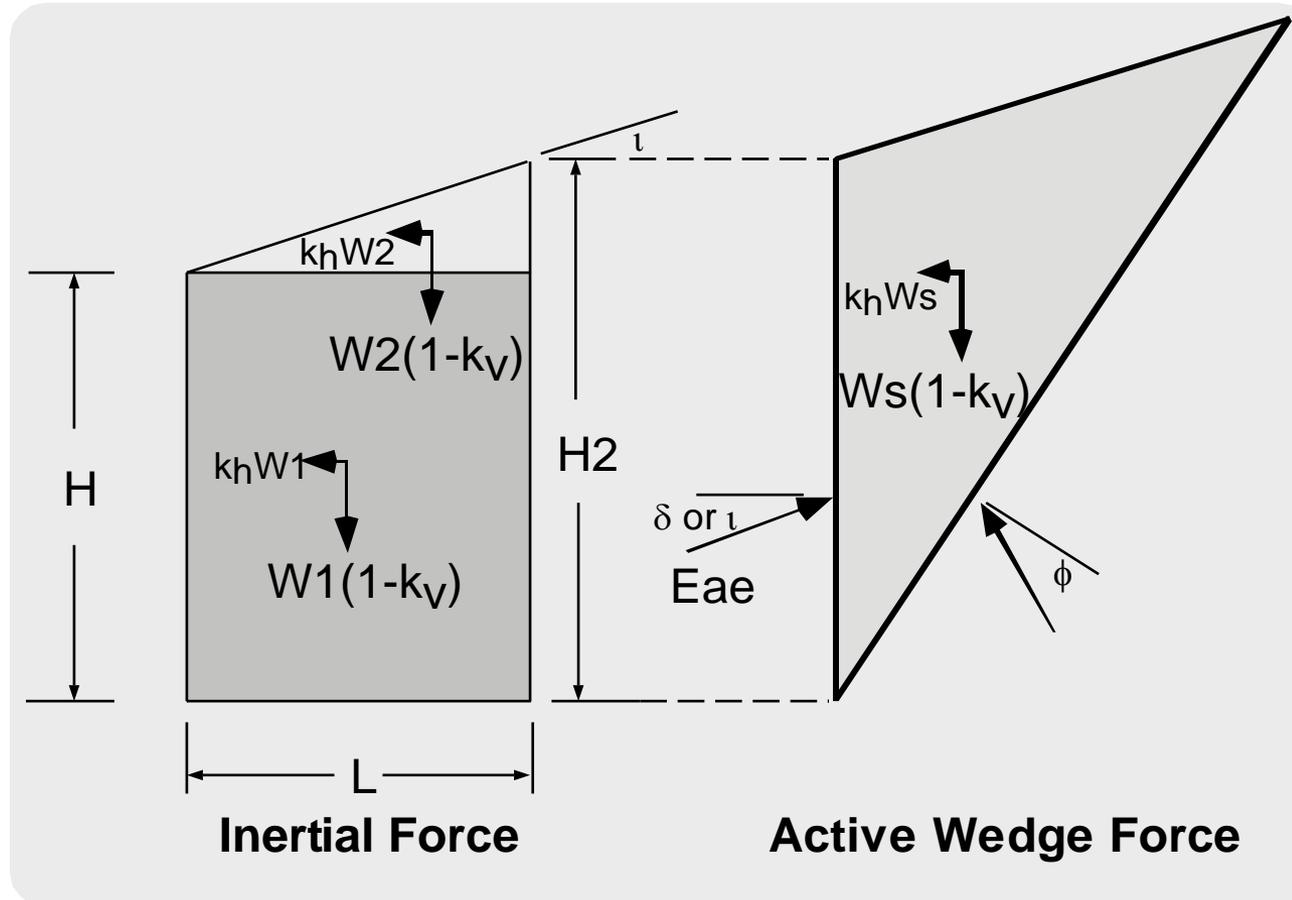
$\sin(28 - 11.3 - 18) = \sin(-1.3) = -0.023$  / Neg. Value

Mononobe Equation doesn't solve

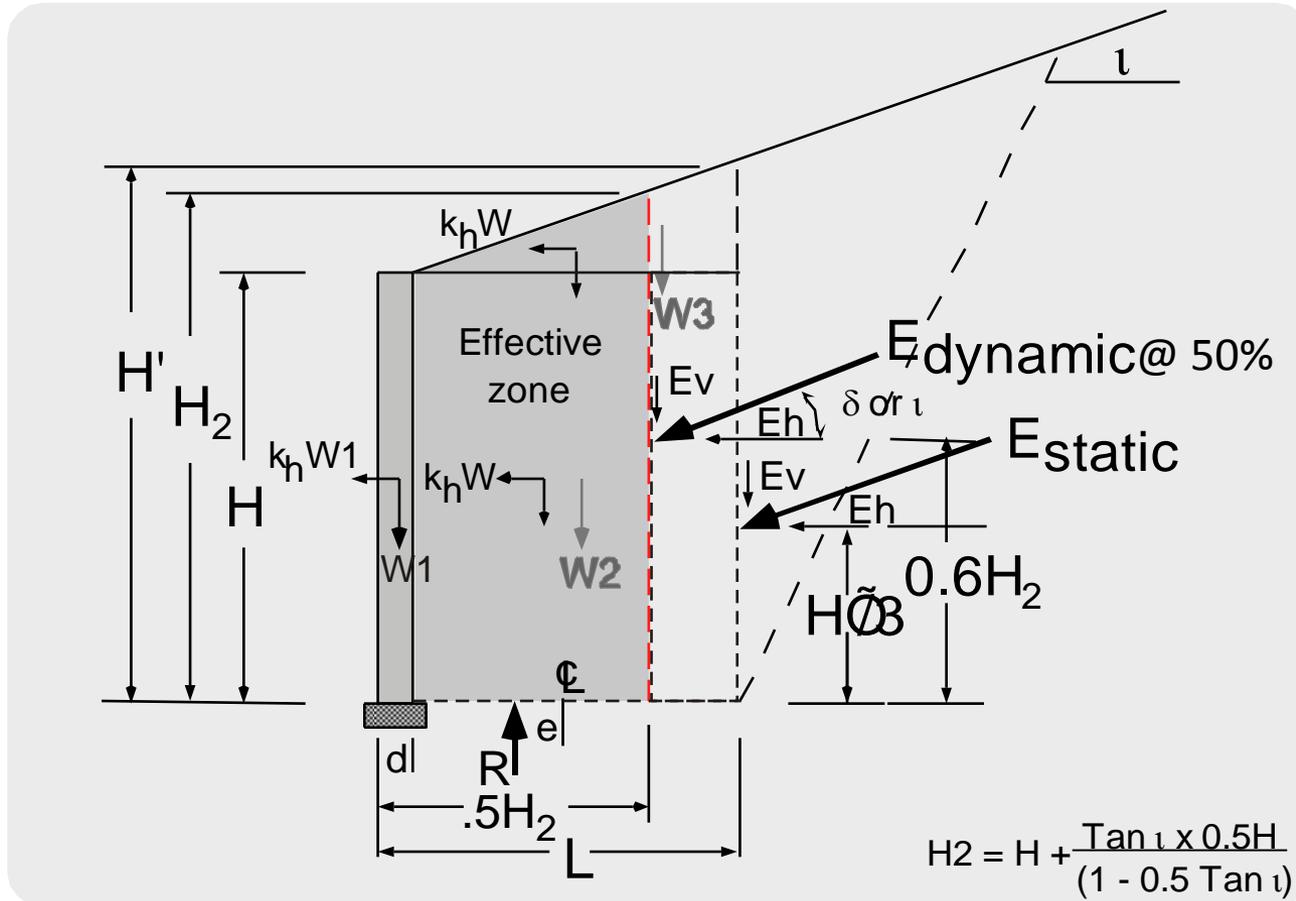
- How are we working around this?
- When  $\sin(\varphi - \theta - i)$  is negative this portion of the equation is often set to 0
- Other forms of analysis is important
  - Slope stability programs using cohesion
  - Finite element analysis
  - Displacement Method?
- The problem is that we are trying to solve something that we don't completely understand or are attempting to put 2-D numbers to a 3-D solutions.

