STATIC AND SEISMIC PRESSURES FOR DESIGN OF RETAINING WALLS

By

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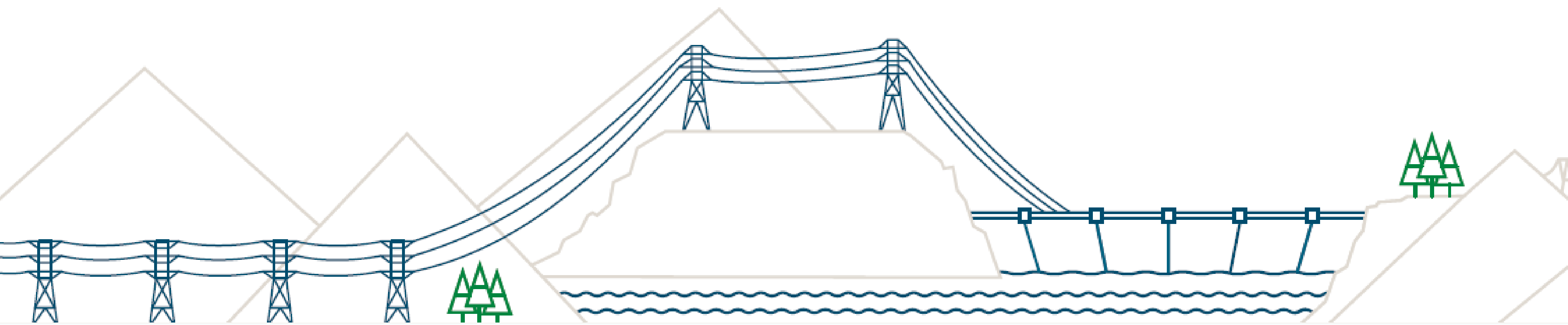
A Presentation to BC Hydro Generation Engineering

on September 27, 2017

(modified pages 64 & 65 in December 2017 for publishing)



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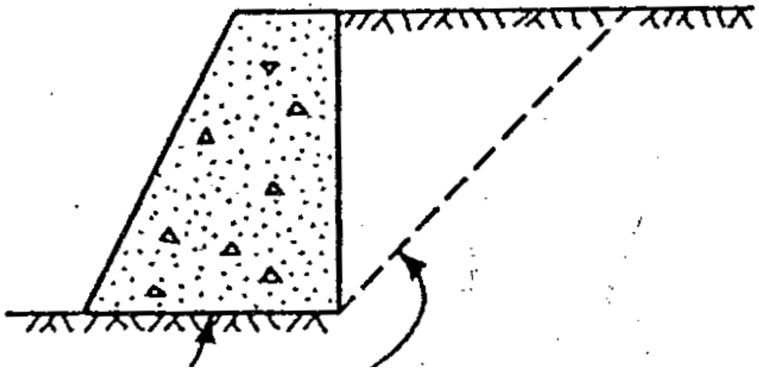
OUTLINE

- **Presentation Outline**
  1. Static behaviour of retaining walls
  2. Seismic pressures on yielding/active walls
  3. Seismic pressures on displacing wall
  4. Seismic pressures on non-yielding walls
  5. Seismic pressures on basement wall

Typical Retaining Walls

A gravity-type stone retaining wall



A gravity wall relies solely on its mass and geometry to resist the soil pressure forces acting on it:

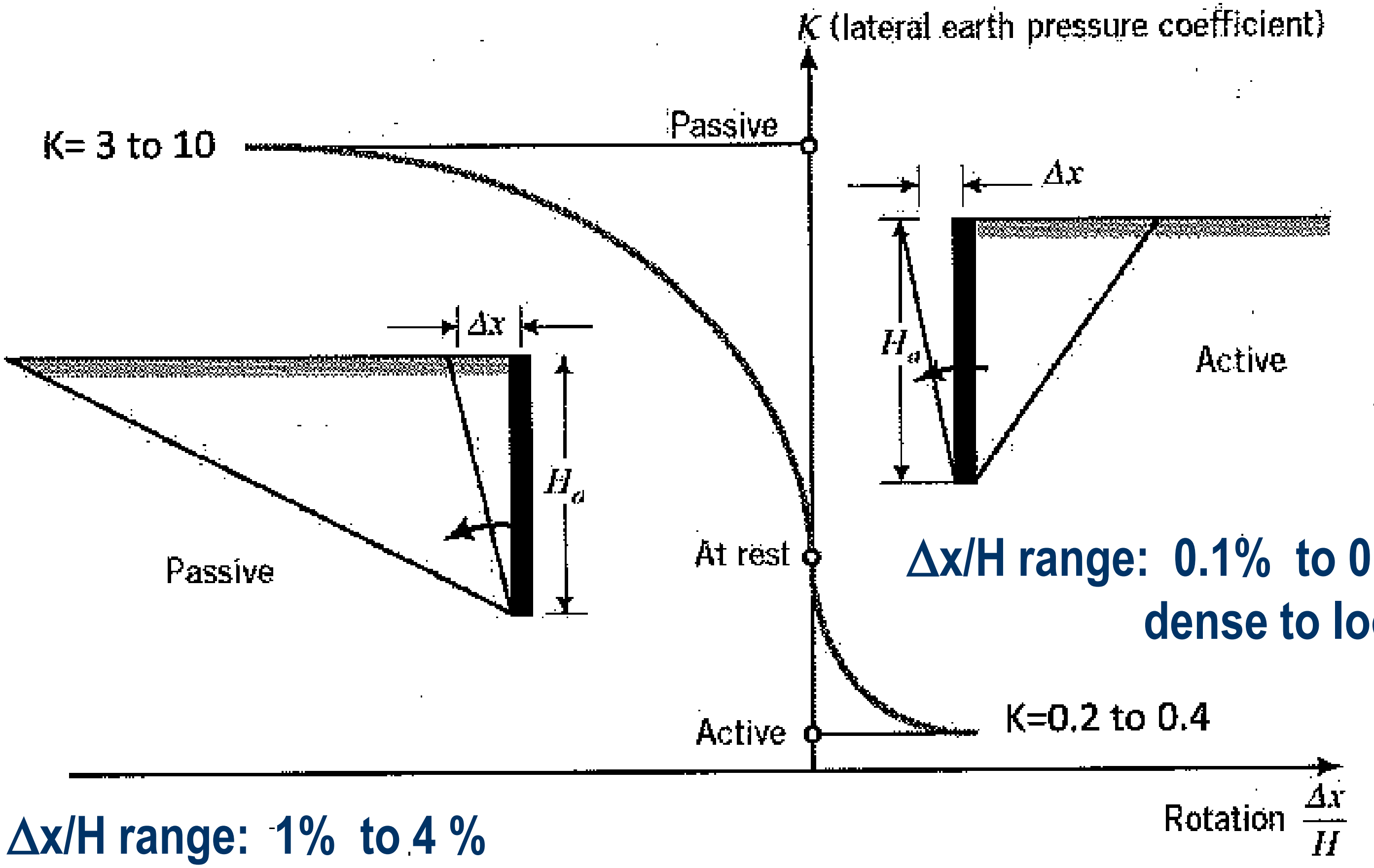
Segmental block wall; one large concrete block or multi-layer blocks (Lock-Block)

An anchored sheet pile wall

Sheet Pile Wall at Ruskin Dam Right Abutment (2009)

1. Over View Static by Dr. Wu

Active and passive soil pressure concept:



Δx/H range: 0.1% to 0.4 %
dense to loose sand

Δx/H range: 1% to 4 %
dense to loose sand

Source: Budhu 2009; AASHTO1998, Caltrans 2004

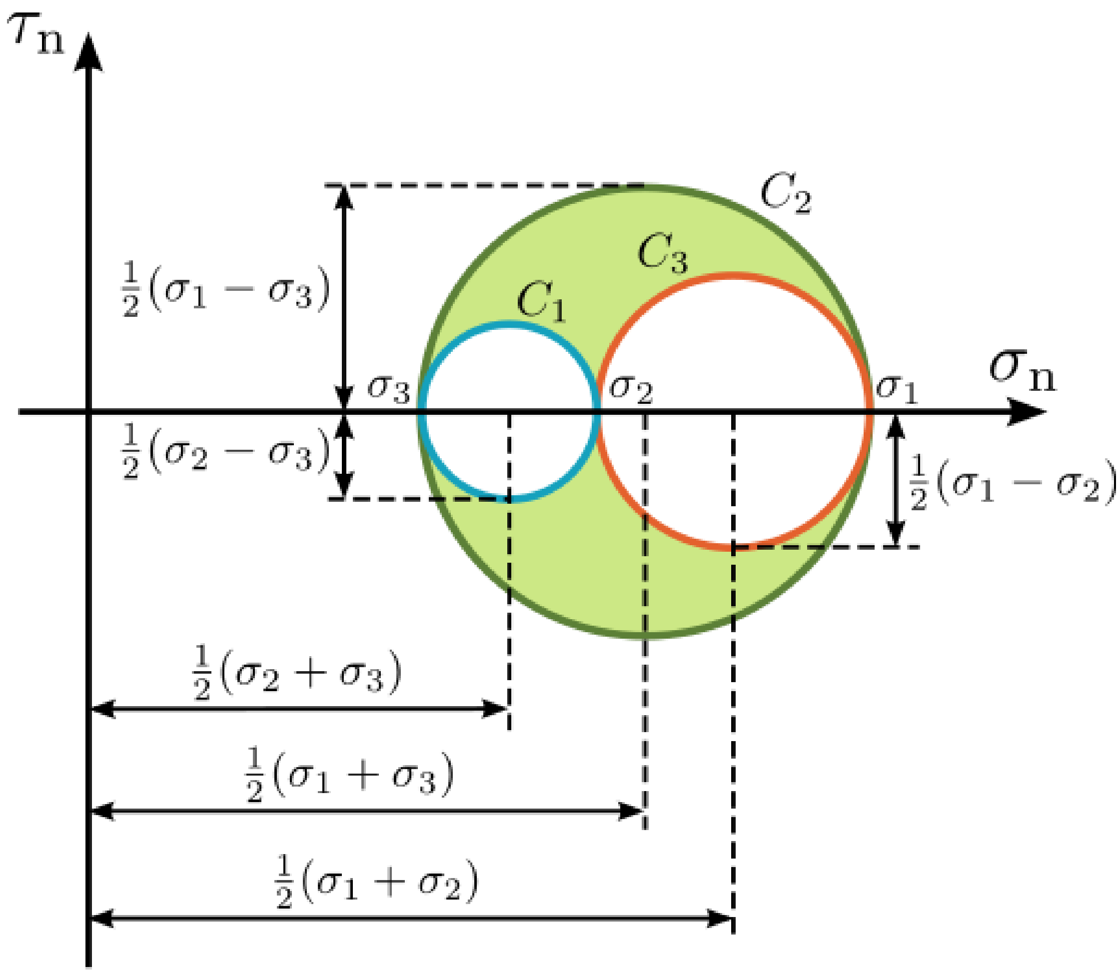
- State of Soil Pressures:
  - Clough and Duncan (1991), and adopted in AASHTO (1998), Caltrans (2004)

Stress State	Dense sand Δ/H	Loose sand Δ/H
Passive	1%	4%
At rest or non-yielding	0	0
Active	0.1%	0.4%
Displacing walls (seismic)	*>>0.1%*	*>>0.4%*

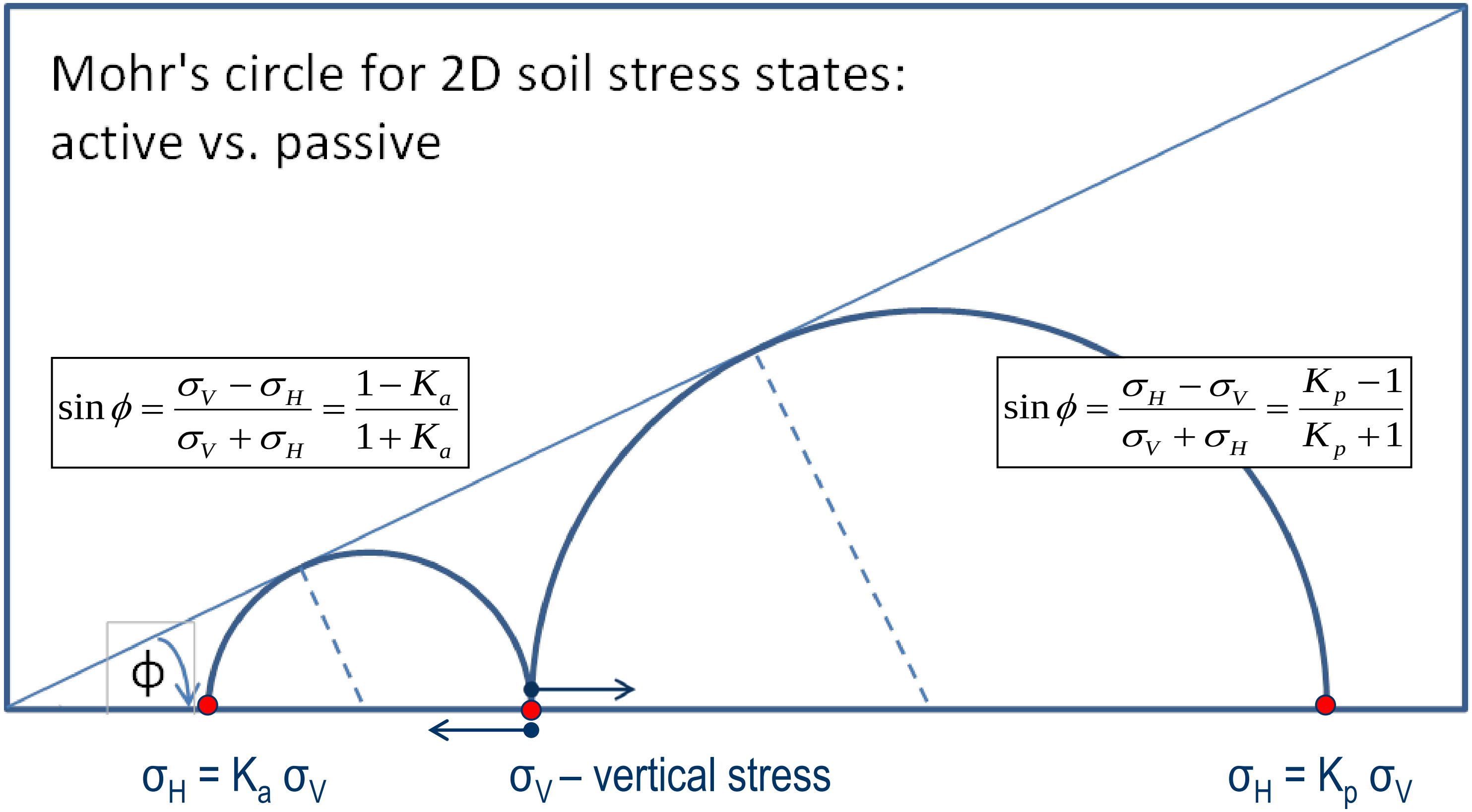
Δ = movement of top of wall required to reach minimum active or maximum passive soil pressure, by tilting or lateral translation.

Source: Budhu 2009; AASHTO1998, Caltrans 2004

Mohr's circle for 3-dimensional stress states:



Rankine Soil Pressure Concept:



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

$$\sin\phi = \frac{\sigma_H - \sigma_V}{\sigma_V + \sigma_H} = \frac{K_p - 1}{K_p + 1}$$

Rankine Soil Pressure Coefficient:

Active:

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

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right)$$

Passive:

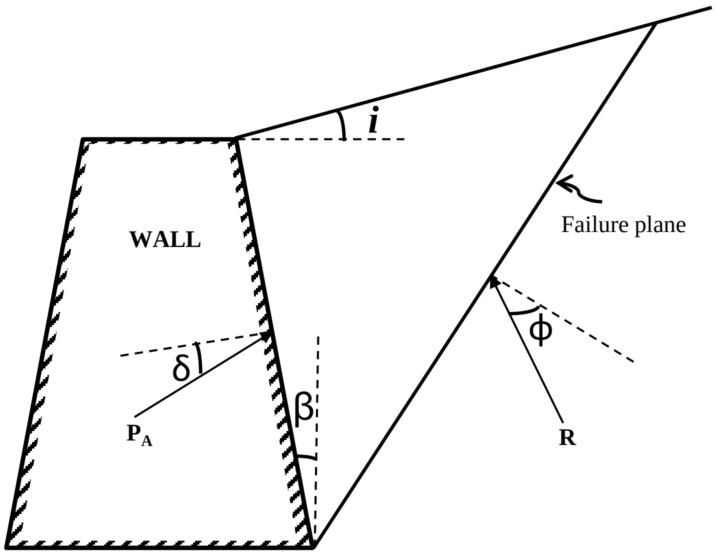
$$\sin\phi = \frac{\sigma_H - \sigma_V}{\sigma_V + \sigma_H} = \frac{K_p - 1}{K_p + 1}$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

Coulomb's Soil Pressure Coefficient :

Active Soil Force: $P_A = \frac{1}{2} k_A \gamma H^2$

$$K_A = \frac{\cos^2(\phi - \beta)}{\cos^2\beta \cos(\delta + \beta)[1 + \sqrt{\frac{\sin(\phi + \delta)\sin(\phi - i)}{\cos(\delta + \beta)\cos(i - \beta)}}]^2}$$



where:

k_A = active soil pressure coefficient

Φ = angle of soil friction;

δ = angle of wall friction;

i = slope of ground surface behind the wall

β = slope of back of wall to vertical

Coulomb's soil pressure coefficient :

Passive Soil Force: $P_P = ½\ k_P\ \gamma H^2$

$$K_P = \frac{\cos^2(\phi + \beta)}{\cos^2\beta \cos(\delta - \beta)[1 - \sqrt{\frac{\sin(\phi + \delta)\sin(\phi + i)}{\cos(\delta - \beta)\cos(i - \beta)}}]^2}$$

where:

k_P = passive soil pressure coefficient

Φ = angle of soil friction;

δ = angle of wall friction;

i = slope of ground surface behind the wall

β = slope of back of wall to vertical

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Coulomb's Passive Soil Pressure Coefficient :

Limitation of application: δ < 0.4Φ

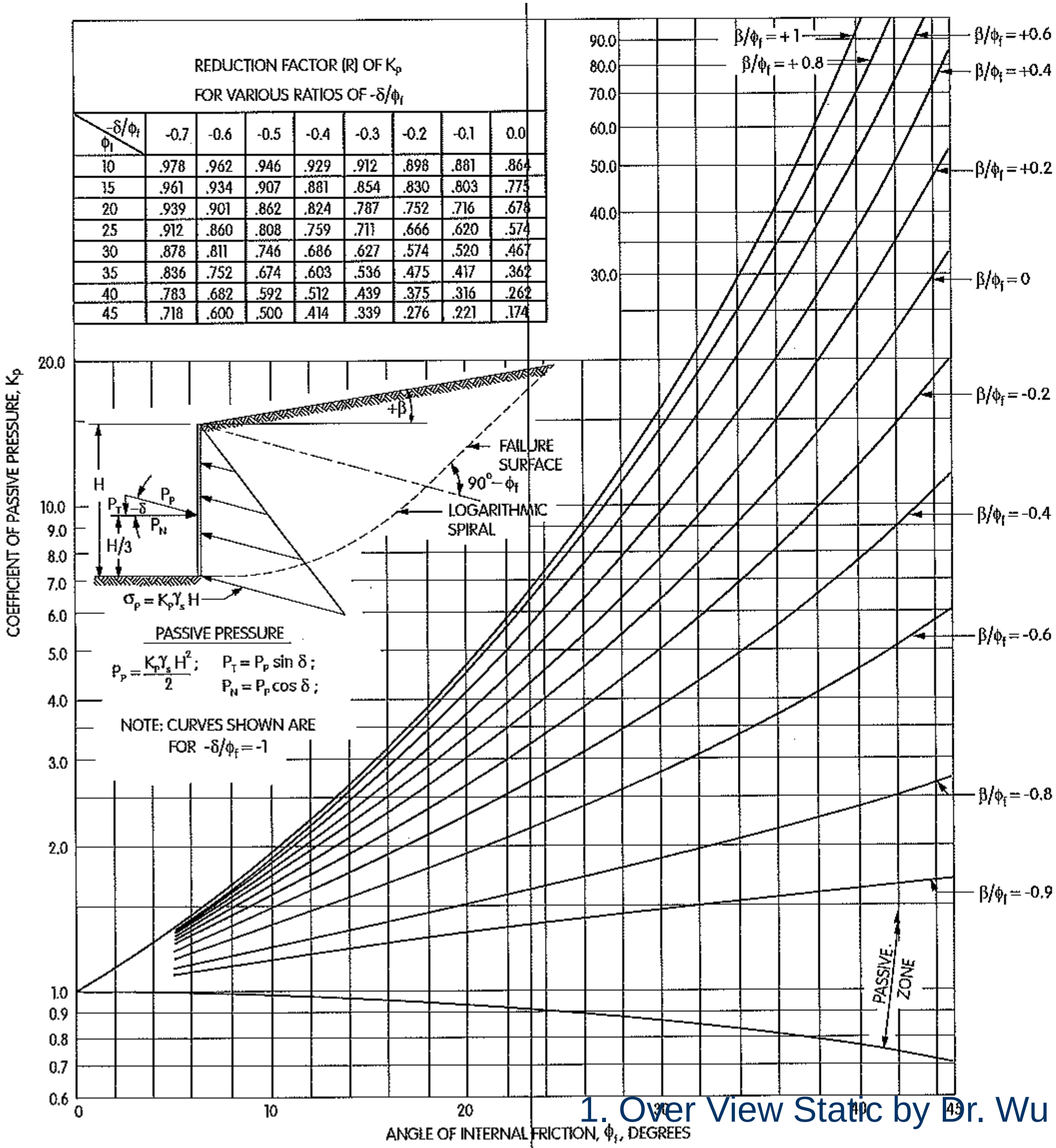
Φ = angle of soil friction,

δ = angle of wall friction.