How do we use M-O -Coulomb seismic analysis if Rankine earth pressure is the prevailing theory for MSE wall design?

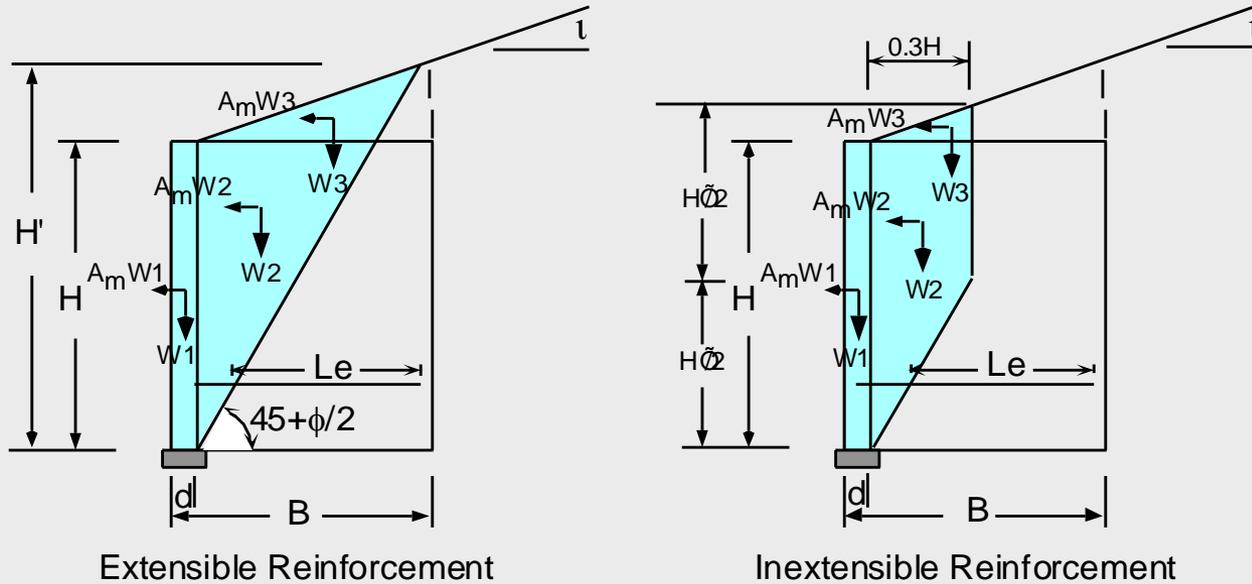
Coulomb = Rankine if delta angle is set equal to backslope angle and batter is set equal to zero.



Neglecting inertial forces  
is a common mistake



$$K_{dynamic} = K_{ae} - K_a$$



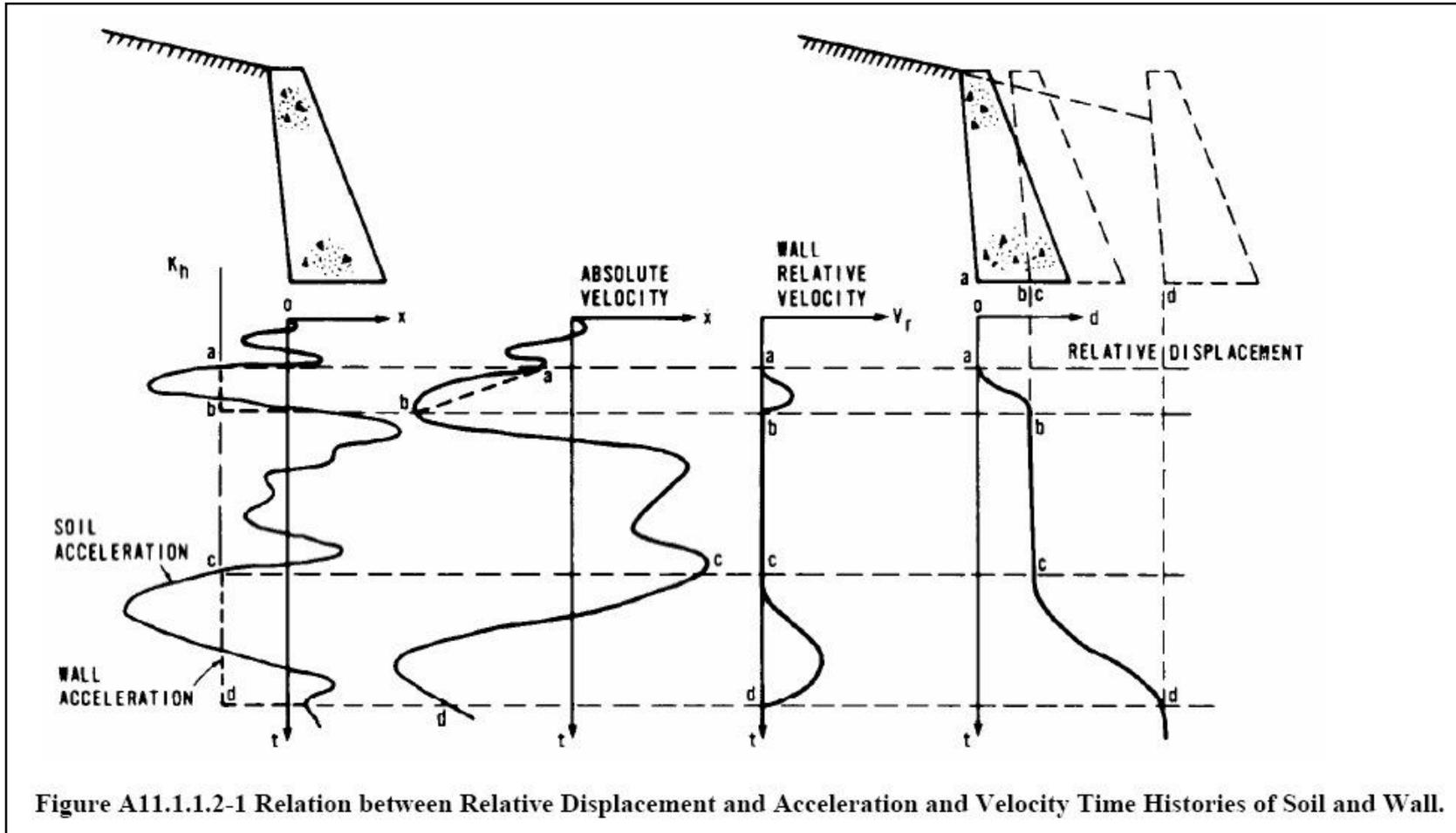
$$\text{Add'l Load/reinf} = (P_i) \frac{\text{Le of reinforcement level}}{\text{Sum of Le for all reinforcement levels}}$$

$$P_i = A_m (W_1 + W_2 + W_3)$$

*The Mononobe-Okabe equation for earth pressure is still used widely for design, although actual conditions during earthquake shaking of retaining structures are quite different from those assumed in developing the equation...*

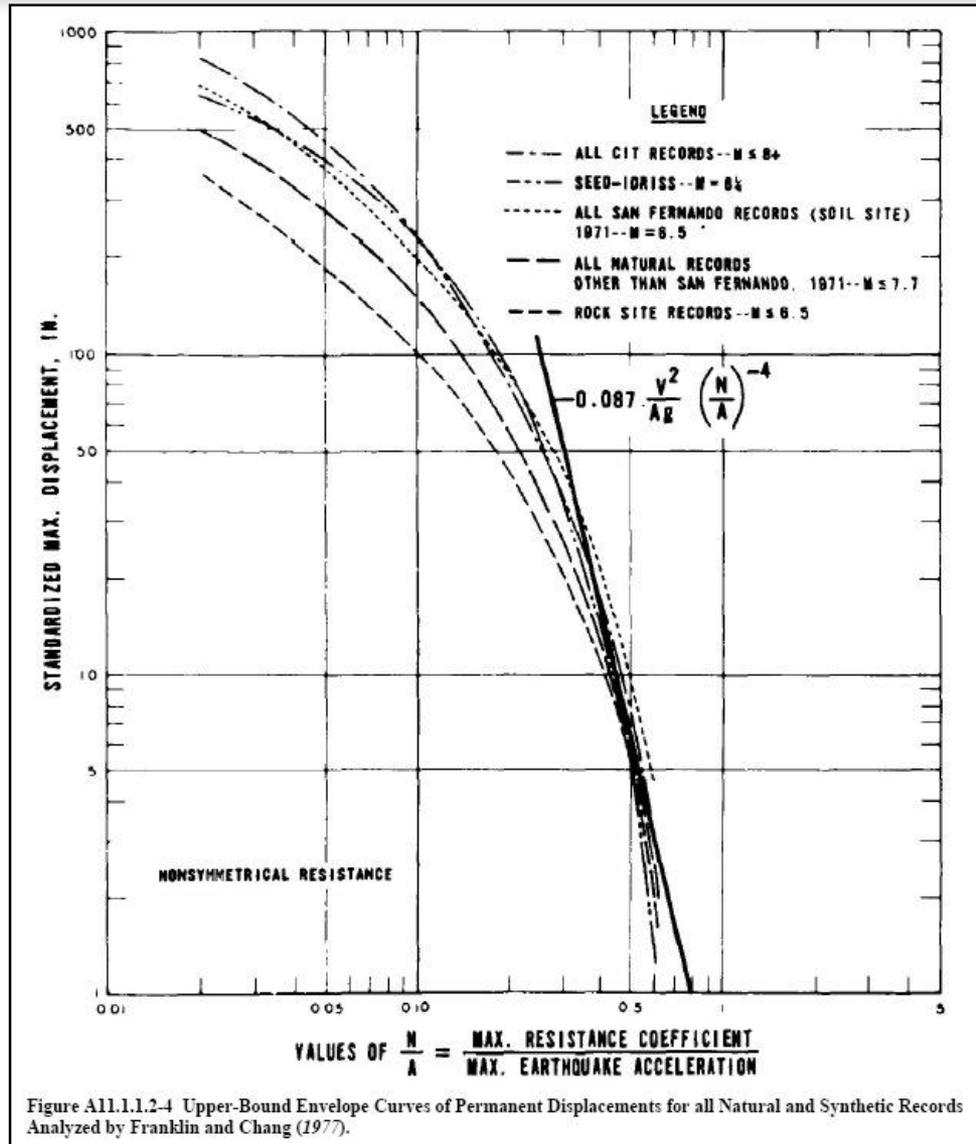
*The proposals of Richards and Elms have simulated use of design methods based upon allowable permanent displacement.*

*Robert V Whitman (1990)*



Displacement Controlled Design  
 Richards and Elms (1979/1990)

# Displacement Analysis



$$N = A \left[ \frac{0.087 V^2}{d A g} \right]^{1/4}$$

Where:

N = Design Cutoff Acceleration

A = Peak Design Acceleration

V = Peak Velocity

d = Allowable Displacement

g = Gravitational Acceleration

Maximum Acceleration, Kh  
Richards and Elms

$$k_h = 0.74 A_s \left[ \frac{A_s}{d} \right]^{1/4}$$

Where:

$k_h$  = Horiz Acceleration Coeff.

$A_s$  = Design Acceleration

$d$  = Allowable Displacement (in)  
(1" - 8" range)

Simplified Acceleration,  $k_h$   
Kavazanjian et al.

*“For most design purposes, it has been shown (Elms and Martin, 1979) that a design value of  $K_h = 0.50A$  is adequate, provided that the wall can accommodate an outward displacement of up to about  $250A$  mm” (10A in inches).*

*Geotechnical Earthquake Engineering by the Federal Highway Administration (1998)*

What is the design earthquake?

- 2% probability in 50 years
- 10% probability in 50 years
- 40% probability in 50 years
- 7% probability in 75 years
- All or some of the above

This is an Owner driven criteria based on the importance of the structure.

- NZ Transportation Agency's Bridge Manual
  - 6.0 Site stability, foundations, earthworks and retaining walls
    - Loads determined from section 6.2.2
- Other NZ design references for MSE walls
  - Road Research Unit Bulletin 84
    - Provides a basis for seismic design
    - Shall be compiled with
  - NZTA research report 239
    - Provides additional guidelines

## 6.2 Design loadings and analysis

### 6.2.1 General

---

Design loads to be considered shall be as specified in section 3 of this manual. In particular, earth loads are specified in 3.4.12 and load combinations in 3.5.

---

### 6.2.2 Earthquake loads and analysis for the assessment of liquefaction and of the stability and displacement of slopes and retaining walls

The design earthquake loading to be applied to soils, rock and independent earth retaining structures shall be derived as set out herein.

Methods for the assessment of liquefaction, slope stability, and slope and retaining wall displacements referred to within this section require the application of peak ground accelerations in combination with a corresponding earthquake magnitude. The peak ground accelerations (PGA) to be applied shall be 'unweighted' and derived for the relevant return period as follows:

$$PGA = C_{0,1000} \times \frac{R_u}{1.3} \times f \times g$$

Where:

$C_{0,1000}$  = 1000 year return period PGA coefficient for a subsoil Class A or B rock site or Class C shallow soil site derived from figure 6.1(a), or for subsoil Class D deep or soft soil site or Class E very soft soil site from figure 6.1(b).

Alternatively, for the locations listed, PGA coefficients may be taken from table 6A.1 contained in addendum 6A

$R_u$  = return period factor derived from table 3.5 of NZS 1170.5 *Structural design actions* part 5 Earthquake actions - New Zealand<sup>(1)</sup> corresponding to the design return period determined from tables 2.2 or 2.3, as appropriate

$f$  = Site subsoil class factor, where

$f = 1.0$  for a Class A, B, D and E soil sites

$f = 1.33$  for a Class C shallow soil site

---

## 6.2.2 continued

The earthquake magnitude shall be derived for the relevant return period from table 6A.1 contained in addendum 6A or figures 6.2(a) to (f).

As a lower bound, the ultimate limit state effects to be designed for shall not be taken to be less than those due to a 6.5 magnitude earthquake at 20km distance, for which the peak ground acceleration coefficients shall be derived from table 6.1.

**Table 6.1: Peak ground acceleration coefficients corresponding to a magnitude 6.5 earthquake at 20 km distance**

Site subsoil class	Class A/B rock	Class C shallow soil	Class D deep or soft soil	Class E very soft soil
PGA coefficient (g)	0.14	0.19	0.16	0.16

Note that PGAs derived using NZS 1170.5<sup>(1)</sup> are magnitude weighted to correspond to an earthquake magnitude of 7.5. Given that the performance of soils, earth structures, slopes and retaining walls exhibit a step-wise behaviour (where a critical acceleration results in a sudden loss of stability, ie dramatic change in behaviour), use of these values may be unconservative. Therefore unweighted PGAs are to be used in the assessment and design of these soil structures for earthquakes.