Duncan and Mokwa (March 2001, ASCE J. Geot.)

Wall friction (δ/Φ)	Coulomb's Theory	Log Spiral Method
0.0	4.6	4.6
0.2	6.3	6.6
0.4	9.4	9.0
0.6	15.3	11.9
0.8	30.4	15.5

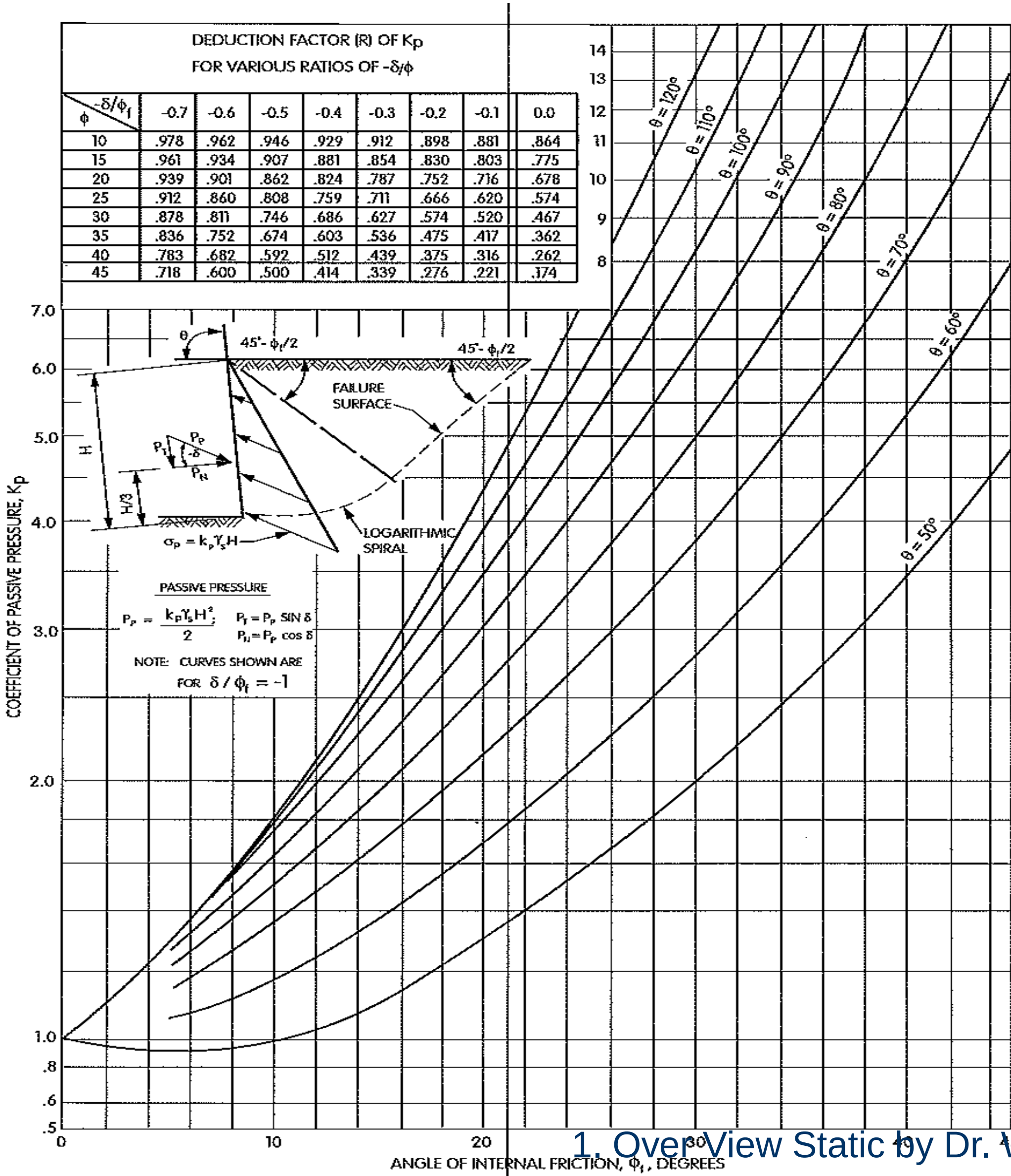
Static Behaviour of Retaining Walls



ϕ_f \ $-\delta/\phi_f$	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Passive soil coefficient
Log Spiral method: **Vertical Wall**
Caquot and Kerisel (1948),
after NAVFAC (1971)
modified by Caltrans (2004)

Static Behaviour of Retaining Walls



DEDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ $-\delta/\phi_f$	-0.7	-0.6	-0.5	-0.4	-0.3	-0.2	-0.1	0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
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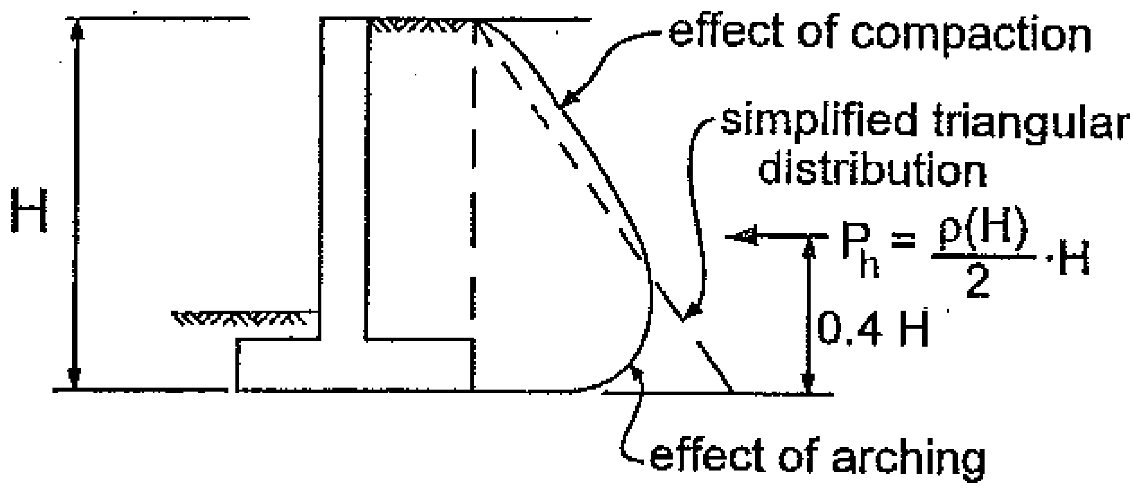
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

Passive soil coefficient
Log Spiral method: **Hori Soil**
Caquot and Kerisel (1948), after NAVFAC (1971) modified by Caltrans (2004)

Soil Pressure Acting Point :

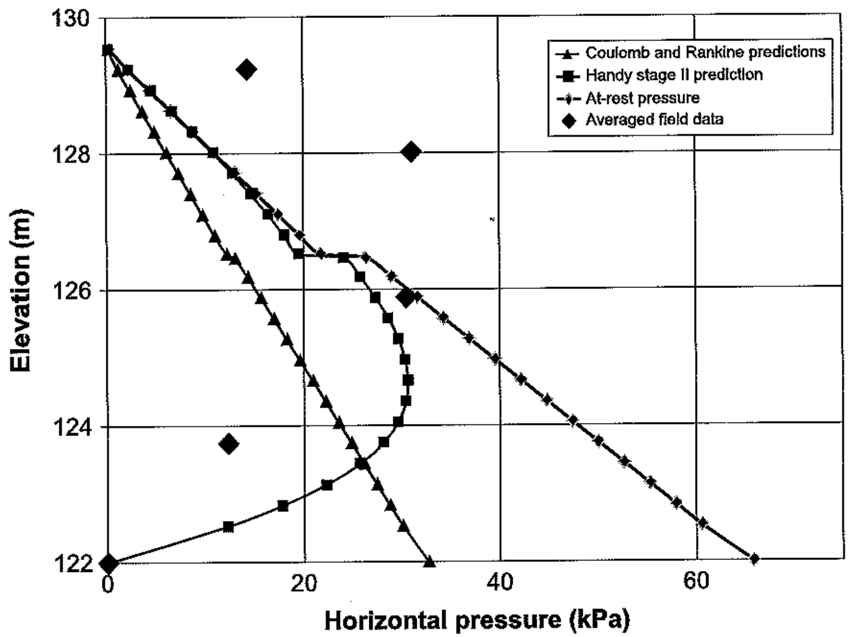
1). One third above the base, 0.33H

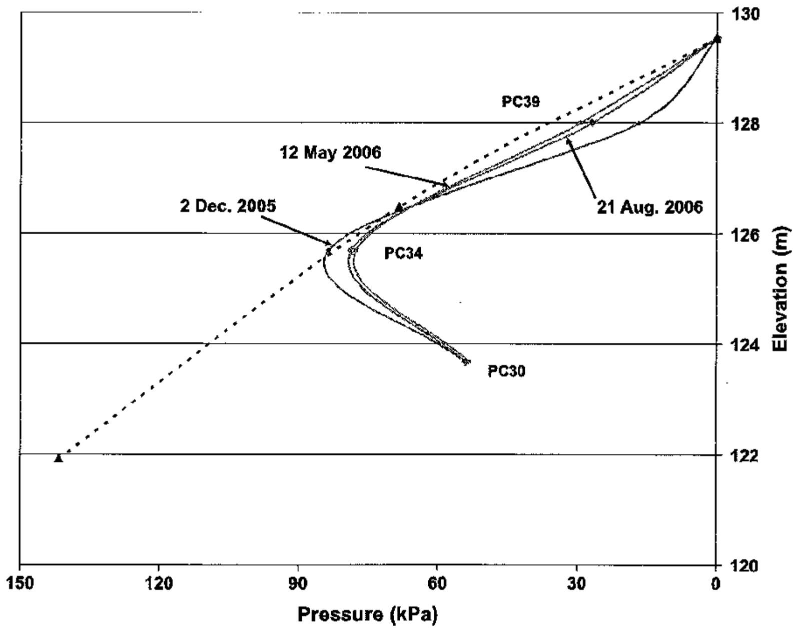
2). 0.40H (AASHTO 1998)
- Effect of compaction
- Effect of arching



Soil pressure distribution:

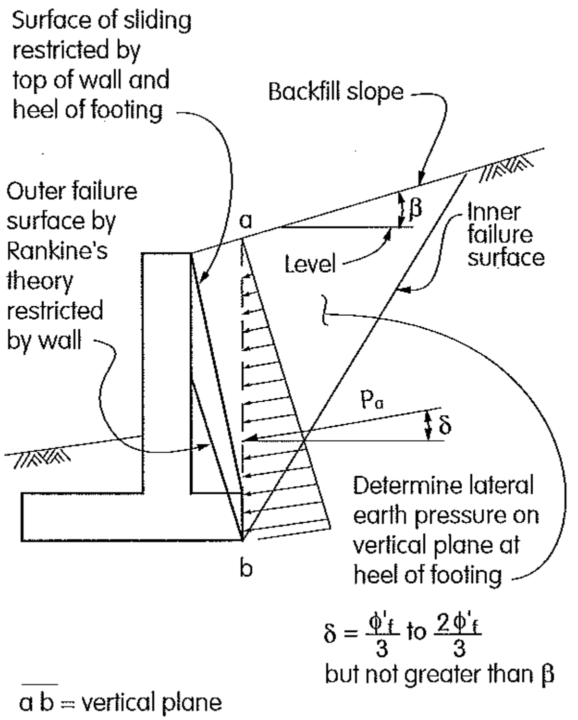
- Arching effect for confined backfill in tall rigid walls:
- Vertical pressures for roller compacted concrete wall (RCC)
- Lateral soil pressures: measured vs. prediction





Source: O'Neal and Hagerty 2011, Can Geotech J. 48: 1188-1197

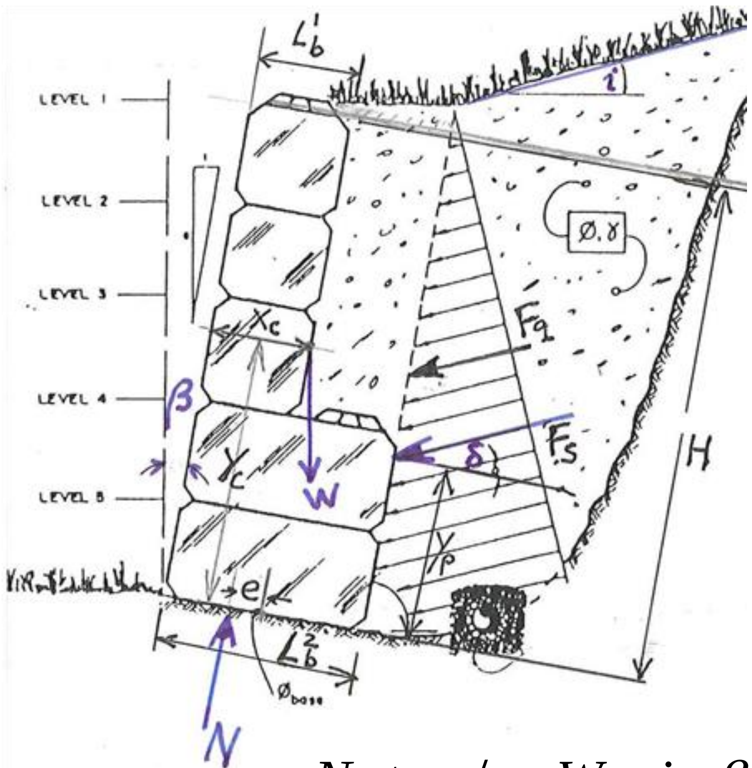
Semi-Gravity Wall: (Caltran 2004)



Semi-gravity wall with short footing heel

Lock Block/Segmental Wall:

http://www.wutecgeo.com



$$FS_{sliding} = \frac{N \cdot \tan\phi_b + W \cdot \sin\beta}{(F_s + F_q)\cos\delta}$$

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A MSE wall:

http://www.wutecgeo.com

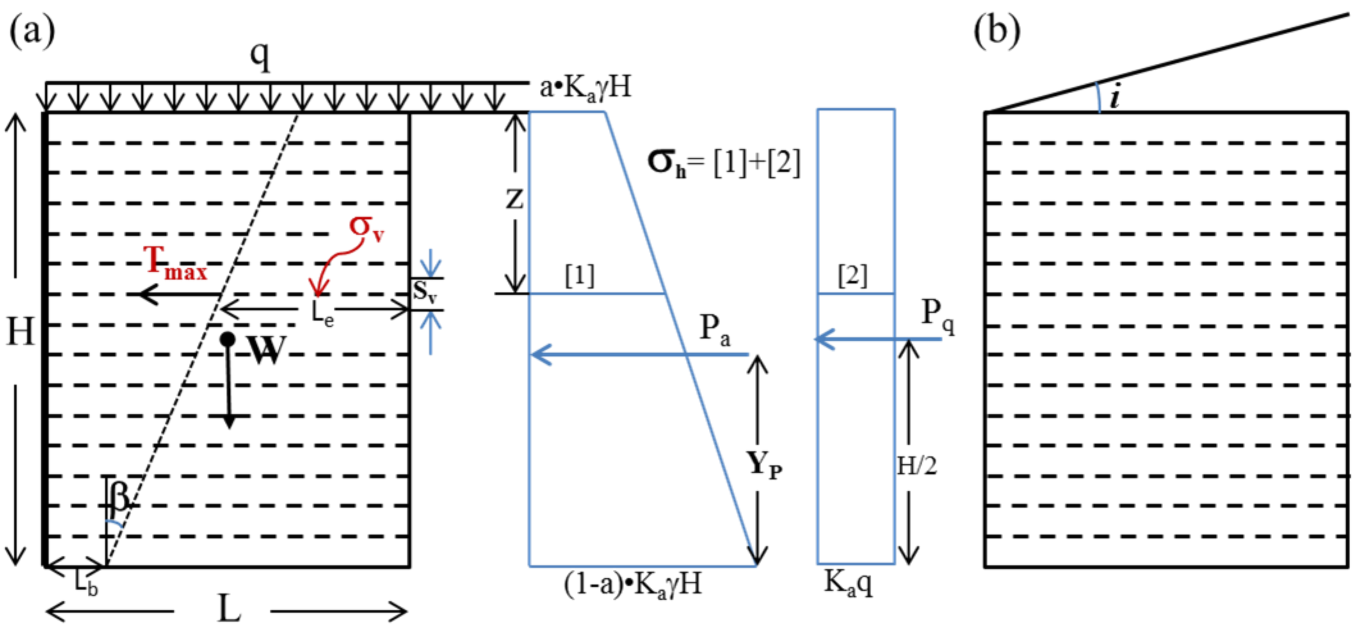

Figure 1 Force diagram for a MSE wall (a). with an surcharge (b). with a sloped backfill

A MSE Wall:
Calculation Note from:
http://www.wutecgeo.com

Notes for MSE Walls: (geo-computing)

2005.09; 2011.08; 2012.08.18

1. Factor of safety against sliding:

$$FS_{sliding} = \frac{N \cdot \eta}{P_a + P_q}$$

Lateral force from soil mass & surcharge:

$$P_a = 0.5K_a \cdot \gamma \cdot (H + L\tan i)^2$$

$$P_q = K_a \cdot q \cdot H$$

Reaction at base and weight of the wall:

$$N = W + q \cdot L$$

$$W = \gamma \cdot H \cdot L + 0.5\gamma \cdot L^2 \tan i$$

H = height of the wall at the wall face; γ = soil unit weight; L = length of wall/reinforcement ; η =base friction coefficient.

2. Factor of safety against over-turning

$$FS_{over-turning} = \frac{Mom(R)}{Mom(D)}$$, and

$$Mom(R) = (W + q \cdot L) \cdot 0.5L$$

$$Mom(D) = P_a \cdot Y_p + P_q \cdot 0.5H$$

Acting point of P_a:

$$Y_p = \frac{1+a}{3}(H + L\tan i)$$

Note: Weight of wall is assumed to be at 0.5L, even for walls with a back slope angle (i).