Unweighted PGAs are to be derived as specified herein. They are not to be back-calculated from NZS 1170.5<sup>(1)</sup> magnitude weighted PGAs as doing so will give rise to inconsistencies due to the approximations that are inherent in the NZS 1170.5<sup>(1)</sup> site hazard spectra.

## Addendum 6A Table 6A.1

Table 6A.1: Unweighted peak ground acceleration coefficients,  $C_{0,1000}$ , corresponding to a 1000 year return at a subsoil Class A or B rock site and subsoil Class D or E deep or soft soil site, and effective magnitude,  $M_{eff}$ , for various return periods for New Zealand towns and cities

Note: For a Class C shallow soil site refer to note 1 at the end of the table.

Town/City	$C_{0,1000}$		Effective magnitudes ( $M_{eff}$ ) for design return period (years)		Town/City	$C_{0,1000}$		Effective magnitudes ( $M_{eff}$ ) for design return period (years)	
	Class A/B rock	Class D&E deep/ soft soil	500 - 2500	50 - 100		Class A/B rock	Class D&E deep/ soft soil	500 - 2500	50 - 100
Kaitia	0.12	0.15	5.75		Opotiki	0.40	0.44	6.1	
Paīhia/Russell	0.13	0.16	5.75		Ruatoria	0.34	0.41	6.1	
Kaikohe	0.12	0.15	5.75		Murupara	0.42	0.41 <sup>2</sup>	6.3	
Whangarei	0.13	0.16	5.8		Taupo	0.38	0.42	6.1	
Dargaville	0.13	0.16	5.8		Taumanuui	0.32	0.36	6.0	
Warkworth	0.13	0.17	5.9		Turangi	0.36	0.40	6.25	
Auckland	0.15	0.19	5.9		Gisborne	0.37	0.41	6.4	
Manakau City	0.17	0.21	5.9		Wairoa	0.35	0.41	6.5	
Waiuku	0.16	0.20	5.9		Waitara	0.27	0.32	6.0	
Pukekohe	0.18	0.22	5.9		New Plymouth	0.28	0.33	6.0	
Thames	0.26	0.30	5.8		Inglewood	0.27	0.32	6.1	
Paeroa	0.28	0.33	5.8		Stratford	0.27	0.32	6.2	
Waihi	0.29	0.34	5.9		Opunake	0.27	0.32	6.1	
Huntly	0.23	0.28	5.8		Hawera	0.26	0.32	6.2	
Ngaruawahia	0.23	0.27	5.8		Patea	0.27	0.34	6.2	
Morrinsville	0.27	0.32	5.8		Whanganui	0.31	0.37	6.0	
Te Aroha	0.29	0.34	5.9		Raetihi	0.35	0.37	6.2	
Tauranga	0.29	0.34	5.9		Ohakune	0.35	0.38	6.2	
Mount Maunganui	0.29	0.34	5.9		Waiouru	0.35	0.39	6.25	
Hamilton	0.24	0.28	5.9		Napier	0.41	0.43	6.9	6.2
Cambridge	0.26	0.32	5.9		Hastings	0.40	0.43	6.9	6.2
Te Awamutu	0.24	0.29	5.9		Waipawa	0.40	0.44	6.75	6.25
Matamata	0.27	0.34	5.9		Waipukurau	0.40	0.44	6.75	6.25
Te Puke	0.30	0.35	6.0		Taihape	0.35	0.41	6.3	
Putaruru	0.29	0.34	6.0		Marton	0.35	0.40	6.25	
Tokoroa	0.31	0.35	6.0		Bulls	0.36	0.41	6.3	
Otorohanga	0.25	0.27	5.9		Feilding	0.40	0.43	6.7	6.1
Te Kuiti	0.26	0.27	5.9		Palmerston North	0.42	0.44	6.9	6.1
Mangakino	0.31	0.36	6.0		Dannevirke	0.43	0.46	7.0	6.2
Rotorua	0.35	0.39	6.0		Woodville	0.44	0.46	7.0	6.2
Kawerau	0.41	0.43	6.2		Pahiataua	0.45	0.47	7.1	6.2
Whakatane	0.43	0.46	6.1		Masterton	0.50	0.45 <sup>2</sup>	7.0	6.3

Table 6A.1: continued

Town/City	$C_{0,1000}$		Effective magnitudes ( $M_{eff}$ ) for design return period (years)		Town/City	$C_{0,1000}$		Effective magnitudes ( $M_{eff}$ ) for design return period (years)	
	Class A/B rock	Class D&E deep/ soft soil	500 - 2500	50 - 100		Class A/B rock	Class D&E deep/ soft soil	500 - 2500	50 - 100
Foxton	0.40	0.42	6.7	6.1	Christchurch <sup>3</sup>	-	-	-	-
Levin	0.43	0.44	6.8	6.1	Akaroa <sup>3,4</sup>	-	-	5.8	6.25
Otaki	0.42	0.44	6.8	6.2	Ashburton	0.25	0.29	6.1	
Paraparaumu	0.42	0.44	6.9	6.2	Geraldine	0.25	0.28	6.0	
Wellington	0.44	0.45	7.1	6.2	Temuka	0.22	0.24	6.0	
Porirua	0.43	0.44	7.0	6.2	Fairlie	0.32	0.32	6.1	
Lower Hutt	0.45	0.45	7.1	6.2	Mt Cook	0.45	0.48	6.9	6.2
Upper Hutt	0.47	0.45 <sup>2</sup>	7.1	6.25	Timaru	0.20	0.23	6.0	
Eastbourne - Point Howard	0.44	0.45	7.1	6.2	Waimate	0.20	0.24	6.0	
					Twizel	0.39	0.37 <sup>2</sup>	6.1	
Wainuiomata	0.47	0.44 <sup>2</sup>	7.1	6.2	Wanaka	0.39	0.42	6.1	
Takaka	0.42	0.46	5.8		Cromwell	0.33	0.37	6.25	
Motueka	0.42	0.46	5.9		Alexandra	0.29	0.32	6.3	
Nelson	0.40	0.43	6.1		Arrowtown	0.39	0.42	6.4	
Pictou	0.35	0.38	6.6	6.1	Queenstown	0.40	0.42	6.5	
Blenheim	0.40	0.42	6.75	6.1	Millford Sound	0.62	0.62	7.1	6.1
St Arnaud	0.46	0.48	6.9	6.1	Danaru	0.22	0.24	6.0	
Westport	0.54	0.52 <sup>2</sup>	5.7		Palmerston	0.22	0.25	6.0	
Reefton	0.53	0.56	6.0		Mosgiel	0.23	0.26	6.0	
Murchison	0.51	0.54	6.2		Dunedin	0.22	0.25	6.0	
Hanmer Springs	0.55	0.53 <sup>2</sup>	7.0	6.5	Te Anau	0.43	0.42 <sup>2</sup>	6.4	
Kaikoura	0.55	0.53 <sup>2</sup>	6.7	6.1	Riverton	0.26	0.30	6.2	
Cheviot	0.43	0.42 <sup>2</sup>	6.6		Winton	0.26	0.28	6.2	
Greymouth	0.50	0.50	6.5		Gore	0.26	0.27	6.2	
Hokitika	0.52	0.54	6.75	6.3	Mataura	0.24	0.26	6.1	
Arthurs Pass	0.65	0.62 <sup>2</sup>	7.0	6.3	Balclutha	0.22	0.24	6.0	
Otira	0.65	0.60 <sup>2</sup>	7.1	6.4	Invercargill	0.21	0.26	6.1	
Darfield <sup>3</sup>	0.40	0.37 <sup>2</sup>	6.25		Bluff	0.20	0.24	6.1	
Rangiora <sup>3</sup>	0.37	0.38	6.4		Oban	0.19	0.23	6.1	

Notes:

1. Shallow soil PGAs are determined from the rock values by multiplying by 1.33.
2. The deep soil PGAs are less than the rock values at some high-hazard locations because of nonlinear site-response effects built into the modeling.
3. The Canterbury earthquake region values are to be determined from a new seismic hazard model for the region in 2014.
4. The  $M_{eff}$  decreases with return period for Akaroa because its estimated hazard has a larger contribution from the Alpine Fault at low acceleration values which is replaced by contributions from local earthquakes as the PGAs increase.
5.  $M_{eff}$  values given in this table may vary slightly from those derived from the maps as they have been assessed conservatively to apply across a range of return periods.

Figure 6.1(a): Unweighted peak ground acceleration coefficients,  $C_{0,1000}$ , corresponding to a 1000 year return at a subsoil Class A or B rock site or Class C shallow soil site



Note: For Class C sites a scale factor of  $\neq 1.33$  needs to be applied to the PGA coefficients derived from this figure.

Figure 6.1(b): Unweighted peak ground acceleration coefficients,  $C_{0,1000}$ , corresponding to a 1000 year return at a subsoil Class D or E deep or soft soil site



- 6.6 Earth Retaining Systems
  - Numerous codes can provide guidance

#### 6.6.2 Design standards

The following standards and codes of practice provide guidance on the design of retaining structures:

- Road Research Unit bulletin 84, volume 2<sup>(3)</sup>.
- BS EN 1997-1 *Eurocode 7 Geotechnical design part 1 General rules*<sup>(13)</sup>, plus BS EN 1998-5 *Eurocode 8 Design of structures for earthquake resistance part 5 Foundations, retaining structures and geotechnical aspects*<sup>(14)</sup>.
- AS 4678 *Earth-retaining structures*<sup>(19)</sup>.
- CAN/CSA-S6 *Canadian highway bridge design code*<sup>(15)</sup>.
- AASHTO LRFD *Bridge design specifications*<sup>(16)</sup>.
- FHWA NHI-99-025 *Earth retaining structures*<sup>(20)</sup>.
- CIRIA C580 *Embedded retaining walls – guidance for economic design*<sup>(21)</sup>.

Road Research Unit bulletin 84<sup>(3)</sup> shall be used in preference to the other documents, particularly for earthquake resistant design.

The NZTA's *Bridge manual* shall take precedence over all other documents.

- 6.6.9 Design performance of earth retaining structures and slopes
  - “Retaining structures and slopes may be designed to remain elastic under the design earthquake load specified in 6.2.2 or to allow limited controlled permanent outward displacement under strong earthquake shaking.”
  - “The displacement likely at the design ultimate limit state seismic response, and under the MCE (maximum considered event), shall be assessed using moderately conservative soil strengths consistent with the anticipated strain and Newmark Sliding Block displacement approach.”

**Table 6.4: Maximum allowable displacement**

Wall and slope situation	Wall and slope type	Maximum displacement
Wall or slope supporting or containing bridge abutments or piers	All types	Refer to 6.6.9(b)
Walls or slopes above road level supporting structures within 2H of wall face at top of wall or bottom of the slope	All types	25mm
Walls or slopes supporting road carriageway with AADT $\geq$ 2500	Rigid wall	100mm
	Flexible wall or slope capable of displacement without structural damage	150mm
Walls or slopes supporting road carriageway with AADT $<$ 2500	Rigid wall	100mm
	Flexible wall or slope capable of displacing without structural damage	200mm

**Notes:**

1. H is the height of the retaining wall including the height of any slope above, or the height of the slope.
2. AADT is the annual average daily traffic count.
3. The designer shall ensure that the displacements will not cause damage to adjacent structures or services.

**b. Walls and earth structures (including slopes) supporting abutments or piers**

In locations of relatively lower seismicity with a hazard factor  $Z < 0.3$ , walls or earth retaining structures supporting abutments or piers shall be designed to prevent permanent displacement under the design earthquake load.

Subject to obtaining the agreement of the road controlling authority, in zones of higher seismicity ( $Z \geq 0.3$ ), where the bridge abutment and superstructure can be designed to remain serviceable with limited abutment displacement and without damage to the bearings or piles, and can retain adequate allowance for temperature change, vibration etc, walls or earth retaining structures supporting abutments or piers may be designed on the basis of sustaining permanent displacement under the design ultimate limit state earthquake event subject to the limitations below. This shall be substantiated in the structure design statement, which shall include quantification of the damage due to the movements and the consequences for the use of the bridge and its permanent repair to full capacity for design loading and movements.

In zones of higher seismicity, the following absolute limits on displacement of the walls or earth retaining structures supporting abutment or piers shall not be exceeded:

- In locations with a hazard factor  $0.3 \leq Z < 0.4$ : vertical displacement shall not exceed 25mm and longitudinal and transverse horizontal displacements shall not exceed 50mm.

Retaining walls around bridges have been designed for a 2% exceedance in 50 year event.

Retaining walls away from bridges have been designed for a 7% exceedance in 75 year event.

Note: The PGA of a 7% probability event is around half of a 2% probability event.

## 11.5.4.2 AASHTO – Extreme Event I, No Analysis

“A seismic design shall not be considered mandatory for walls located in Seismic Zones 1 through 3, or for walls at sites where the site adjusted peak ground acceleration,  $A_s$ , is less than or equal to 0.4g, unless one or more of the following is true:”

## Cont'd

- Liquefaction induced lateral spreading or slope failure, or seismically induced slope failure, due to the presence of sensitive clays that lose strength during the seismic shaking, may impact the stability of the wall for the design earthquake.
- The wall supports another structure that is required based on the applicable design code or specification for the supported structure to be designed for seismic loading and poor seismic performance of the wall could impact the seismic performance of that structure.

## Cont'd

- In Seismic Zones 2 and 3
  - Exposed wall height plus average surcharge depth is  $> 30'$
  - Tiered walls the sum of the exposed height of all the tiers plus the average soil surcharge depth is  $> 30'$
  - The wall has abrupt changes in its alignment geometry (e.g., corners and short radius turns at an enclosed angle of 120 degrees or less)
  - For gravity and semi-gravity walls, the wall backfill does not meet the requirements of Article 7.3.6.3 of AASHTO.