3. Maximum base pressure

$$P_{max} = \frac{N}{L - 2e}$$ using the Meyerhof equation, and

eccentricity: $e = \frac{Mom(D)}{N}$ (Note: e ≤ L/6)

4. Internal stability

(a). over-stressing of reinforcement (rupture):

$$FS_{rupture} = \frac{T_a}{T_{max}}$$, and

$T_{max} = \sigma_h \cdot S_V$, and the lateral stress

$$\sigma_h = [(1-a) + (2a-1)(1 - \frac{z + L\tan i}{H'})] \cdot K_a \gamma \cdot H' + K_a \cdot q$$

H' = H + L tan(i); and

z = depth to this layer of reinforcement at wall face.

(b). pullout of reinforcement:

$$FS_{pullout} = \frac{B\_strength \cdot L_e}{T_{max}}$$, and

$B\_strength = 2 \cdot \sigma_V \cdot \mu$, and overburden stress, σ_V, is:

$$\sigma_V = \gamma \cdot [(z + 0.5L\tan i) + (L - 0.5L_e)\tan i] + q$$

$L_e = L - L_b - (H - z)\tan\beta$ [Note: L_e = R_length]

The active stress wedge is defined by:

L_b = length at base of wall, β=angle to vertical

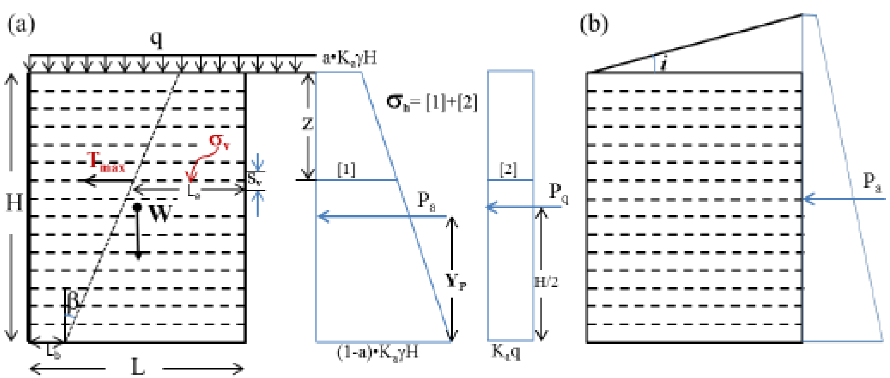

Figure 1 Force diagram for a MSE wall (a). with an surcharge (b). with a sloped backfill

Calculation for A MSE wall from: http://www.wutecgeo.com

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DESIGN CALCULATION OF MECHANICALLY STABILIZED EARTH STRUCTURE (MSE WALL)

(verified by G. Wu on 2005.09.02; 2011.08)

WALL AND SOIL DATA: [Show additional notes]

1. wall height, H (m) = 5.0 2. soil unit weight, γ (kN/m^3)= 20.0

3. length of reinforcement, L (m) = 4.0 4. back slope angle to horizontal, i(°) = 0.0

5. surcharge pressure, q (kPa) = 1.0

6. base friction coefficient, $\eta(=\tan\phi_b)$= 0.43 7. ultimate bearing capacity at base(kPa)= 400

8. soil pressure coefficient, K_a or K_{as}= 0.27 & distribution parameter (a=0 $_{\text{for triangle}}$ to a=1 $_{\text{for invert triangle}}$), a = 0.0

9. active stress wedge: length at base of wall, L_b(m) = 0.0 & angle to vertical, β(°) (e.g., 45-ϕ_{soil}/2)= 27.5

EXTERNAL STABILITY RESULTS:

weight of reinforced wall, W(kN/m) = 400.0 eccentricity, e(m) = 0.287

lateral load from soil pressures & surcharge (kN/m) = 68.850

FoS_sliding = 2.523 FoS_overturning = 6.973 FoS_bearing = 3.392

REINFORCEMENT DESIGN PARAMETERS (top-down):

10. number of layers of reinforcement = 7 with a vertical spacing (m) equal to 0.5 ,overlying

11. number of layers of reinforcement = 3 with a vertical spacing (m) equal to 0.4

12. ultimate tensile/rupture load capacity, Ta (kN/m)= 15.0

13. soil/reinforcement frictional coefficient, μ= 0.42 (Note: bonding strength = $2\sigma_V\mu$) [Computing]

INTERNAL STABILITY RESULTS:

layer_Sv=contributory wall height for the layer; Sig_h=lateral pressure on the wall at the layer; T_max = tensile load at the layer per unit wall width; R_length=length of reinforcement that provides resistance to a pull-out failure; B_strength=bonding strength between soil and reinforcement (kPa); R_pullout=pull out resistance of the reinforecement per unit width (kN/m)

Calculation Okay!

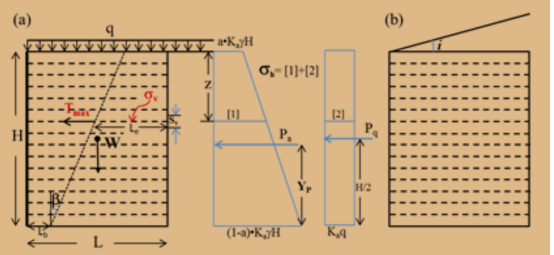

Figure 1 Force diagram for a MSE wall (a). with an surcharge (b). with a sloped backfill

DOWNLOAD CALCULATION NOTE

NOTES:

1. The backfill or back slope is considered to be dry. Thus, appropriate drainage behind the wall is an essential requirement;

2. When K_{as} is used to represent soil pressures from seismic loading, seismic inertia forces, either horizontal or vertical, on the wall have NOT been included in this calculation of wall force equilibrium.

Calculation for A MSE wall from: http://www.wutecgeo.com

Results of Calculation by Layer:

Layer	Depth(m)	Layer_Sv(m)	Sig_h (kPa)	T_max(kN/m)	FoS_rupture	R_length(m)	B_strength(kPa)	R_pullout(kN/m)	FoS_pullout
1	0.500	0.750	2.97	2.228	6.734	1.657	9.240	15.315	6.875
2	1.000	0.500	5.67	2.835	5.291	1.918	17.640	33.829	11.933
3	1.500	0.500	8.37	4.185	3.584	2.178	26.040	56.716	13.552
4	2.000	0.500	11.07	5.535	2.710	2.438	34.440	83.975	15.172
5	2.500	0.500	13.77	6.885	2.179	2.699	42.840	115.607	16.791
6	3.000	0.500	16.47	8.235	1.821	2.959	51.240	151.612	18.411
7	3.500	0.450	19.17	8.627	1.739	3.219	59.640	191.990	22.256
8	3.900	0.400	21.33	8.532	1.758	3.427	66.360	227.441	26.657
9	4.300	0.400	23.49	9.396	1.596	3.636	73.080	265.690	28.277
10	4.700	0.500	25.65	12.825	1.170	3.844	79.800	306.738	23.917
sum =		5.000		69.282					

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Section 2:

Seismic Soil Pressures on Yielding / Active Retaining Walls

<back to outline>

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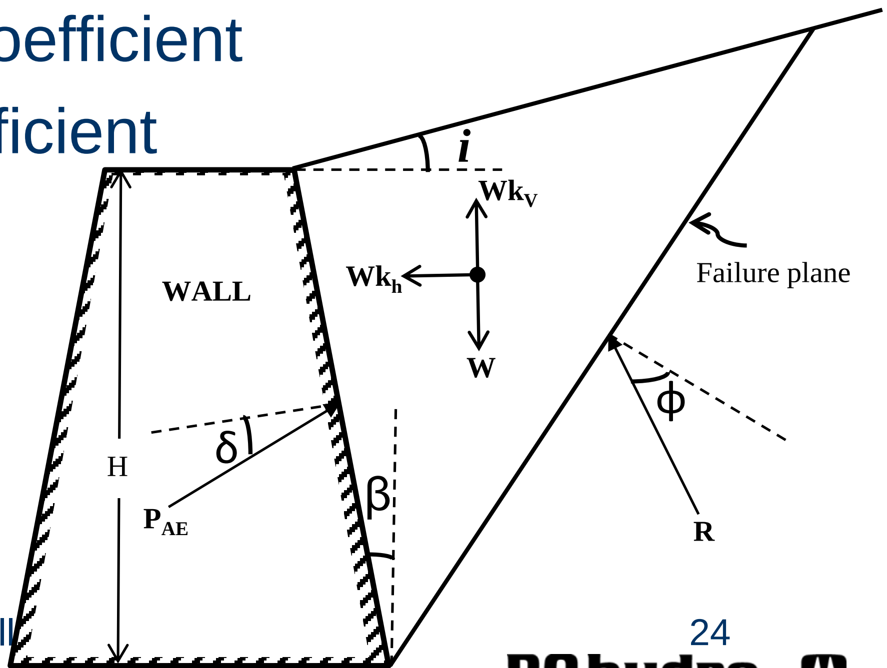
Seismic Soil Pressures on Active Walls

- Mononobe - Okabe Method:
  - extension of Coulomb's soil pressure theory by including seismic inertia forces
  - use of force equilibrium in the soil wedge
    - W – weight of soil wedge
    - k_h – horizontal seismic coefficient
    - k_v – vertical seismic coefficient
  - total active force

$$P_{AE} = ½ K_{AE} (1- k_v) \gamma H^2$$

where:



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Mononobe-Okabe Active Pressure

- Mononobe (1929) & Okabe(1926) active soil pressure coefficient

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

where:

K_{AE} = active soil pressure coefficient

Φ = angle of soil friction;

δ = angle of wall friction;

i = slope of ground surface behind the wall

β = slope of back of wall to vertical

$$\theta = \tan^{-1}(\frac{k_h}{1-kv})$$

Reference: Seed and Whitman (1970)

Mononobe-Okabe Passive Pressure

- Mononobe (1929) & Okabe(1926) passive soil pressure coefficient

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

where:

K_{PE} = passive soil pressure coefficient

Φ = angle of soil friction;

δ = angle of wall friction;

i = slope of ground surface behind the wall

β = slope of back of wall to vertical

$$\theta = \tan^{-1}(\frac{k_h}{1 - kv})$$

Notes: 1). Equation cited in Seed and Whitman (1970) is incorrect;

2). This equation for K_{PE} is much less used in practice than K_{AE}

Mononobe-Okabe equations

- Mononobe-Okabe equations: http://www.wutecgeo.com

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SOIL PRESSURE COEFFICIENTS FOR RETAINING WALLS WITH SANDY SOILS (verified by G. Wu on 2005.09; 2010.01)

STATIC CONDITION:

1. soil friction angle, $\phi(°)$ = 35
2. wall friction angle, $\delta(°)$ = 8.5
3. wall inclination angle, $\beta(°)$ = -10
4. back slope angle, $i(°)$ = 7.5

RESULTS:

Coulomb's active soil pressure coefficient, K_a = 0.207

Coulomb's passive soil pressure coefficient, K_p = 11.156

Notes:

1. Wall inclination angle, β, carries a negative sign for an inward wall and is equal to zero for a vertical wall;
2. Coulomb's $K_p$applicable ONLY for $\delta<0.4\phi$, otherwise too high.

Computing

SEISMIC CONDITION:

1. seismic coefficient (horizontal), k_h = 0.2
2. seismic coefficient (vertical), k_v = 0.1
3. wall height, H (m) = 3.0
4. soil unit weight (kN/m^3) = 20.0

RESULTS:

static+seismic active soil pressure coefficient, K_{ae} = 0.372

static+seismic passive soil pressure coefficient, K_{pe} = 9.521

static+seismic active soil pressure P_{ae} (kN/m) = 30.093

horizontal component of P_{ae} = 30.083

vertical component of P_{ae} = -0.788

static+seismic passive soil pressure P_{pe} (kN/m) = 771.167

Note: Equations are based on the Mononobe-Okabe method as described in:
Seed, H. B., and Whitman, R. V. 1970. Design of earth retaining structures for dynamic loads. Proceedings of ASCE Special Conference on Lateral Stresses, Ground Displacement and Earth Retaining Structure, Ithaca, N.Y., pp. 103-147.

An anchored sheet pile wall: considering seismic soil pressure in calculation:

http://www.wutecgeo.com

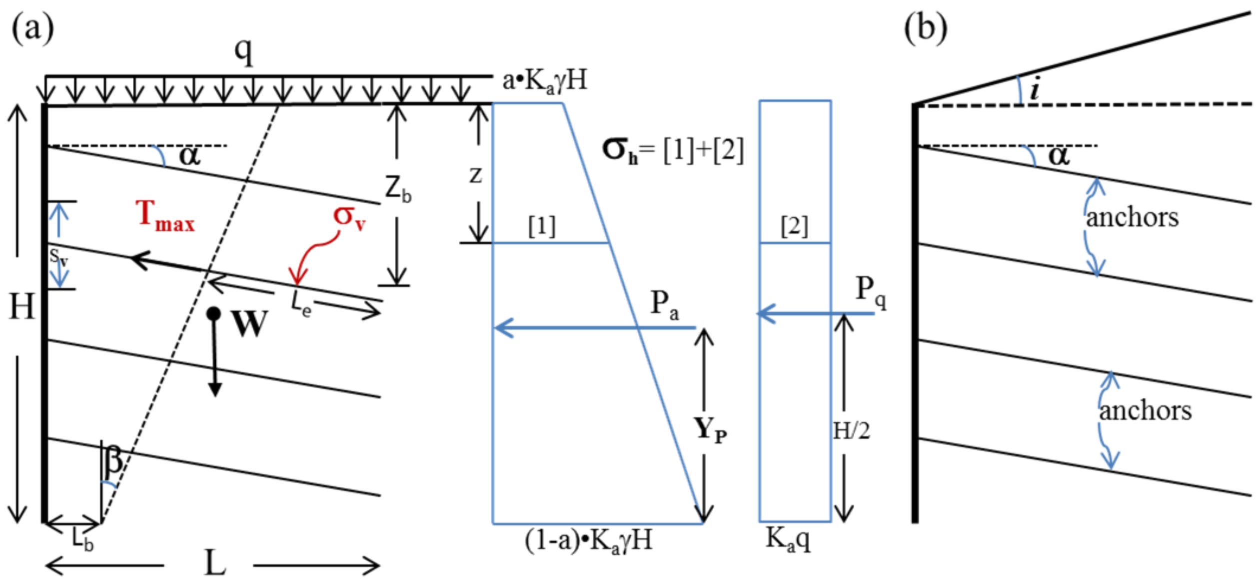

Figure 1 Force diagram for an anchored wall (a). with an surcharge (b). with a sloped backfill

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Job Title: A Demo Calculation for An Anchored Wall, including seismic soil pressure

CALCULATION OF ANCHORAGE WALL

(verified by G. Wu on 2005.09.04; 2011.08)

WALL AND SOIL DATA:

1. wall height, H (m) = 6.5 2. soil unit weight, γ (kN/m^3) = 21.0

3. length of anchors (m) = 7.5 4. dip-down angle of anchors, α (°)= 15.0

5. back slope angle to horizontal, i(°) = 0.0 6. surcharge pressure, q (kPa) = 1.0

7. base friction coefficient, η(=tan ϕ_b)= 0.43 8. ultimate bearing capacity at base (kPa)= 400.0

9. soil pressure coefficient, K_a or K_{ae}= 0.50 & distribution parameter (a=0 for triangle to a=1 for invert triangle), a= 0.4

10. unbonded zone: length at base of wall, L_b(m) = 0.0 & angle to vertical, β(°) (e.g., 45-ϕ_{soil}/2)= 27.5

EXTERNAL STABILITY RESULTS:

weight of reinforced wall, W (kN/m) = 988.9 eccentricity, e(m) = 0.686

lateral load from soil pressures & surcharge (kN/m) = 225.063

FoS_sliding = 1.903 FoS_overturning = 5.280 FoS_bearing = 2.358

ANCHOR DESIGN (top-down): Number of anchor rows (max. 9) = 4

anchor row	1	2	3	4	5	6	7	8	9
depth(m)	1.0	2.8	4.6	6.1	0.0	0.0	0.0	0.0	0.0

11. anchor horizontal spacing, s (m) = 1.5 12. anchor yield load in tension, Ty (kN) = 147.0

13. anchor bonding coeffecient, K_f = 1.6 13. grout (bonding) zone hole diameter, d (m) = 0.1

INTERNAL STABILITY RESULTS: Computing

Calculation Okay!

An anchored sheet pile wall:

http://www.wutecgeo.com

Seismic soil pressure could be included in Input box 9:

K_{AE} = 0.5 & a distribution parameter a=0.4

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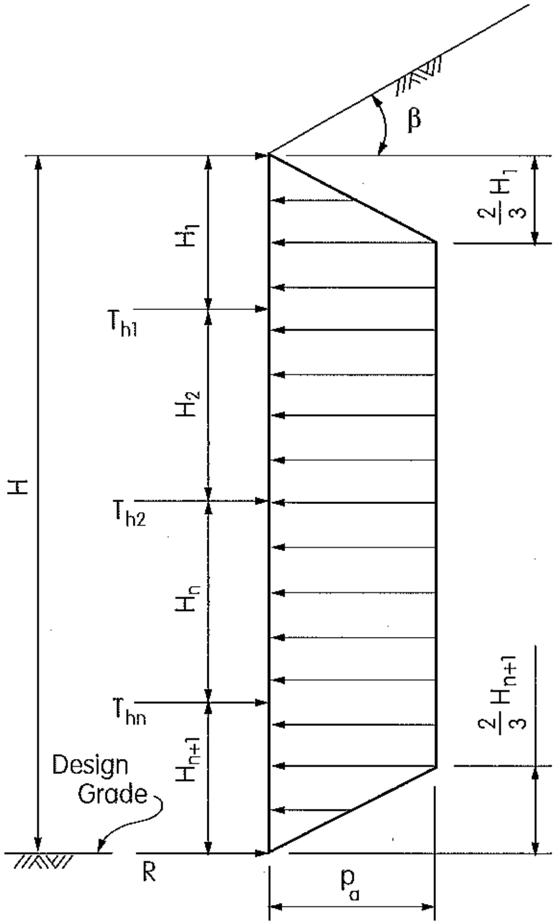
Alternative soil pressure diagram for an anchored wall (Caltran 2004)

- Anchors constructed from the top-down in sandy soils
- Wall with multiple levels of anchors
- $P_a = P_{total} /(H-1/3*H_1-1/3H_n+1)$
- P_{total} = total lateral force on the wall face to provide a factor of safety of 1.3 for the backfill equilibrium.



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Section 3.