## Observed Seismic Performance for Walls

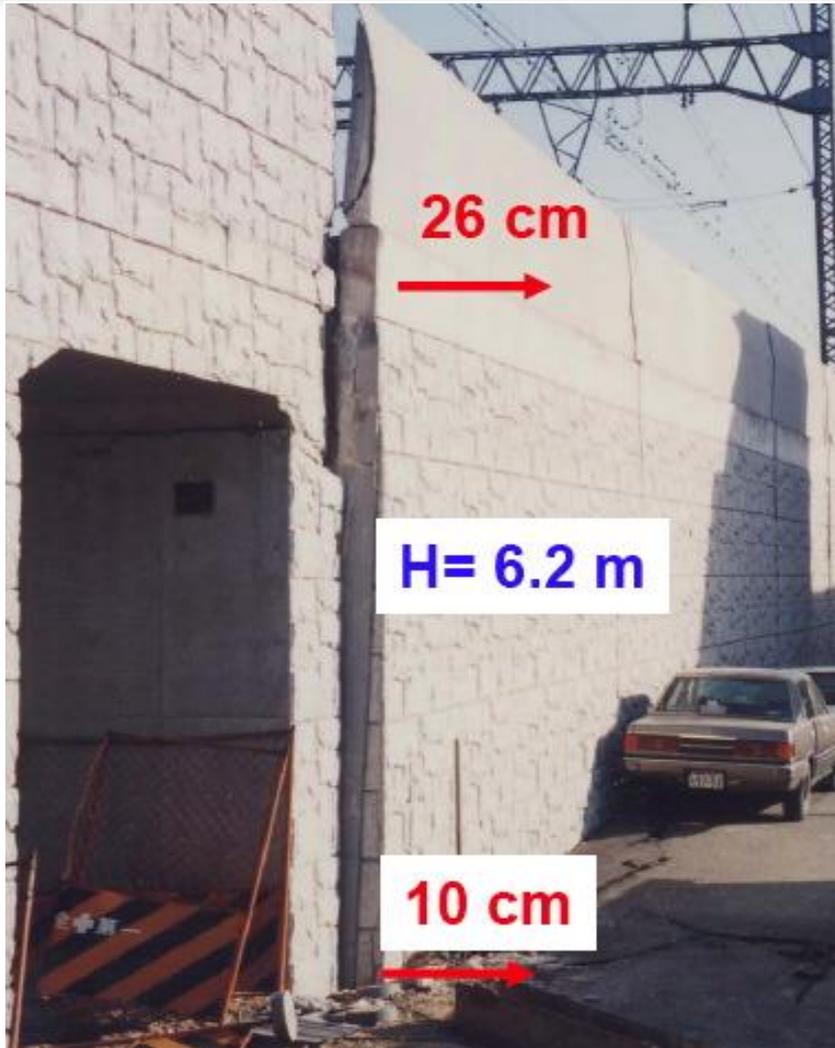
- Good performance of MSE Walls in Seismic Events
  - 1995 Kobe Earthquake; masonry and concrete gravity walls collapsed due to weak soils, heavy soil surcharges, or structural failure, mainly where  $A_s > 0.6g$ ; MSE Walls had some damage but did not collapse even up to  $0.8g$
  - 1999 Izmit Earthquake ( $A_s > 0.40g$ ); Rigid Structure Collapse MSE Structures remained in place
  - 2001 San Salvador Earthquake ( $A_s > 0.30g$ ); Example wall shown



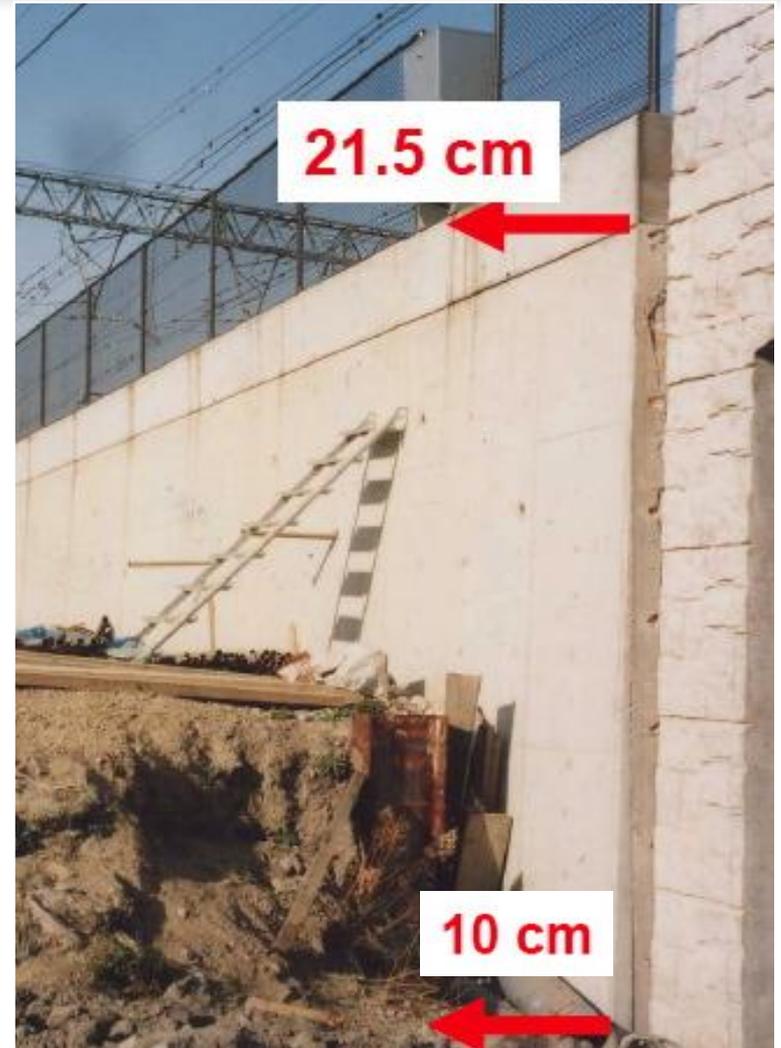
GRS Wall - 1992 Before



GRS Wall - 1995 After



GRS Wall - After



RCW Wall - After







## Laboratory Seismic Studies

- Two seismic forces are out of phase
  - Dynamic earth pressure was at its maximum, the wall inertial force was at its minimum or very close to 0
  - When the wall inertial force was at its maximum, the total seismic earth (Pae) was close to its static value.
    - Nakamura, S. (2006). “Re-examination of Mononobe-Okabe theory of gravity retaining walls using centrifuge model tests.” *Soils Foundation* 46(2), 135-146
    - Al Atik, L., and Sitar, N. (2010). “Seismic earth pressures on cantilever retaining structures.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(10), 1324-1333
- Seismic earth pressures appear to not develop until  $A_s > 0.4g$ 
  - Al Atik, L., and Sitar, N. (2010). “Seismic earth pressures on cantilever retaining structures.” *Journal of Geotechnical and Geoenvironmental Engineering*, 136(10), 1324-1333

## **Seismic Earth Pressures on Cantilever Retaining Structures**

Linda Al Atik and Nicholas Sitar, 2010

Abstract in the Journal of Geotechnical and  
Geoenvironmental Engineering – ASCE

Laboratory Centrifuge Experiments

**Abstract:** An experimental and analytical program was designed and conducted to evaluate the magnitude and distribution of seismically induced lateral earth pressures on cantilever retaining structures with dry medium dense sand backfill. Results from two sets of dynamic centrifuge experiments and two-dimensional nonlinear finite-element analyses show that maximum dynamic earth pressures monotonically increase with depth and can be reasonably approximated by a triangular distribution. Moreover, dynamic earth pressures and inertia forces do not act simultaneously on the cantilever retaining walls. As a result, designing cantilever retaining walls for maximum dynamic earth pressure increment and maximum wall inertia, as is the current practice, is overly conservative and does not reflect the true seismic response of the wall-backfill system. The relationship between the seismic earth pressure increment coefficient ( $\Delta K_{AE}$ ) at the time of maximum overall wall moment and peak ground acceleration obtained from our experiments suggests that seismic earth pressures on cantilever retaining walls can be neglected at accelerations below 0.4 g. This finding is consistent with the observed good seismic performance of conventionally designed cantilever retaining structures.