Seismic soil pressures

on rigid wall or non-yielding wall

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Seismic Pressure on Rigid Wall

- Rigid wall or non-yielding wall
  - no relative displacements occurring between the wall and its foundation base

- Rigid wall dynamic soil pressure
  - Wood (1973) 2D elastic solution
  - Wu (1994), Wu and Finn (1999) elastic solution
  - Wu (2010) nonlinear solution for walls with sloped backfill

Seismic Pressure on Rigid Wall

- Wood (1973) elastic solution
  - Elastic backfill subject to harmonic motion
  - Dynamic solution is not readily available for engineering practice
  - Generally, the approximate solution is used in practice by ignoring the dynamic amplification:

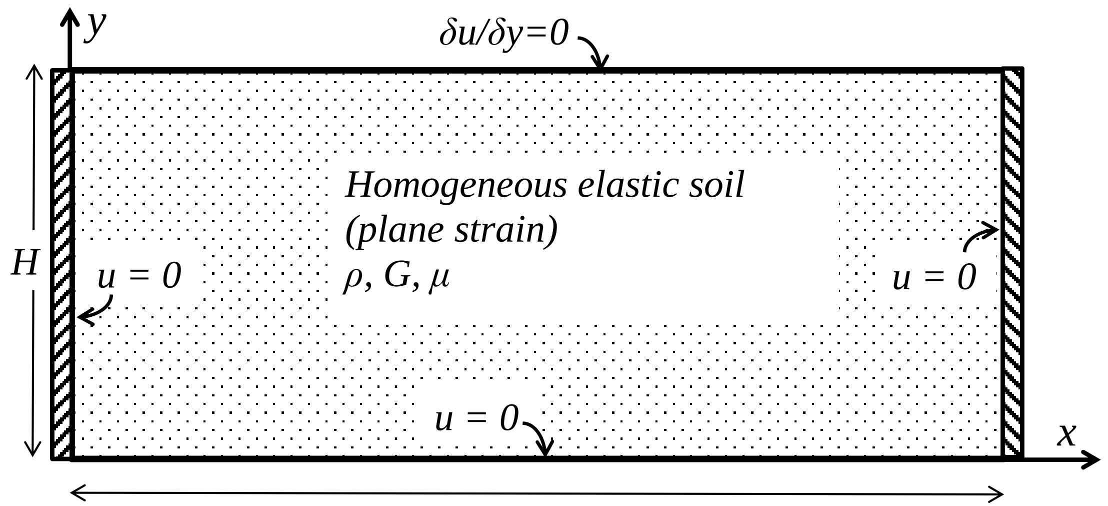
For Poisson's ratio of the elastic backfill μ=0.4,

total lateral force, $F_{sr} = \frac{1}{2}\gamma H^2 * (2k_h)$

where k_h is the seismic coefficient, and F_{sr} acts 0.63H above the base of the wall

Seismic Pressure on Rigid Wall

- Wu and Finn (1999) simplified model



- Assumption:
  - 2D model consisting of 2D displacement (u, v)
  - considering force equilibrium in x-direction but ignoring contribution from vertical displacement

i.e. $\frac{\delta v}{\delta x} = 0$

Wu (1994) Ph.D thesis;

Can Geotech J. 36: 509-522 (1999)

Rigid Wall: Wu and Finn (1999) Solution

- Dynamic force equilibrium equation:

$$G\frac{\partial^2 u}{\partial y^2} + \frac{2}{1-\mu} G \frac{\partial^2 u}{\partial x^2} - \rho \frac{\partial^2 u}{\partial t^2} = \rho \ddot{u}_b(t)$$

Where:

G = shear modulus;

ρ= mass density; and

$\frac{\delta^2 u_b(t)}{\delta t^2}$ = base acceleration

- Lateral stress

$$\sigma_x = \frac{2}{1-\mu} G \frac{\partial u}{\partial x}$$

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Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution:
  - Total dynamic force on the wall

$$Q(t) = \frac{2G}{1-\mu} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{16 f_{mn}(t)}{\pi^2 (2n-1)^2 L/H}$$

$$\ddot{f}_{mn}(t) + 2\lambda\omega_{mn} \cdot \dot{f}_{mn}(t) + \omega^2{}_{mn} \cdot f_{mn}(t) = -\ddot{u}_b(t)$$

where

λ = model damping ratio, and the system frequency

$$\omega_{mn} = \sqrt{\frac{G}{\rho}\left(b_n^2 + \frac{2}{1-\mu} a_m^2\right)}$$

$$a_m = \frac{(2m-1)\pi}{2L} \qquad m = 1,2,3,\ldots.$$

$$b_n = \frac{(2n-1)\pi}{2H} \qquad n = 1,2,3,\ldots.$$

Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution:
  - For harmonic base acceleration (A_{max}, ω):

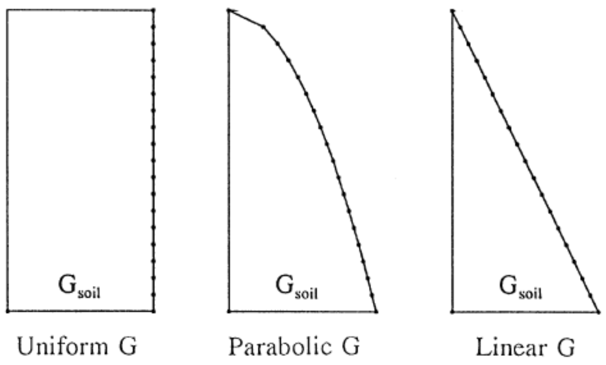
$$f_{mn}(t) = -\frac{A_{max}}{(\omega_{mn}^2 - \omega^2) + 2i \cdot \lambda\, \omega_{mn} \cdot \omega} \cdot e^{i\omega t}$$

  - For infinitely long period base acceleration (i.e., static solution, ω→0)

$$Q^{st} = \frac{2G}{1-\mu} \frac{16\, A_{max}}{\pi^2 L / H} \sum_{m=1}^{\infty} \sum_{n=1}^{\infty} \frac{1}{(2n-1)^2\, \omega_{mn}^2}$$

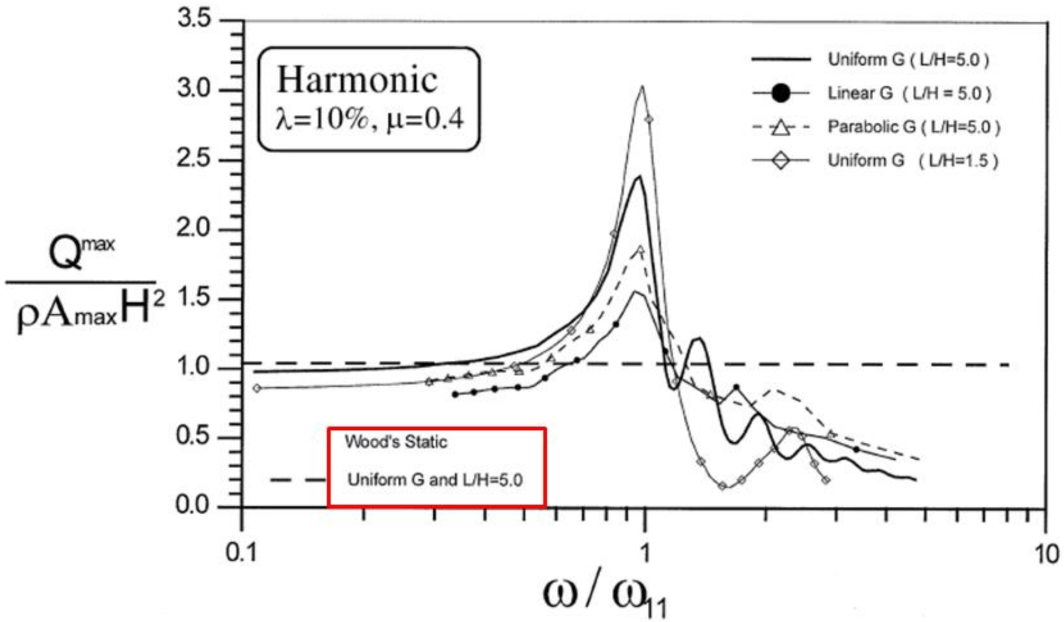
Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution:
  - Three typical soil profiles
  - Closed-form solution for uniform G
  - Finite element solution for parabolic and linear G
  - Finite element method:
    - 6-node quadratic element
    - Elastic analysis
    - Modal analysis performed
    - A constant modal damping used
    - Calibrated against the closed-form solution to be exact (no loss of accuracy)



Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution: HARMONIC
  - Steady-state peak dynamic forces
    - Dynamic amplification factor at resonance ranges from 2.4 to 3.5 for the uniform G
    - Wood (1973): a good estimation with ω/ω_{11}<0.5



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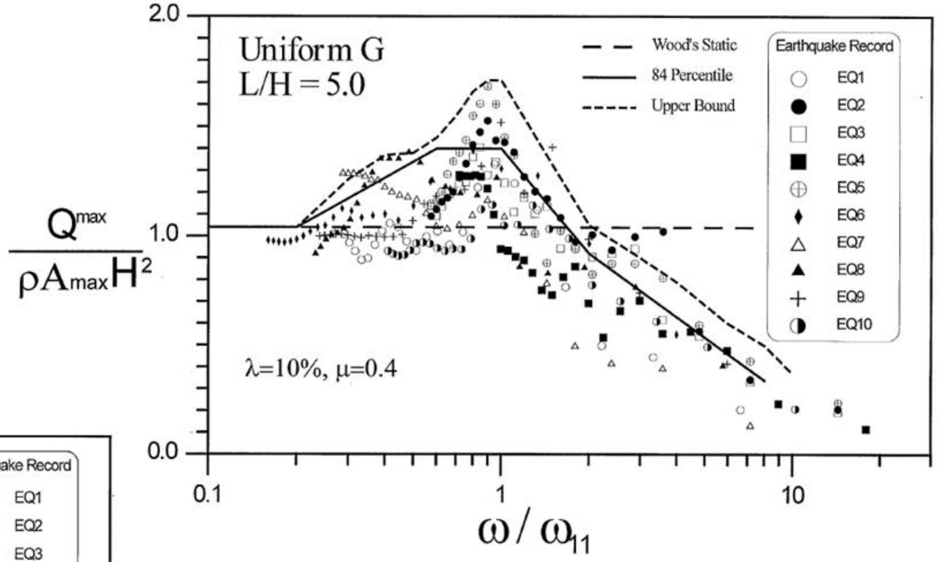
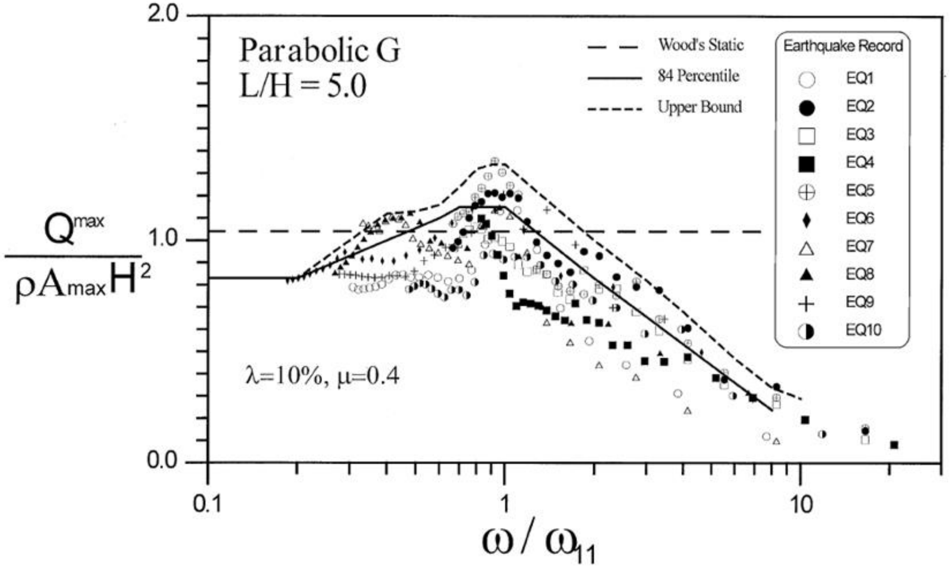
Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution: SEISMIC

  For each soil profile,

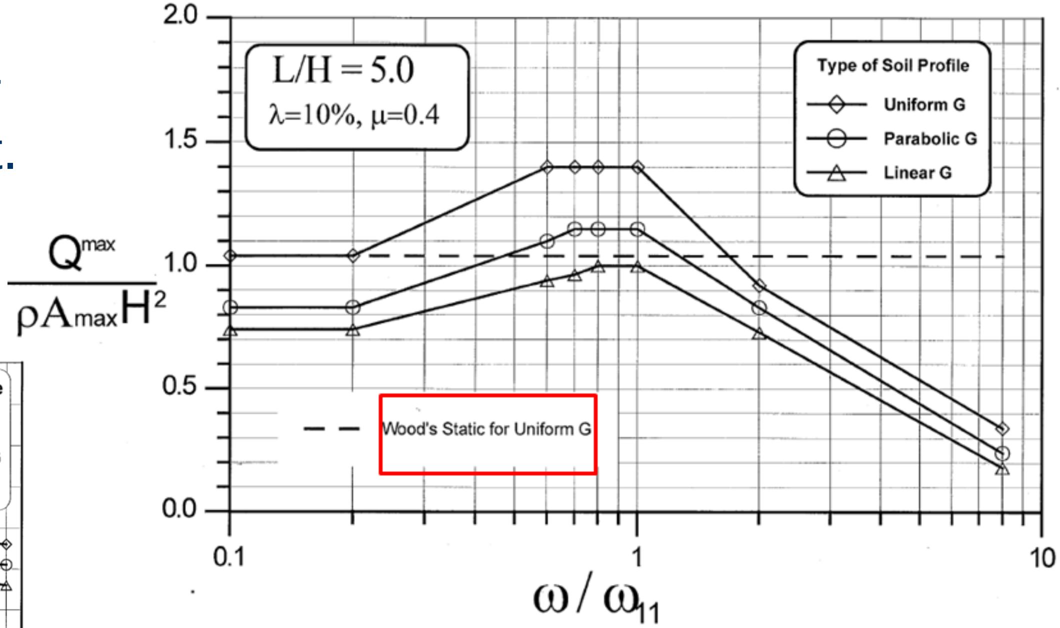
  10 x 25 = 250 runs

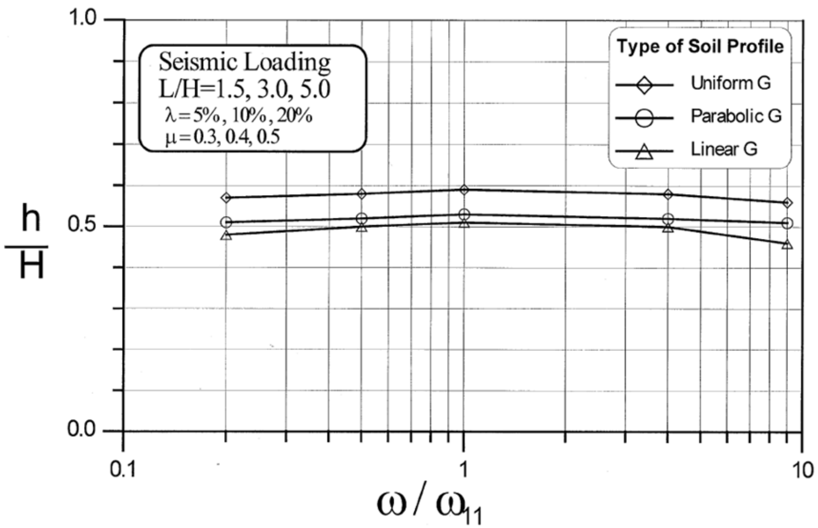




Rigid Wall: Wu and Finn (1999) Solution

- Dynamic soil pressure solution: SEISMIC
  - Peak dynamic forces: 84th percentile design chart:
    - Dyn amplification ~1.4
    - Wood (1973) good est. with $\omega/\omega_{11}<0.2$





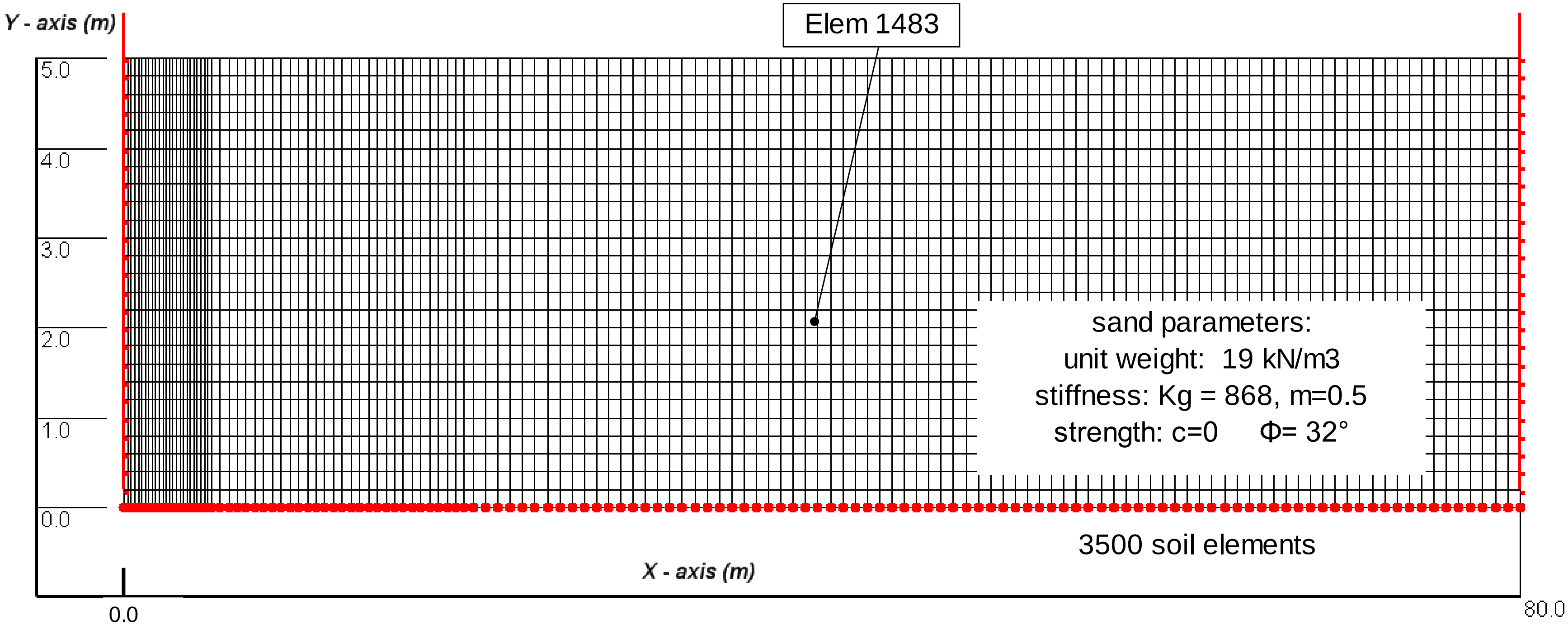
←Acting height of dynamic force

Seismic Pressure on Rigid Wall

- Wu (2010) nonlinear finite element results for walls with sloped backfill
  - VERSAT-2D dynamic time history analyses
  - Comparison with Wood (1973), Wu (1994), Wu and Finn (1999) for horizontal backfills
  - Provide soil pressures for walls with sloped backfills
    - 2H:1V (27º) sloped backfills
    - loose sand (φ=32°) and dense sand (φ=40°)
    - three levels of ground motions with a nominal PGA of 0.26g, 0.48g and 0.71g (8 records)

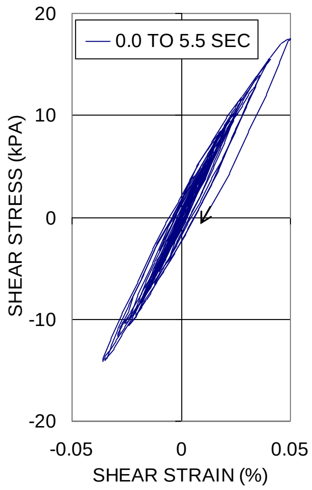
Rigid Wall: Wu (2010) for sloped backfills

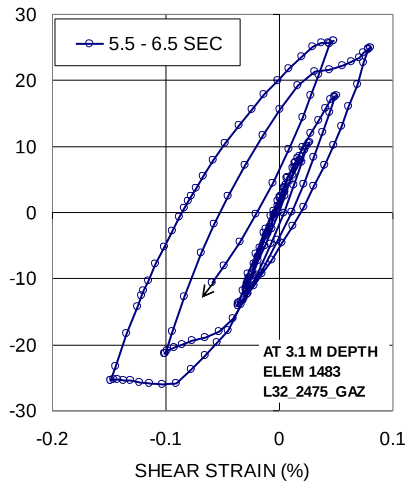
- VERSAT-2D model for nonlinear time history analyses – horizontal backfill