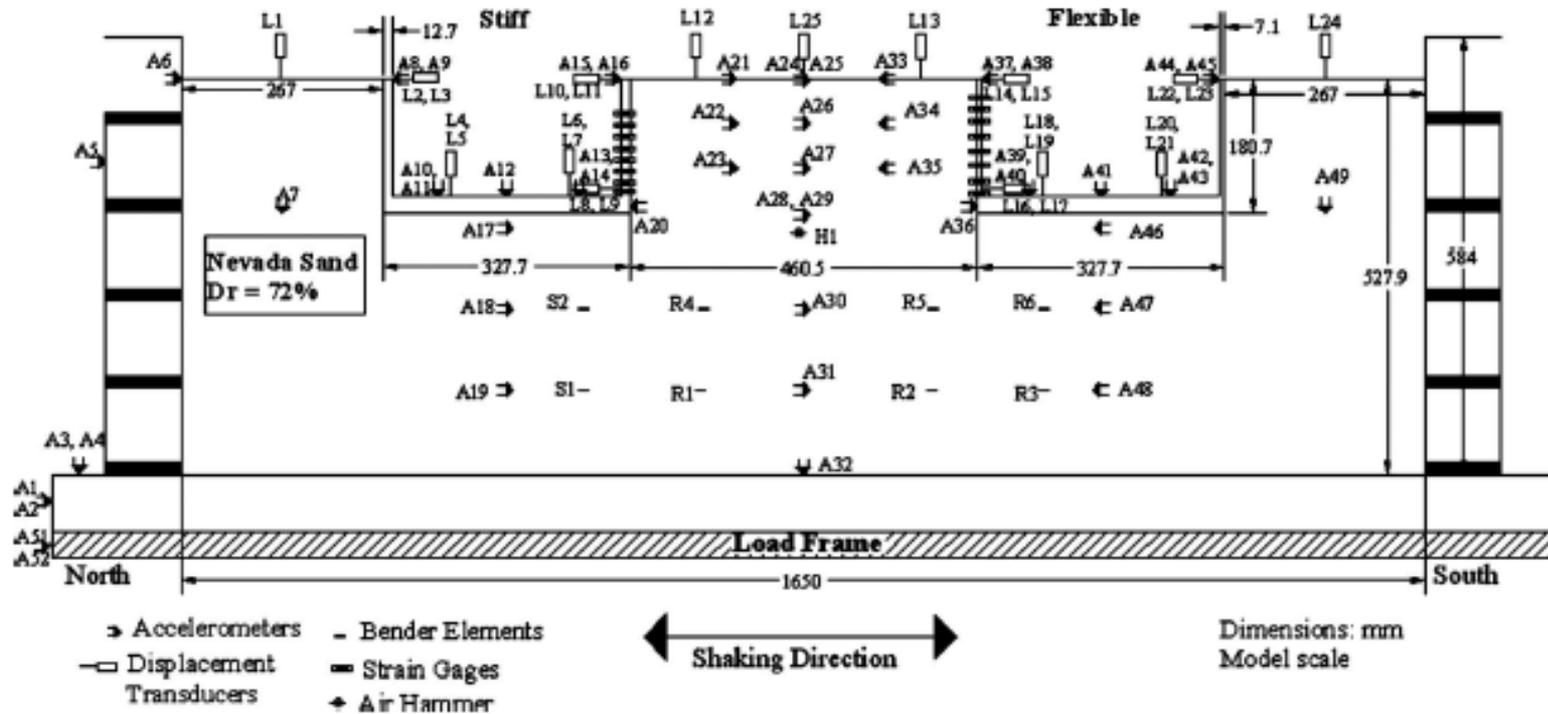
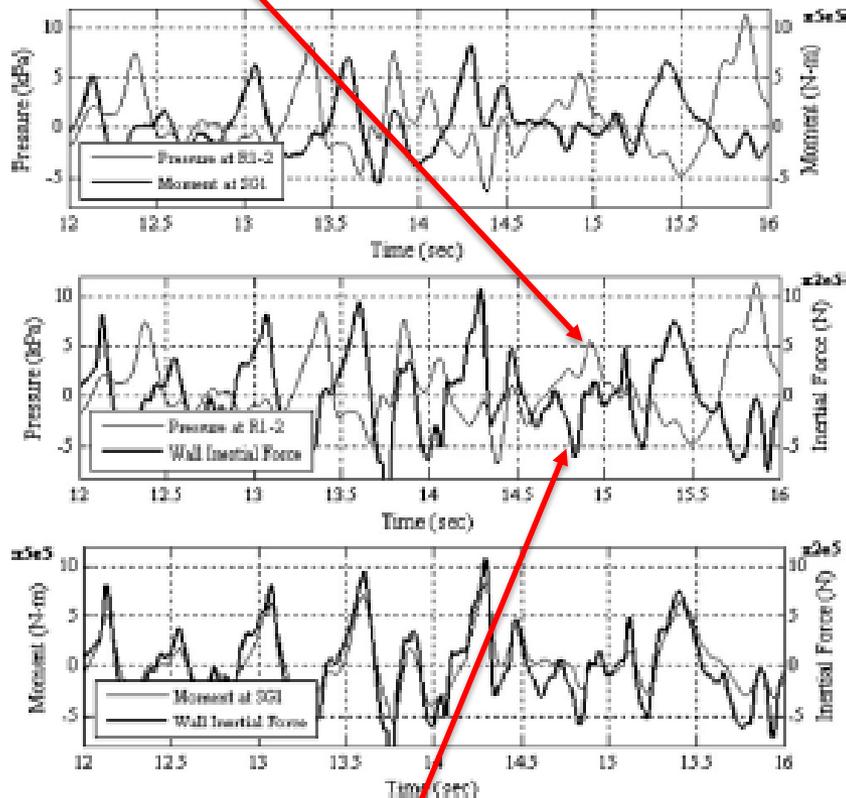


Fig. 1. LAA02 model configuration, profile view

Dynamic Earth Pressure near maximum



## Seismic Behavior of Wall – Backfill System

- Comparison of dynamic moment increments, dynamic earth pressure increments and wall inertial forces.

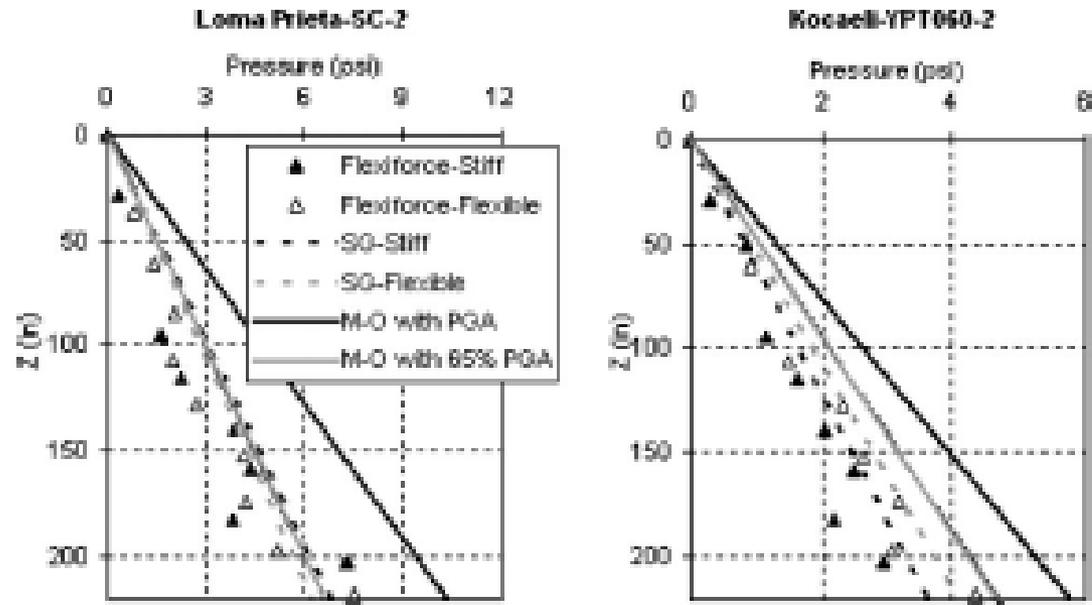
Wall mass inertial force near zero or negative

## Observations and Interpretations

“...when the inertial force acts in the active direction, the total earth pressure is equal to or less than the static earth pressure. Dynamic earth pressure increment is at its maximum when the inertial force is close to zero (i.e. static case) or when the inertial force acts the passive direction.”