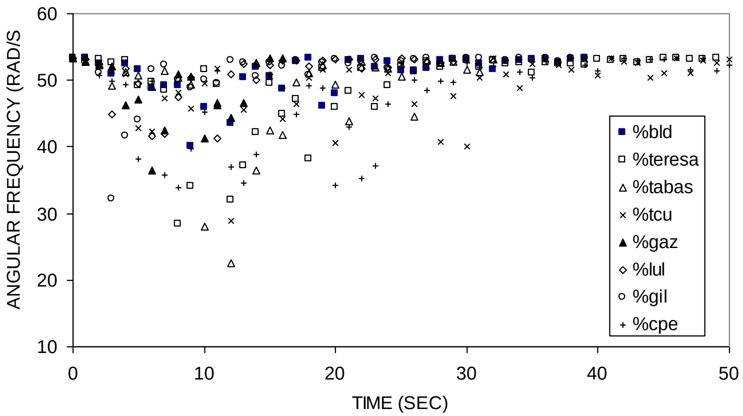


Rigid Wall: Wu (2010) for sloped backfills

- VERSAT-2D nonlinear response
  - Nonlinear Hysteretic shear stress and strain traces at Element 1483 (*record %gaz,* 048g)



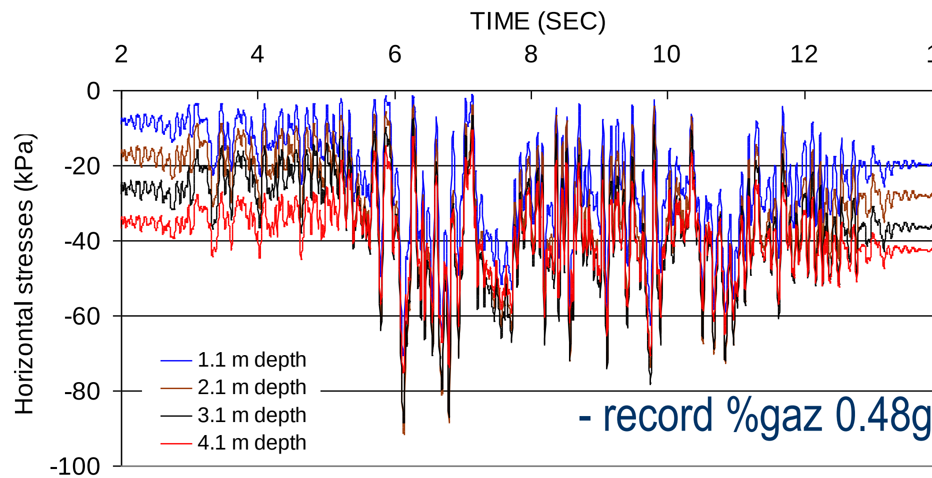




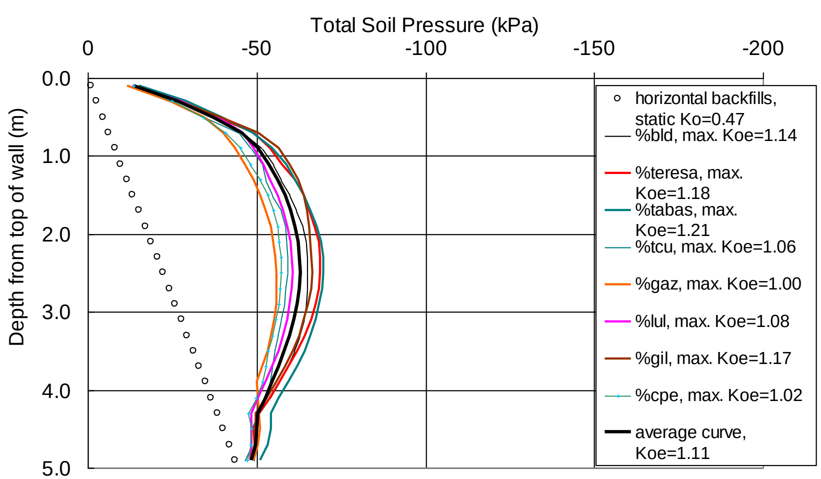
  - Time histories of 1st mode frequency of wall-soil system for the 8 ground motions (0.48g)

Rigid Wall: Wu (2010) for sloped backfills

- VERSAT-2D dynamic soil pressures
  - Time histories of soil pressures along the wall



  - Peak soil pressures along the wall (0.26g)



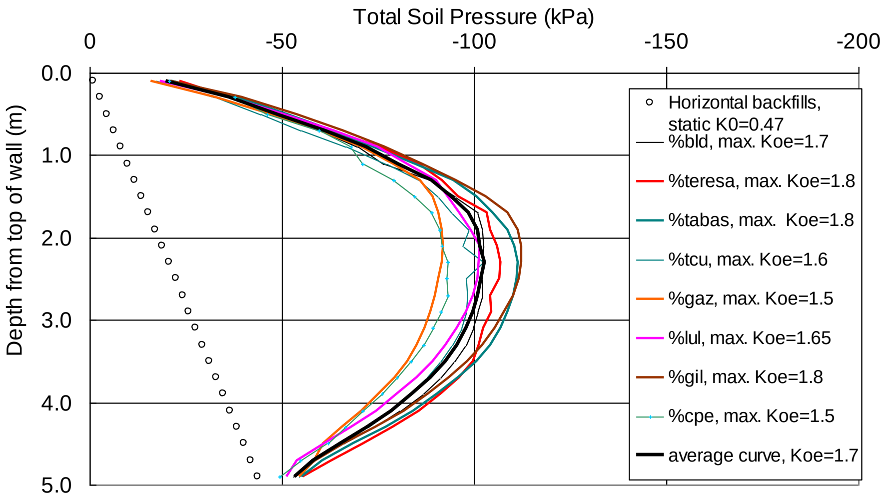
$P_{0E} = \frac{1}{2} K_{0E} \gamma H^2$

K_{0E} a soil pressure coefficient for rigid wall (*new*);

← For PGA=0.26g, K_{0E} =1.0 to 1.21

Rigid Wall: Wu (2010) for sloped backfills

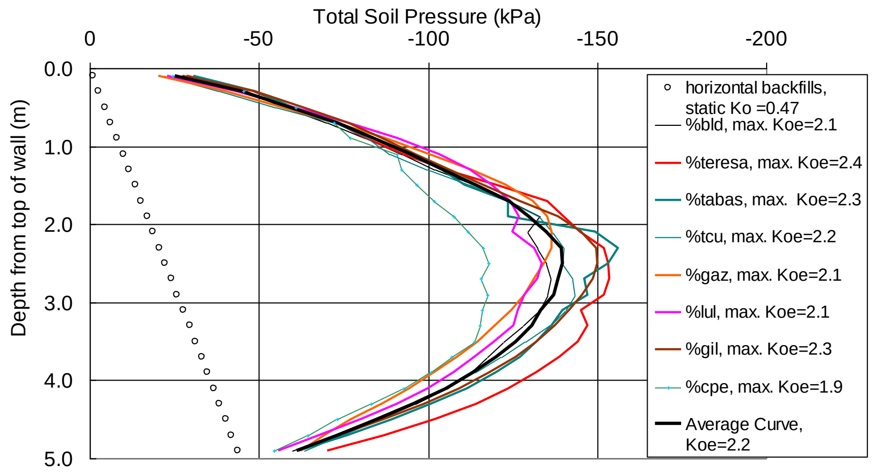
- VERSAT-2D peak dynamic pressures



$P_{0E} = ½\, K_{0E}\, \gamma H^2$

K_{0E} a soil pressure coefficient for rigid wall (*new*);

For PGA=0.48g, K_{0E} =1.5 to 1.8



For PGA=0.71g, K_{0E} =1.9 to 2.4

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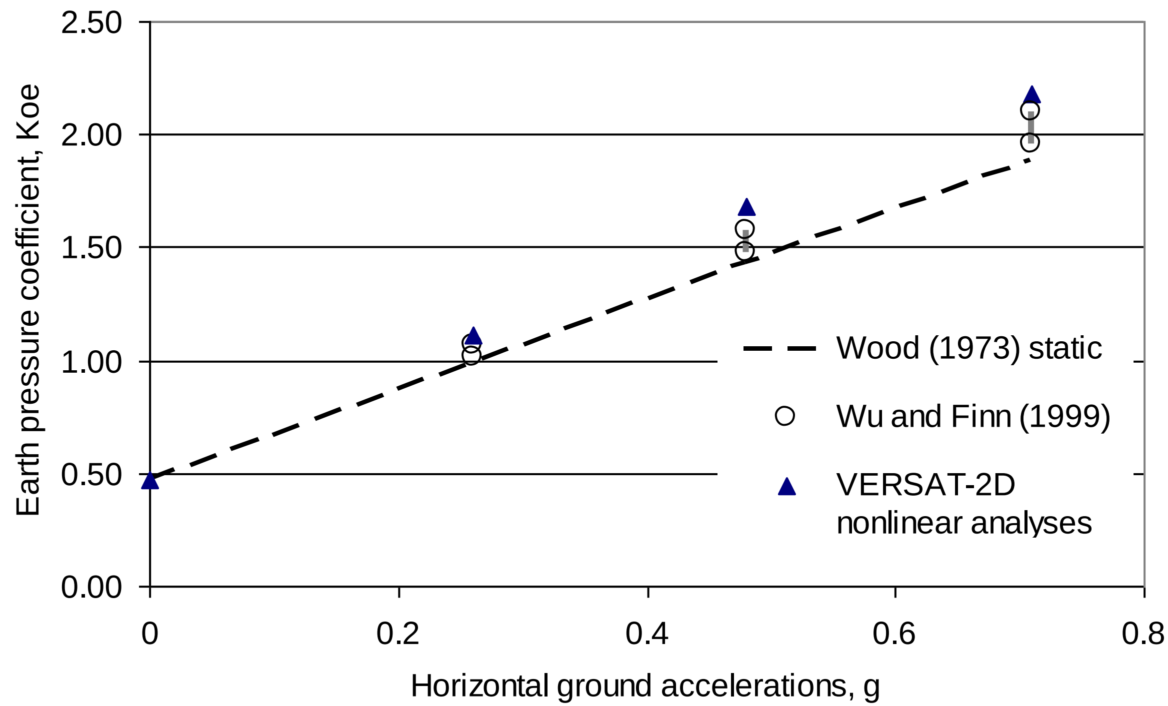
Rigid Wall: Wu (2010) for sloped backfills

Comparison of K_{OE} from VERSAT-2D nonlinear analyses, Wood (1973) and Wu and Finn (1999) for horizontal backfills

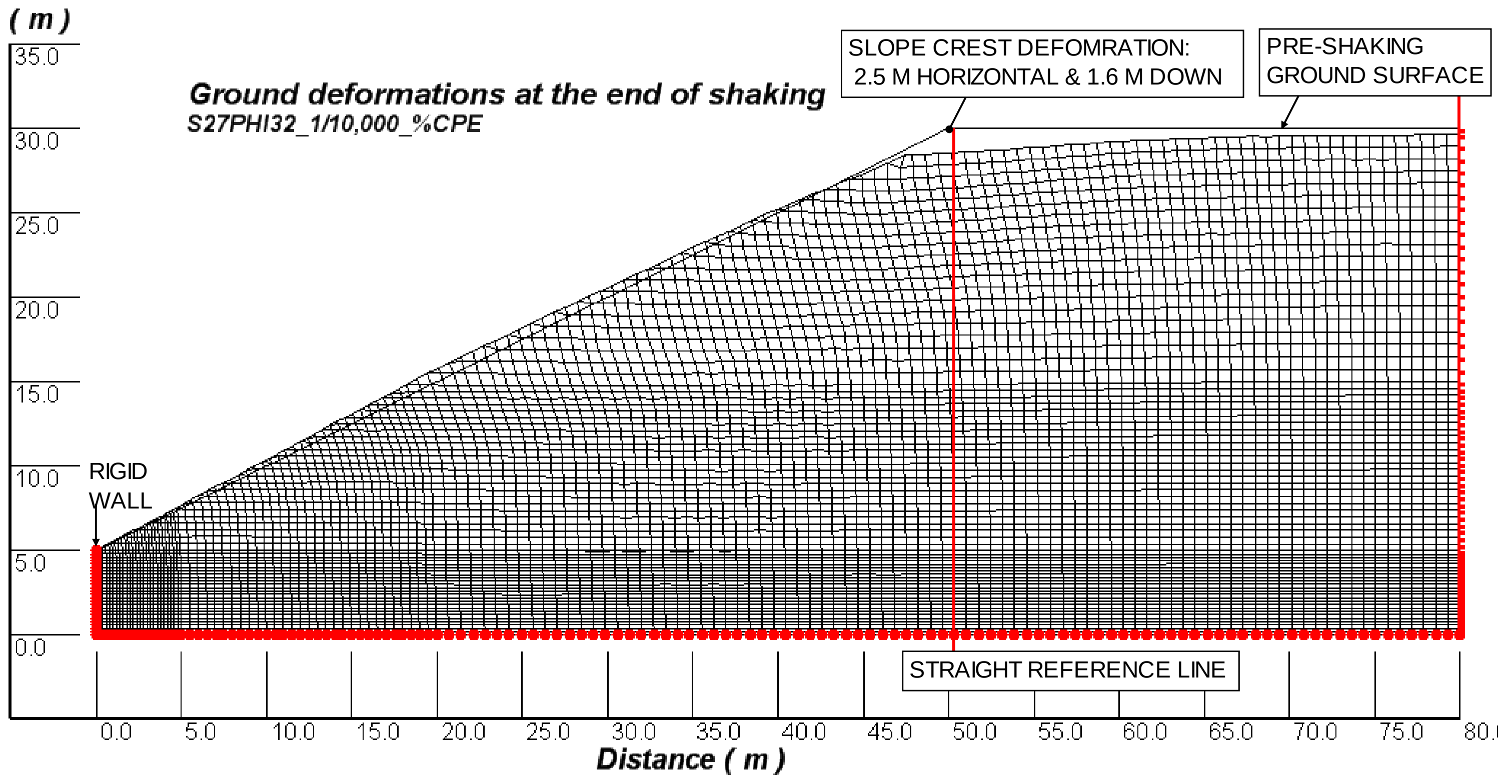


$P_{0E} = ½ K_{0E} \gamma H^2$

K_{0E} a soil pressure coefficient for rigid wall (*new*), including contribution from both static and seismic soil pressures.

Rigid Wall: Wu (2010) for sloped backfills

Deformed 2H:1V sloped soil backfill (φ=32°) under 0.71g



Rigid Wall: Wu (2010) for sloped backfills

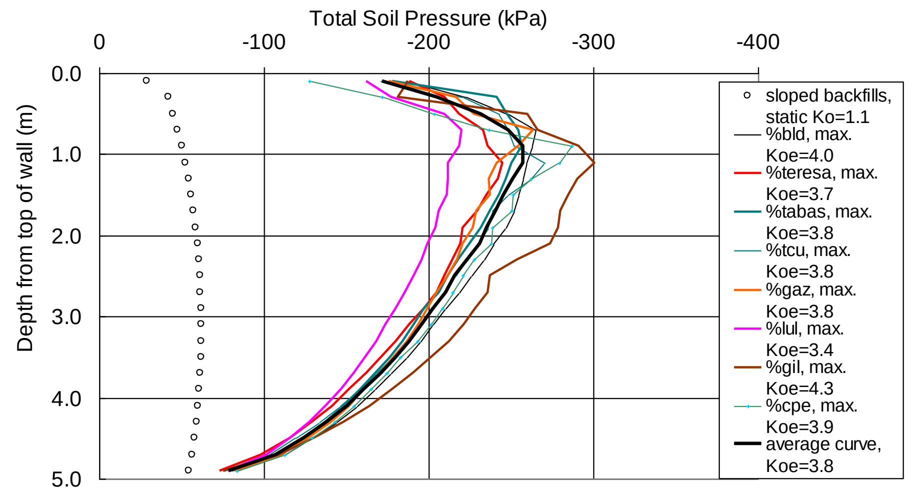
Maximum soil pressures on walls with 2H:1V sloped backfills (φ=40°)

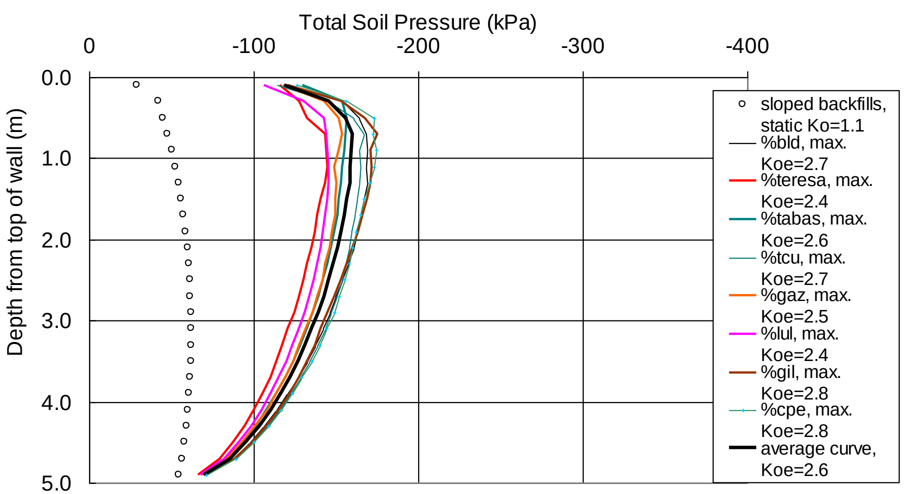
$P_{0E} = ½ K_{0E} \gamma H^2$

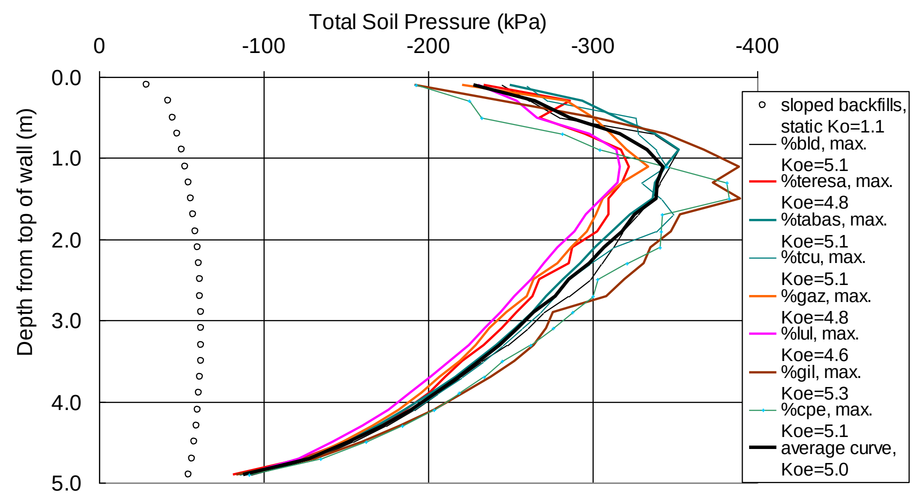
For PGA=0.26g, K_{0E} =2.4 to 2.8 *(2.6)*

For PGA=0.48g, K_{0E} =3.4 to 4.3 *(3.8)*

For PGA=0.71g, K_{0E} =4.6 to 5.3 *(5.0)*

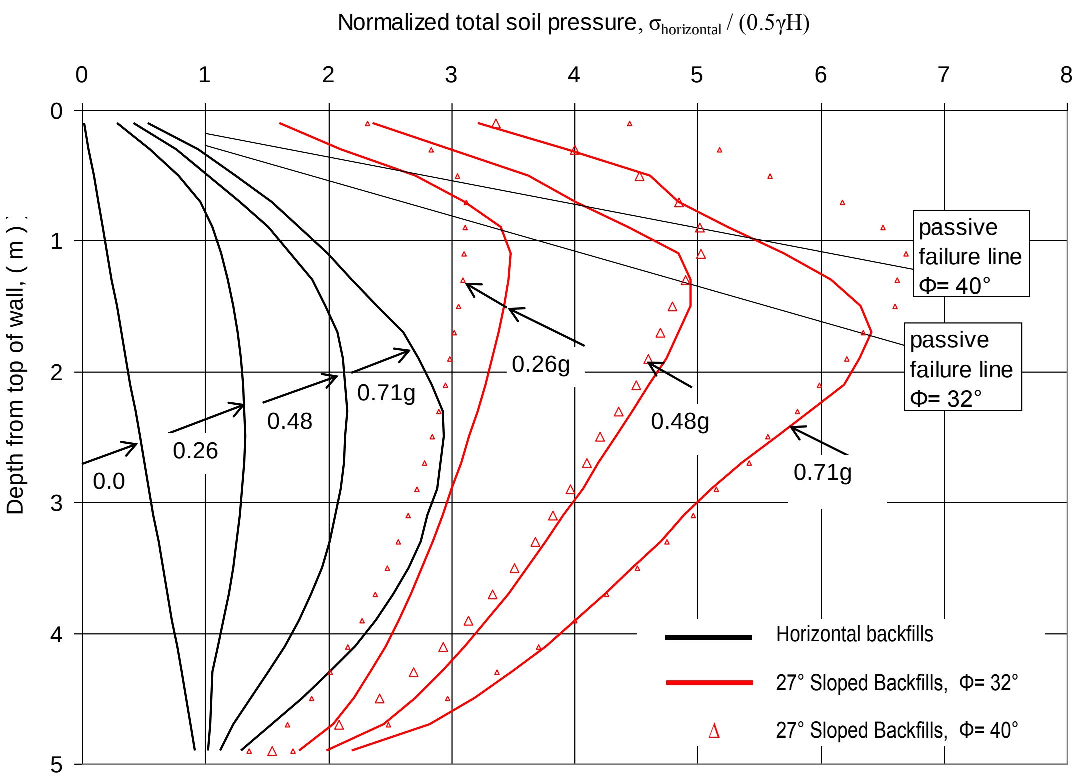






Rigid Wall: Wu (2010) for sloped backfills

Comparison of K_{0E} for horizontal & 27º (2H:1V) sloped backfills (ϕ=32º and 40º)



$P_{0E} = ½ K_{0E}\, \gamma H^2$

K_{0E} is a soil pressure coefficient, proposed for rigid wall (*new*)

A_{max}	Hori. ϕ=32º	2H:1V ϕ=32º	2H:1V ϕ=40º
0.0g	0.47	1.5	1.1
0.26g	1.1	2.8	2.6
0.48g	1.7	3.8	3.8
0.71g	2.2	4.8	5.0

Basic static behaviour of retaining walls

References:

1. CalTrans 2004 August, Bridge Design Specifications Section 5 - Retaining Walls
2. Mononobe, N., and Matuo, H. 1929. On the determination of earth pressure during earthquakes. Proceedings of World Engineering Conference, Vol. 9.
3. Okabe, S. 1926. General theory of earth pressure. Journal, Japanese Society of Civil Engineers, Vol. 12, No. 1.
4. Scott, R.F. 1973. Earthquake-Induced Earth Pressures on Retaining Walls. Proceedings of the 5th World Conference on Earthquake Engineering, Rome, Italy.
5. Seed, H. B., and Whitman, R. V. 1970. Design of earth retaining structures for dynamic loads. Proceedings of ASCE Special Conference on Lateral Stresses, Ground Displacement and Earth Retaining Structure, Ithaca, N.Y., pp. 103-147.
6. Wood, J. H. 1973. Earthquake-induced soil pressures on structures. Ph.D thesis , the California Institute of Technology, Pasadena, California, USA.
7. Wu, G. 2010. Seismic soil pressures on rigid walls with sloped backfills. Proceedings of the 5th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, California, US, May 24-29.

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Section 4:

Seismic pressures on displacing walls

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Seismic Pressures on Displacing Walls

- Seismic pressures on displacing walls
  - Newmark (1965) method for displacements on slopes
  - Bray & Travasarou (2007)
  - Richards & Elms (1979) method for estimating displacements for gravity walls
    - Limit equilibrium method for yield acceleration a_y
    - Kramer (1996) equation for yield acceleration a_y

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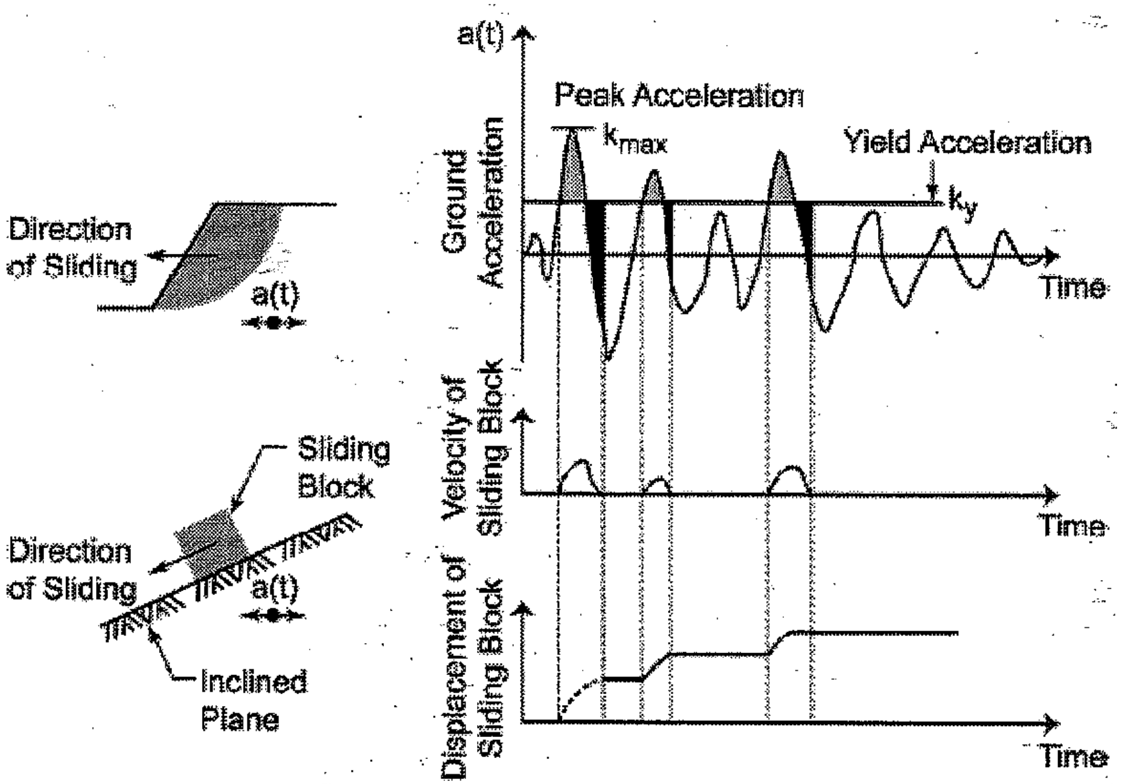
Seismic Pressures on Displacing Walls

Newmark (1965) rigid block for earthquake induced displacements on slopes

- Rigid block on an inclined plane



Seismic Pressures on Displacing Walls

Newmark (1965) rigid block for earthquake induced displacements on slopes

Approximately for 0.15<N/A <0.5

$$\Delta = \frac{V^2}{2gN}\frac{A}{N}$$

Where:

g = gravity acceleration (m/s^2, in/s^2)

N = yield acceleration in g (or K_y)

A = peak acceleration in g (or K_h)

V = peak velocity (m/s, in/s)

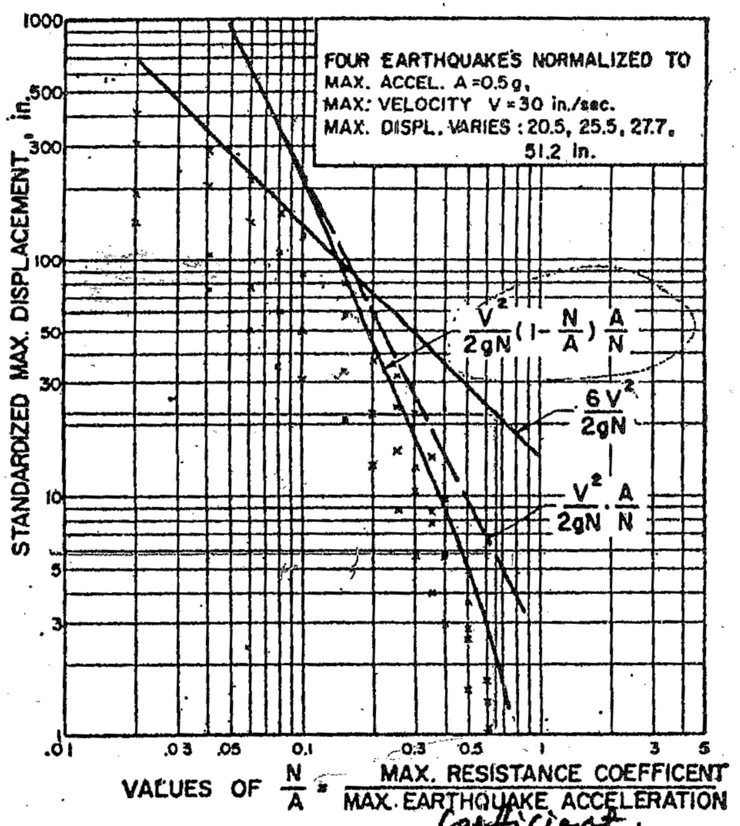


Fig. 22. Standardized displacement for normalized earthquakes (unsymmetrical resistance)

Limitations:

- Only based on 4 earthquake records
- Characteristics of earthquake not taken into account

Seismic Pressures on Displacing Walls

Bray & Travasarou (2007) method for estimating displacements on slopes:

- Based on Newmark's approach
- An update of Makdisi and Seed (1978) method to include large data base of earthquake records and the concept of probability of zero-displacement.
- The amount of non-zero displacement is estimated from:
  - Yield acceleration coefficient, K_y
  - Initial fundamental period of the sliding mass, T_s
  - Spectra acceleration of the input ground motion at $S_a(1.5T_s)$
  - Earthquake magnitude, M

Reference: ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 133, No. 4, April 1, 2007. pp. 381–392

Seismic Pressures on Displacing Walls

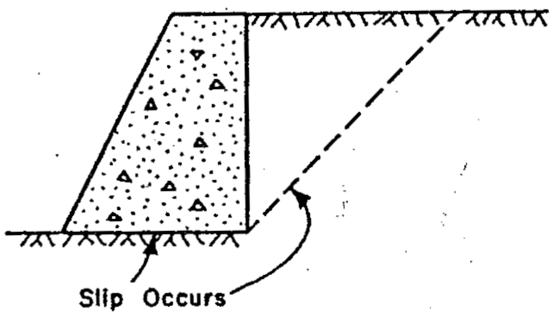
- Richards & Elms (1979) equation for estimating seismic displacements on gravity retaining walls

A variation of Newmark's equation:

$$\Delta = 0.087 \frac{V^2}{Ag} \left(\frac{A}{N}\right)^4$$

Or:

$$\Delta = 0.174 \left(\frac{A}{N}\right)^2 \Delta_{Newmark}$$

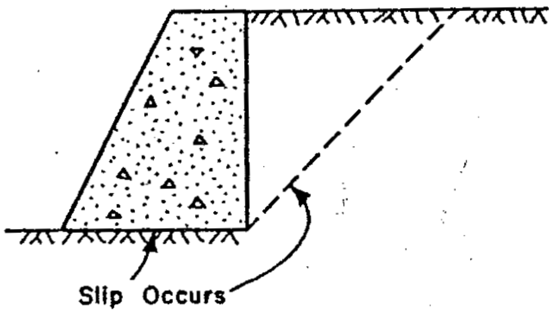


Therefore:

- Displacement from Richards & Elms (R&E) represents the upper bound, especially for N/A=yield(acce)/peak(acce) < 0.4
- At N/A=0.1, R&E displacement is about 17.4 times that by Newmark
- Be aware of these limitation when applying this method

Seismic Pressures on Displacing Walls

- Richards & Elms (1979) equation for estimating seismic displacements on gravity retaining walls
- Two methods for estimating $a_y = k_y \cdot g$ (or N•g)
  - Method 1: Limit equilibrium method
    - » Slip surface to include both the wall base (frictional) and the soil mass
  - Method 2: trial and error method
    - P_{AE} from MO equation, seismic coefficient k_y decreasing from K_h (peak)
    - Obtain acceleration a_y of the wall by applying force equilibrium at the wall base, including soil pressure P_{AE}
    - At convergence, $a_y = k_y \cdot g$
    - Also see Kramer (1996)



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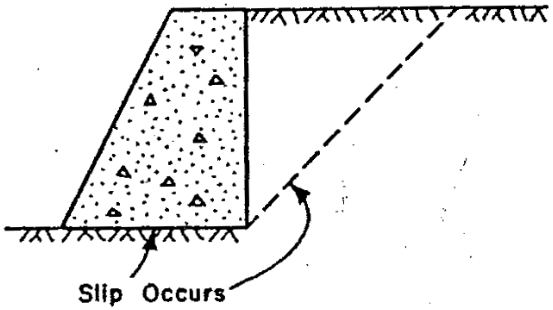
Seismic Pressures on Displacing Walls

Design seismic upgrade of gravity walls for displacements using Richards & Elms (1979) equation

Step 1: Estimating yield acceleration N

based on the design displacement

Δ and peak acceleration coefficient A :



$$\mathrm{N} = A \sqrt[4]{\frac{0.087}{\Delta} \frac{V^2}{Ag}}$$

Be aware of limitations for the equation

Step 2: Calculate P_{AE}

Step 3: Design the required upgrade (adding weight, or increasing base resistance) to provide force equilibrium at the wall base, with a factor of safety (such as 1.10).

Step 4: Check the design with time history disp analysis, if needed.

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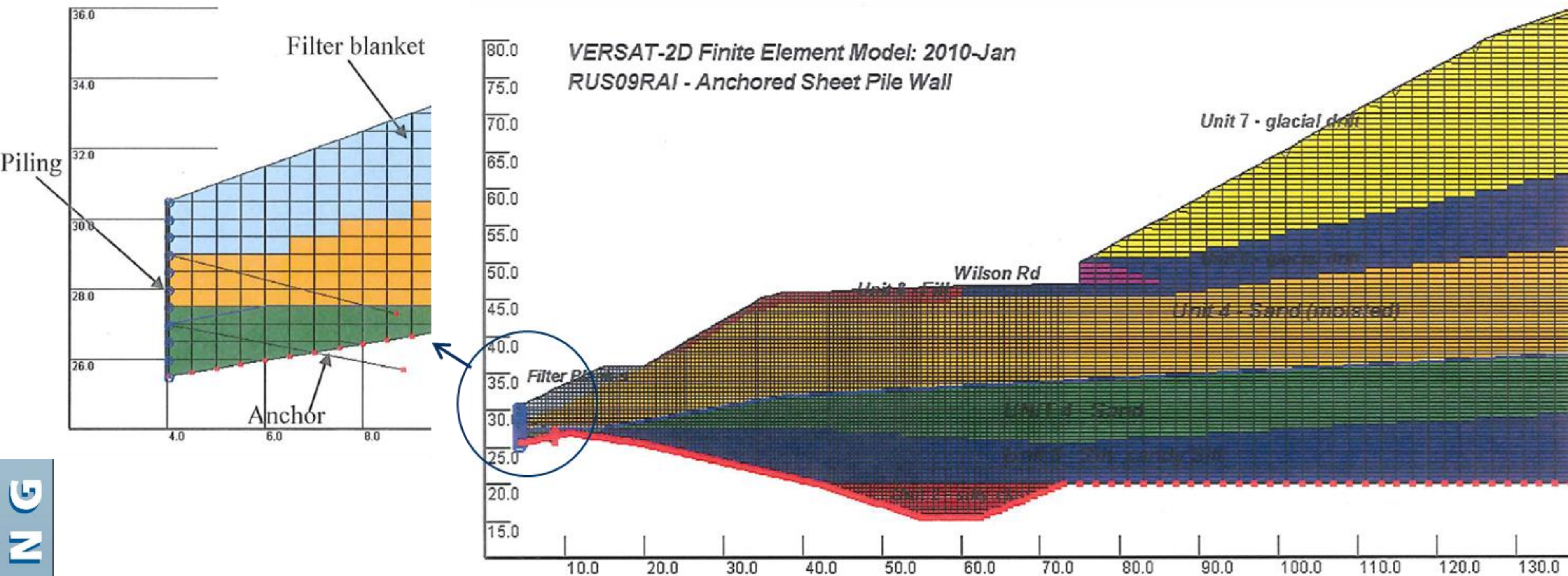
Seismic Pressures for Displacing Walls by Dynamic Finite Element Analysis: **VERSAT-2D dynamic analysis**

1. **Ruskin Dam Right Abutment Upgrade (completed)**
   - Lower Slope Retaining Wall - **anchored sheet pile wall (in 2 slides)**
   - Upstream gravity wall
   - Downstream concrete wing wall

2. **John Hart Dam training walls (H=7.6 m) – in progress**
   – disp. up to 150 mm (2%) without anchors;
   – disp. Up to 60 mm (0.8%) with anchors

Displacing Walls by VERSAT-2D Analysis



Ruskin Dam Lower Slope Retaining Wall

anchored sheet pile wall

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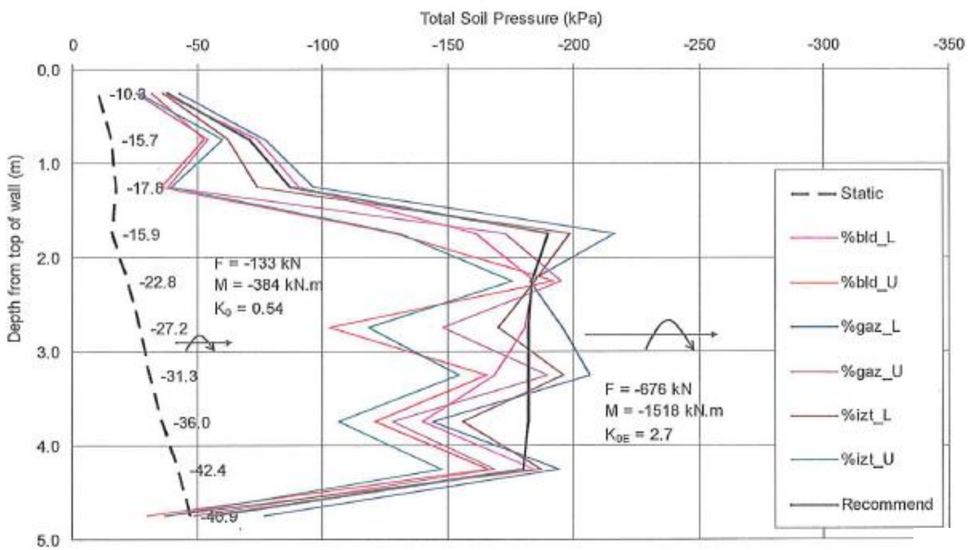
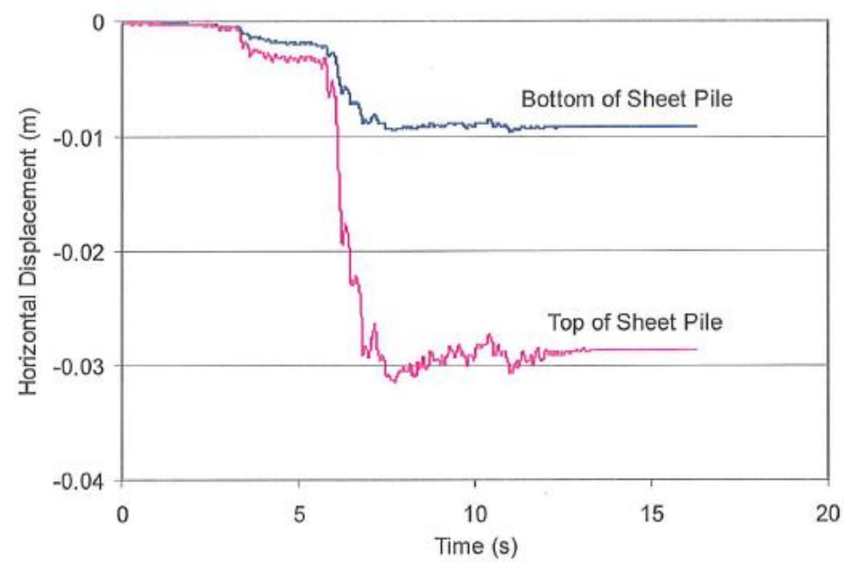
Displacing Walls by VERSAT-2D Analysis



Below:
Displacement response TH at top/bottom of the sheet pile wall
for 1 time history (GAZ-00)

Above:
Seismic soil pressures along the sheet pile wall for 3 time histories (upper and lower strength para.)