## Observations and Interpretations

“In contrast, the limit equilibrium assumption inherent in MO theory means that the earth pressure increases when the inertia force is loaded in the active direction and stability analyses of retaining are usually conducted for maximum dynamic earth pressures and inertia forces.”



**Fig. 5.** Maximum total earth pressure profiles measured and estimated using the MO method on the south stiff and north flexible walls during Loma Prieta-SC-2 and Kocaeli-YPT060-2 shaking events

Shows earth pressure distribution is triangular, indicating resultant at  $h/3$  and less than 65% of M-O earth pressure.

1. The experimental and numerical analysis results consistently show that the maximum dynamic earth pressures increase with depth and can be reasonably approximated by triangular distribution analogous to that used to represent static earth pressures. Consequently, there seems to be no basis for the currently accepted position of the point of application the dynamic earth pressure force in dynamic limit equilibrium analyses at  $0.6$  to  $0.67 H$  and, instead, the point of application should be at  $1/3H$ , as originally suggested by Mononobe and Matsuo (1932).

2. An important aspect of the dynamic interaction between the cantilever retaining walls and retained soils is the fact that the maximum dynamic earth pressures and maximum wall inertial forces do not tend to occur simultaneously. As a result, the current design methods based on the MO theory were found to significantly overestimate the recorded dynamic earth pressures and moments.

3. The relationship between the back-calculated seismic earth pressure increment coefficient ( $\Delta K_{ae}$ ) at the time of maximum dynamic wall moment and peak ground acceleration obtained from our experiments suggests that seismic earth pressures on cantilever retaining walls can be neglected at accelerations below 0.40g.

4. The analytical results show that the FE analysis is able to capture quite well the essential system responses observed in centrifuge experiments. However, the veracity of the numerical analyses is strongly dependent on access to high quality experimental or field performance data for model calibration and, therefore, field performance predictions using numerical models should be approached with caution.

- Take everything with a grain of salt
  - Theory is that M-O Seismic design is overly conservative
  - MSE Walls that have been designed for Static Conditions at:
    - Reinforcement Lengths =  $0.7 * H$
    - AASHTO Select backfill
    - Proper vertical spacing and strengths design for internal stability
- Will perform well in seismic applications under 0.40g without Seismic design.

- Does this mean that seismic design can / should be eliminated below 0.40g?
  - Use your judgement
  - Slopes are still a big issue, especially high backslope low friction angle soils
  - Likely 20 states or so have  $A > 0.40g$
  - Most seismic states will require a design even if it is below 0.40g.
    - $K_{ho} = F_{pga} \times PGA = A_s$ , For  $PGA > 0.50g \rightarrow F_{pga} = 1.0$ , site class B, C, D
    - $K_h = 0.5 \times K_{ho}$  (or  $A_s$ )
- The below 0.40g criteria was broadcast generally to all wall types including gravity and semi-gravity.
  - The idea that segmental gravity walls including large gravity block walls (no geosynthetic reinforcement) can withstand seismic below 0.40g without toppling over seems aggressive. Especially considering the large rigid structures failing in seismic conditions.
- Review counter points
  - Leshchinsky, D., and Vahedifard, F., and Shahrokhbabadi, S., “Does No-seismic Design in AASHTO violate AASHTO’s rules?” *2015 Geosynthetics Conference, February 15-18, Portland, OR*

## 5. CONCLUSIONS

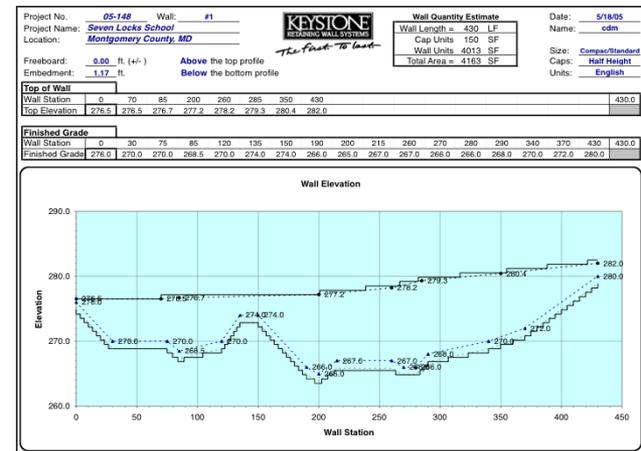
In its 2012 edition, AASHTO has drastically changed its design rules for MSE walls subjected to seismic loading. The changes were justified by observed field performance, limited laboratory work, and recommendations by some researchers. Most notable change was an exemption from seismic design when the design acceleration is  $A_s \leq 0.4g$ . This exemption was examined for consistency in this paper using AASHTO own design rules. That is, a sound design code should not allow for two substantially different outcomes when doing the design for identical data.

It is shown that AASHTO's no seismic analysis design rules are generalizations that are inconsistent with AASHTO. Using AASHTO's calculations, the following is observed:

- Backslope inclination has significant impact on sliding stability. For realistic data, unacceptable performance may occur when  $A_s \ll 0.4g$ .
- Interface friction with the foundation soil has significant impact on sliding stability. For realistic data unacceptable performance may occur when  $A_s \ll 0.4g$ .
- The backslope itself, regardless of the wall, could be unstable when  $A_s \ll 0.4g$ . This possibility is not addressed explicitly by AASHTO.

It is concluded that when using AASHTO's own criteria, its broad exemption from seismic design must be revisited. Essentially, the rule of  $A_s \leq 0.4g$  is the core issue. One possible solution is to augment the criteria for exemption so as to render a universal design that is indeed practically consistent for  $A_s \leq 0.4g$ . This is not simple as there many possible situations in design and not all are easy to capture with such a singular number. The preferred approach is rational, coinciding with sound engineering; i.e., do the design considering seismicity for any  $A_s$  and then realize whether it impacts the outcome. Clearly, AASHTO's exemption from seismic design of MSE walls may render an unsafe outcome using AASHTO's own calculations. This paradoxical situation is to be expected when an ill-defined magic number is used in engineering.

- Design Software
  - KeyWall®
  - Excel spreadsheets
- Technical Notes
- Standard Drawing Details
- Design Manual
- Construction Manual
- Specifications



## **Dan Tix, P.E.**

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Keystone Retaining Wall Systems, LLC*

Dan has been working in the segmental retaining wall industry for 12 years. Dan has over 9 years of MSE Wall design experience working in Keystone's engineering department. His current position is Engineered Structures Technical Manager, where he coordinates and manages Keystone's DOT product submittals, is responsible for managing structural wall product offerings nationwide and teams with Keystone's region managers and international partners to provide technical / engineering support for project sales and promotion. Prior to working for Keystone, he worked 2 years as a geotechnical consulting engineer in Michigan. Dan graduated from Michigan Technological University with a B.S. in Civil Engineering in 2001. He is a licensed Professional Engineer in the state of Minnesota since 2007.