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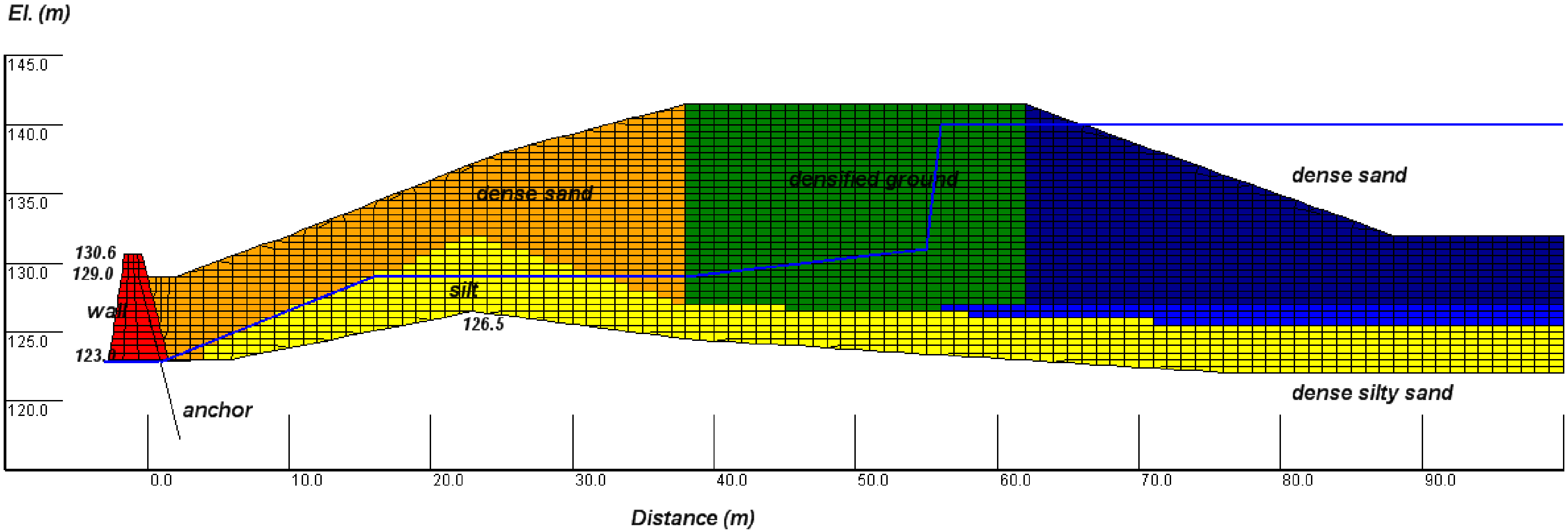
Displacing Walls by VERSAT-2D Analysis



VERSAT-2D model for dynamic analyses of **John Hart Dam training walls (H=7.6 m) – in progress**

- disp. up to 150 mm (2%) without anchors;
- disp. Up to 60 mm (0.8%) with anchors

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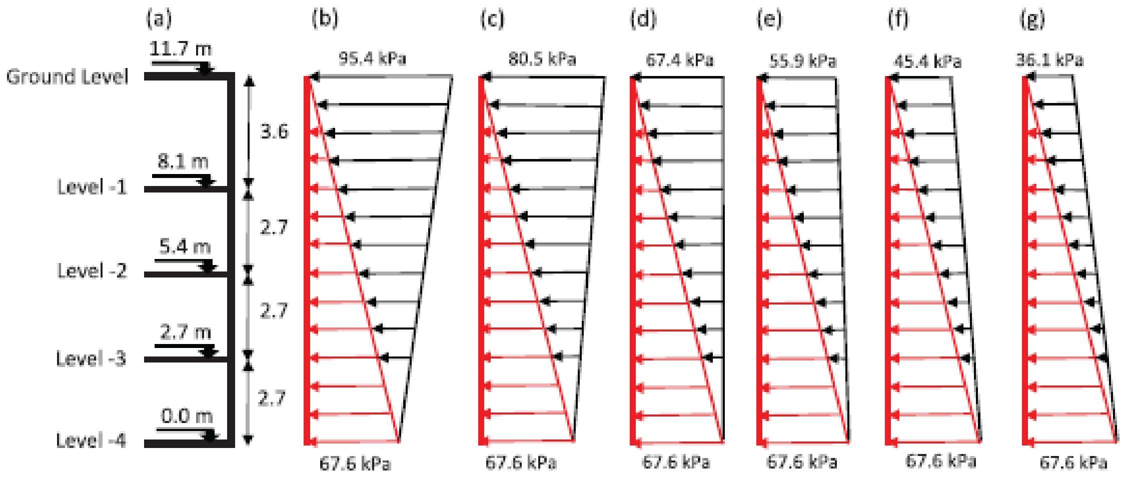
Section 5.
Seismic Soil Pressure on Basement Wall



Reference: Taiebat et al. (2014) CGJ Vol.51 pp.1004-1020 for soil and wall material properties

Floor heights in the 4-level basement wall with 3.6 m top storey, and the calculated lateral earth pressure distributions using the M-O method with:

(b) 100% PGA, (c) 70% PGA, (d) 60% PGA, and (e) 50% PGA, where PGA=0.46g.

BC Hydro
Power smart

Background

omitted

BC Hydro
Power smart

Reference:

1. Amirzehni et al., 2015 on 2015 ICEGE Conference
2. Taiebat et al. (2014) CGJ Vol.51 pp.1004-1020 for soil and wall material properties
3. Finn W.D.Liam and Wu, Guoxi, 2013. Dynamic Analyses of an Earthfill Dam on Over-Consolidated Silt with Cyclic Strain Softening. Keynote Lecture, Seventh International Conference on Case Histories in Geotechnical Engineering, Chicago, US, April 29 - May 4
4. Wu, G. (2010 San Diego Int. Conference) "Seismic Soil Pressures On Rigid Walls With Sloped Backfills" http://www.wutecgeo.com/pubwu.aspx & http://www.wutecgeo.com/pubv2d.aspx
5. Wu, G., and Finn, W.D.L. 1999. Seismic lateral pressures for design of rigid walls. Canadian Geotechnical Journal, 36: 509-522
6. Guoxi Wu 1994. Dynamic soil-structure interaction: Pile foundations and retaining structures. Ph.D. thesis, Department of Civil Engineering, the University of British Columbia, Vancouver
7. Jaw-Nan (Joe) Wang, 1993, Seismic Design of Tunnels, 1991 William Barclay Parsons Fellowship Parsons Brinckerhoff, Monograph 7

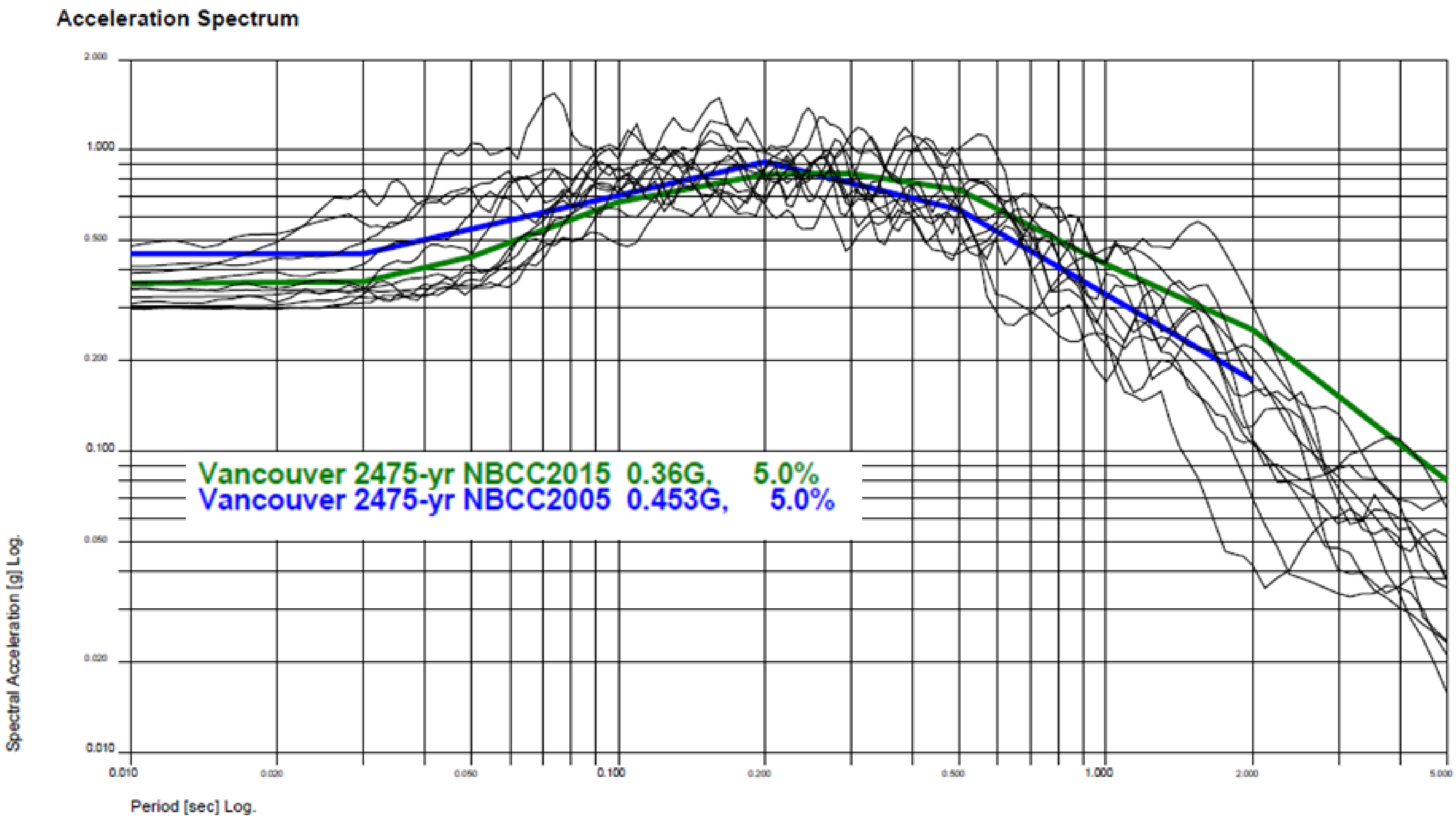
Meta data of 11 scaled horizontal input acceleration time histories (THs):

(average PGA = 0.35 g)

Record #	Record Name & Component	Short Name	Duration (s)	PGA (g)	PGV (m/s)	PGD (m)	AI (m/s)
1	Chi-Chi Taiwan 9/20/1999 TCU071 EW	TCU071	90.0	0.302	0.299	0.092	3.044
2	Northridge-01 1/17/1994 LA - Chalon Rd 70	CHL070	31.1	0.341	0.304	0.058	1.563
3	Northridge-01 1/17/1994 LA - Baldwin Hills 360	BLD360	40.0	0.304	0.316	0.096	2.021
4	Loma Prieta 10/18/1989 San Jose - Santa Teresa Hills 225	SJTE225	50.0	0.324	0.331	0.272	1.8
5	Loma Prieta 10/18/1989 Gilroy - Gavilan Coll. 67	GIL067	40.0	0.356	0.309	0.108	0.897
6	Mammoth Lakes-01 5/25/1980 Long Valley Dam (Upr L Abut) 90	LUL090	30.0	0.408	0.209	0.049	1.633
7	Imperial Valley-06 10/15/1979 Cerro Prieto 147	CPE147	63.8	0.296	0.204	0.092	3.757
8	Tabas Iran 9/16/1978 Tabas L	TAB_L	33.0	0.341	0.396	0.154	1.887
9	Gazli USSR 5/17/1976 Karakyr 0	GAZ000	13.5	0.376	0.355	0.149	1.516
10	San Fernando 2/9/1971 Palmdale Fire Station 120	PDL120	57.7	0.295	0.361	0.117	2.354
11	San Fernando 2/9/1971 Pacoima Dam (upper left abut) 164	PUL164	41.7	0.47	0.441	0.15	1.328

Spectrum of the 11 THs scaled to fit NBC (2015) from 0.05 to 1.0 sec

NBC(2005) spectrum is shown for comparison only.



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Racking Coefficient: R

(from Wang 1993)

Racking Coefficient. A racking coefficient, R, defined as the normalized structure racking distortion with respect to the free-field ground distortion is given as:

$$R = \frac{\gamma_s}{\gamma_{free-field}} = \frac{\left(\frac{D_s}{H}\right)}{\left(\frac{D_{free-field}}{H}\right)} = \frac{D_s}{D_{free-field}} \qquad \text{(Eq. 5-8)}$$

A. Structure Geometries Other than Single Barrel

B. Single Barrel Structure

R = 0.3 Stiff in racking where D_structure ➡ small; (but not rigid)

R = 1.0 Flexible in racking; D_structure ➡ $D_{\text{free field}}$ of soil (from SHAKE etc)

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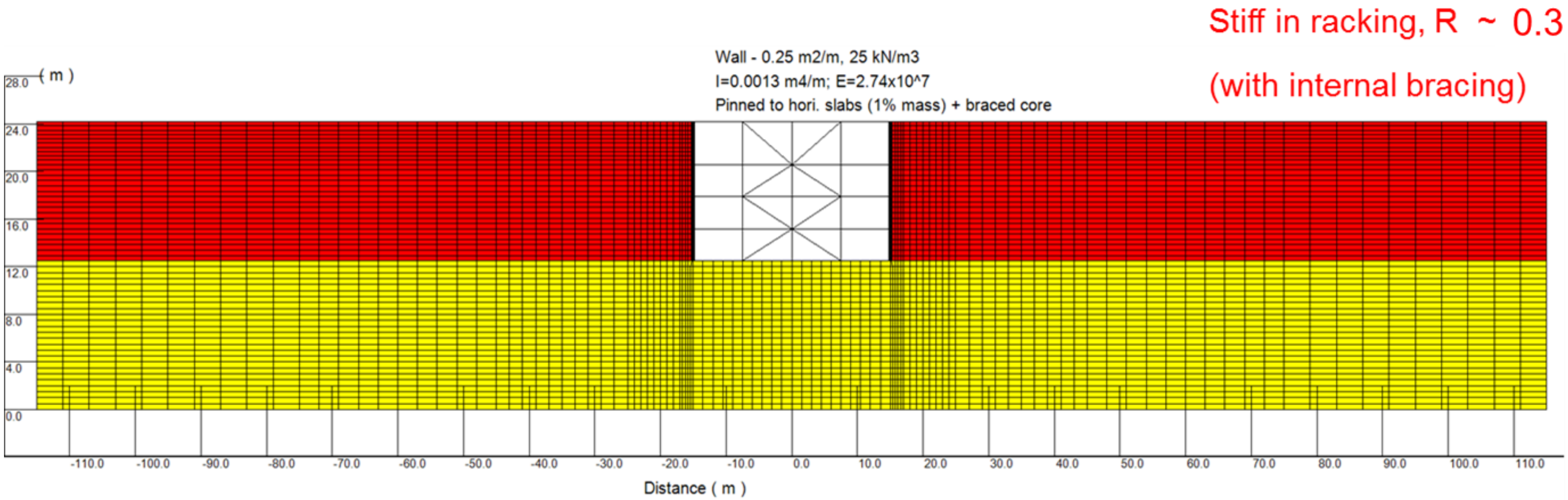
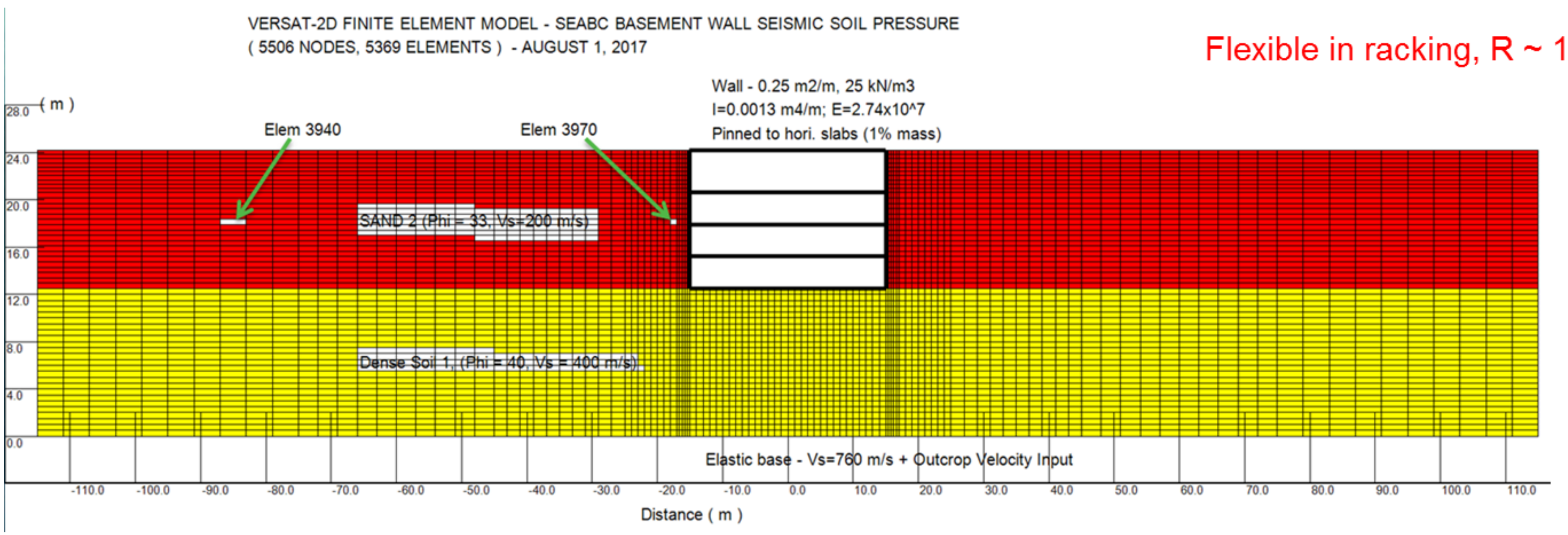
Seismic soil pressure on basement wall – Kinematic interaction only

Wall friction δ= 0° (also refer as "phi" in this document)

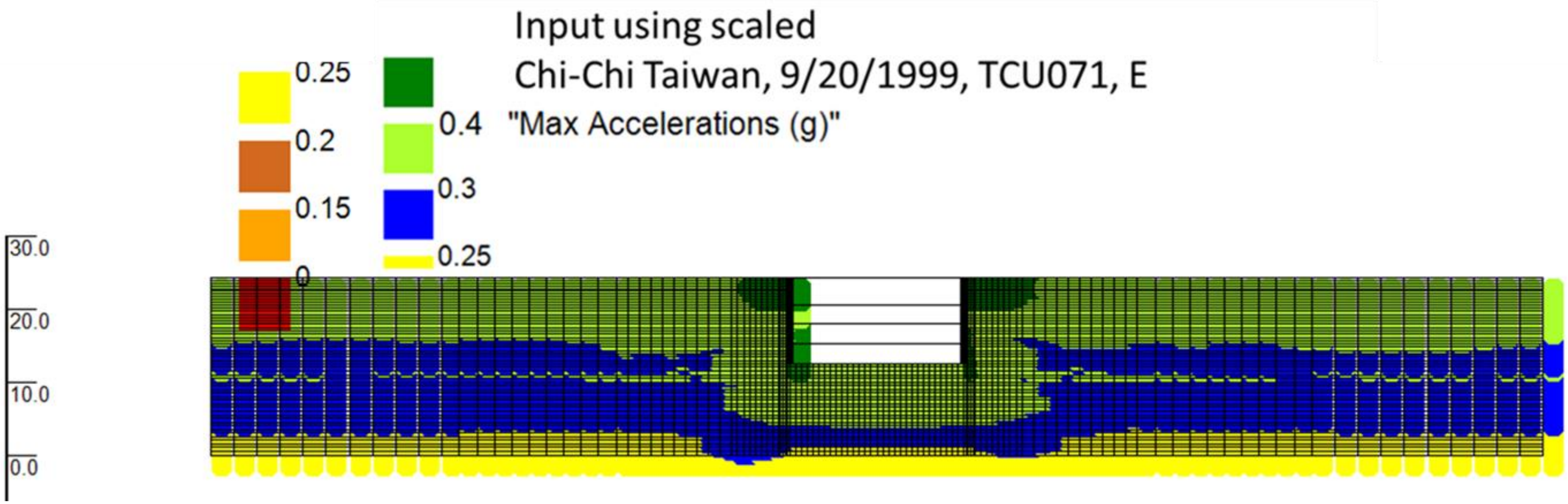
Stiff Racking (R=0.3) vs. Flexible Racking (R=1.0)

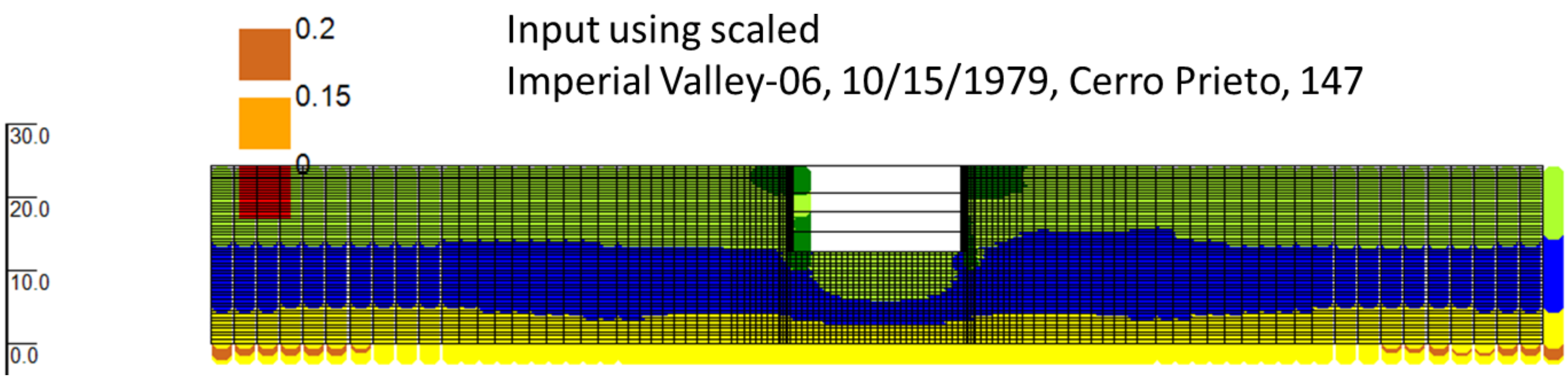

Stiff in racking, R ~ 0.3
(with internal bracing)
Wall - 0.25 m2/m, 25 kN/m3
I=0.0013 m4/m; E=2.74x10^7
Pinned to hori. slabs (1% mass) + braced core
(m)
28.0
24.0
20.0
16.0
12.0
8.0
4.0
0.0
-110.0 -100.0 -90.0 -80.0 -70.0 -60.0 -50.0 -40.0 -30.0 -20.0 -10.0 0.0 10.0 20.0 30.0 40.0 50.0 60.0 70.0 80.0 90.0 100.0 110.0
Distance (m)


VERSAT-2D FINITE ELEMENT MODEL - SEABC BASEMENT WALL SEISMIC SOIL PRESSURE
(5506 NODES, 5369 ELEMENTS) - AUGUST 1, 2017
Flexible in racking, R ~ 1
Wall - 0.25 m2/m, 25 kN/m3
I=0.0013 m4/m; E=2.74x10^7
Pinned to hori. slabs (1% mass)
Elem 3940
Elem 3970
SAND 2 (Phi = 33, Vs=200 m/s)
Dense Soil 1, (Phi = 40, Vs = 400 m/s)
Elastic base - Vs=760 m/s + Outcrop Velocity Input
(m)
28.0
24.0
20.0
16.0
12.0
8.0
4.0
0.0
-110.0 -100.0 -90.0 -80.0 -70.0 -60.0 -50.0 -40.0 -30.0 -20.0 -10.0 0.0 10.0 20.0 30.0 40.0 50.0 60.0 70.0 80.0 90.0 100.0 110.0
Distance (m)

Acceleration Response (max.) (R ~1)

Subject to Chi-Chi TCU; and Imperial Valley CPE





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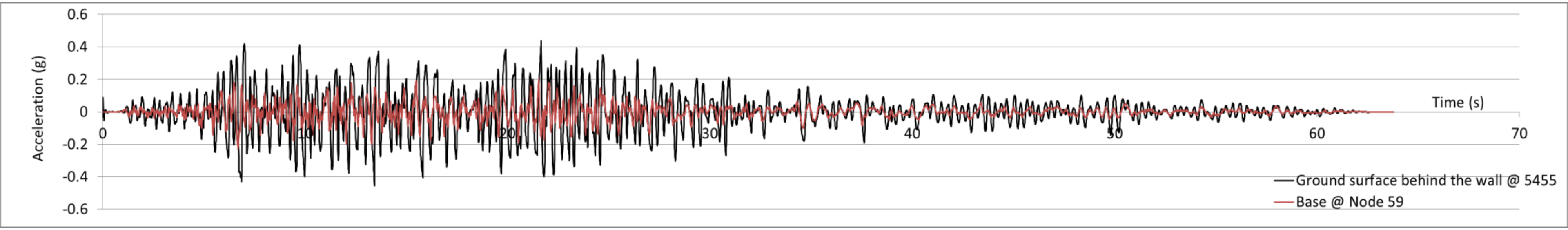
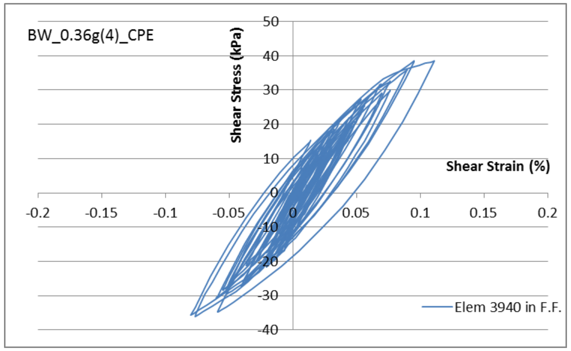
Acceleration THs (R ~1)

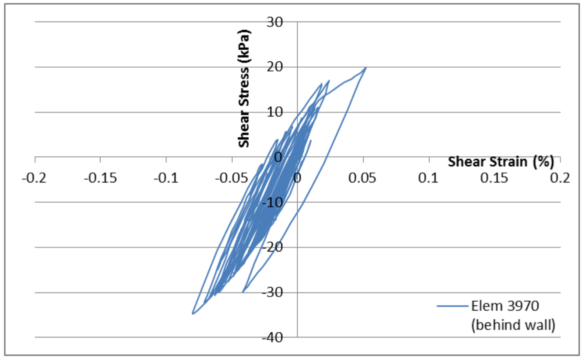


Shear Stress - Strain THs (R ~1)

Subject to Imperial Valley CPE (Free field at E3940, and behind the wall E3970



Nonlinear hysteretic in soil elements;
Elastic beam for walls
Elastic bar for hori slabs



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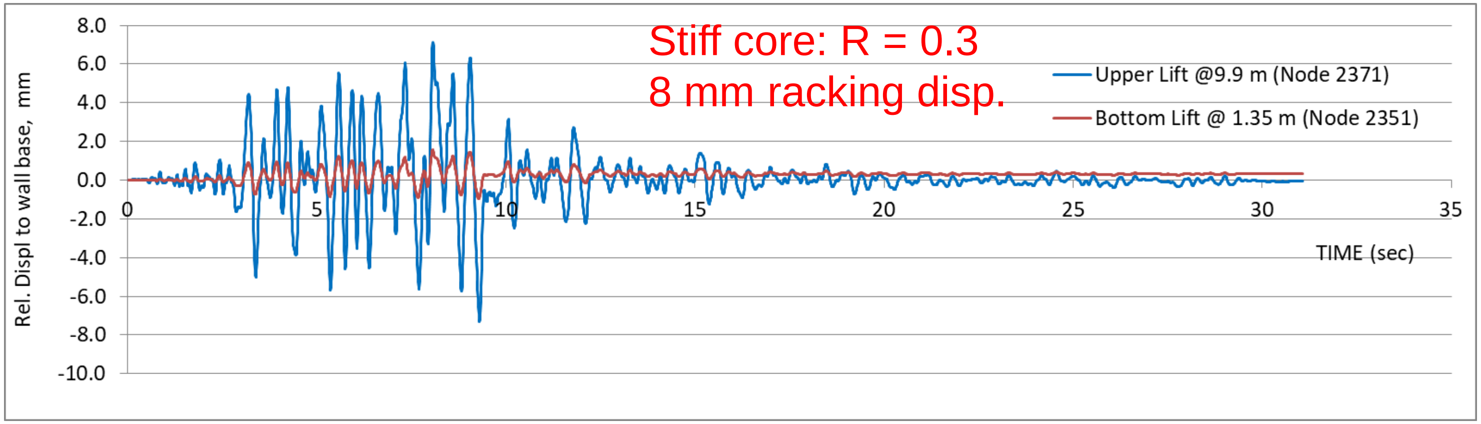
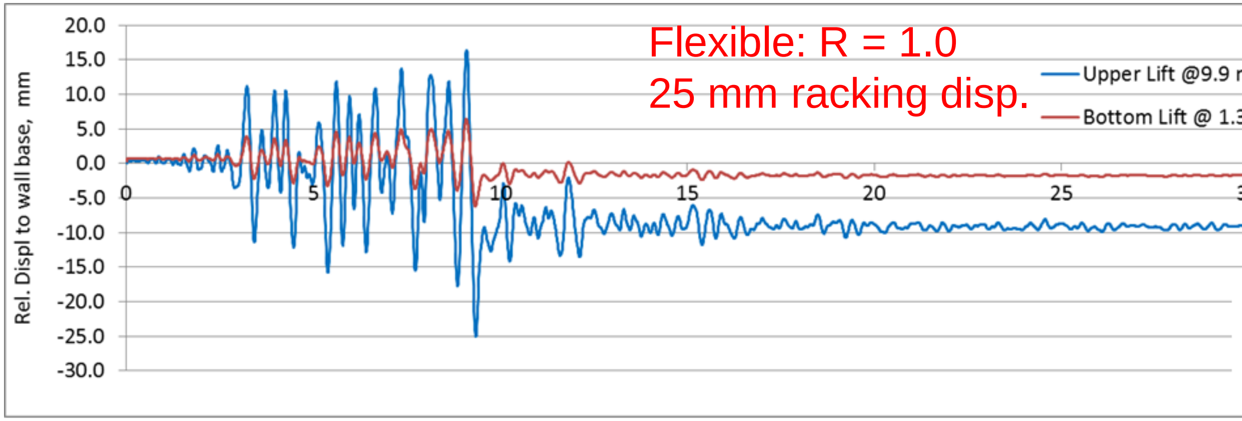
Wall Displacement THs (R ~ 0.3 vs. R ~1)

Subject to North Ridge CHL





CHL &
Phi = 5°

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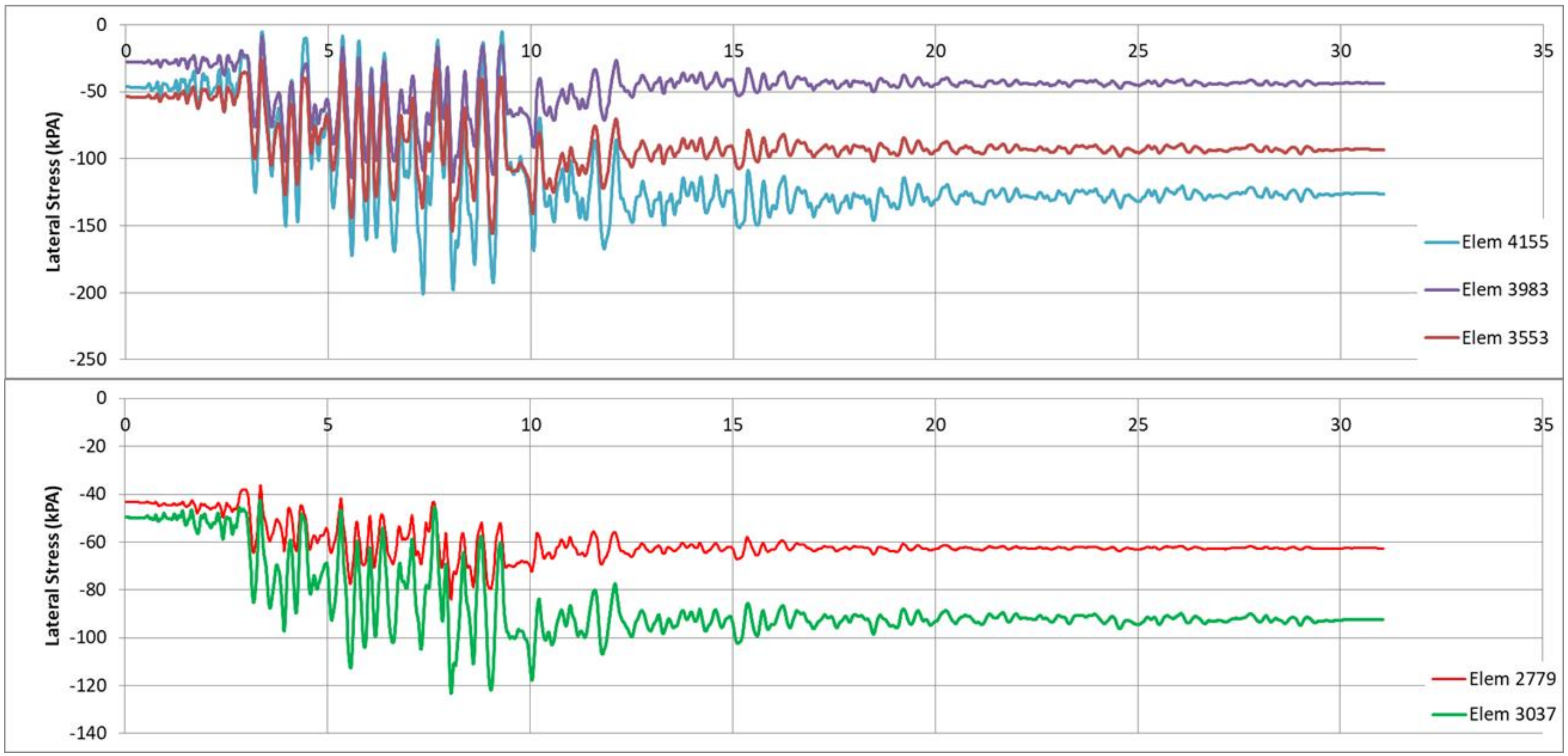
Horizontal stress THs of soil elem against the left wall:

stiff core case (5.0) with CHL input: (the pressures are in phase !)



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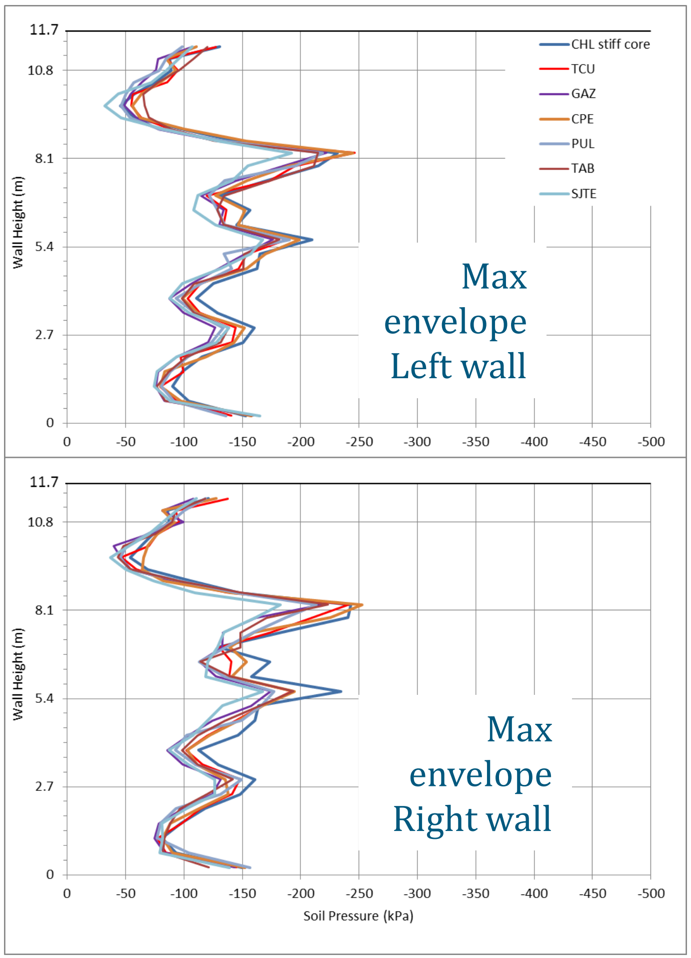
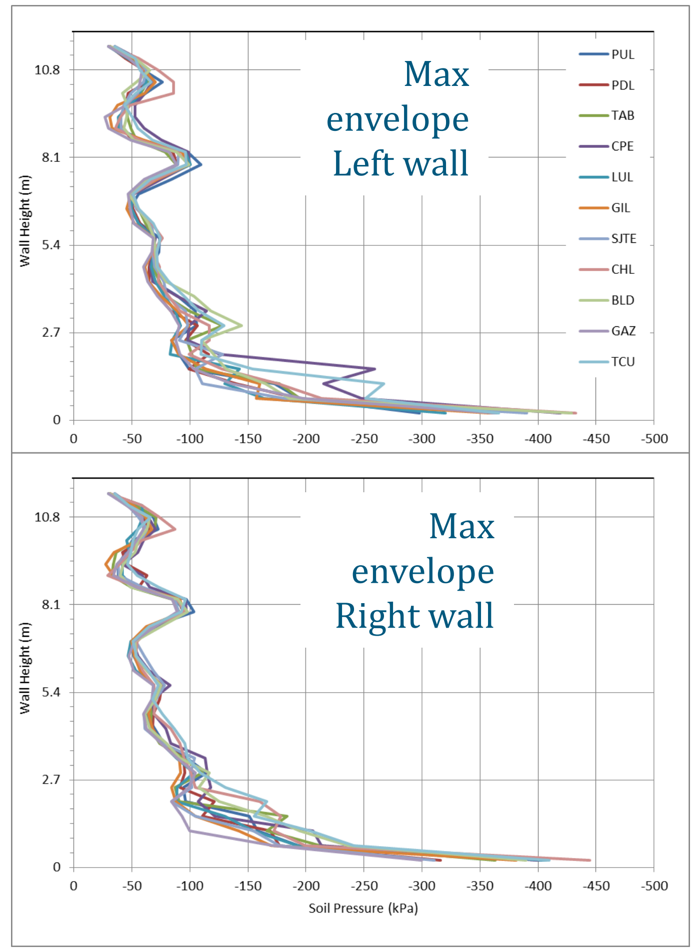
Finite Element Dyn. Analysis Results:

Soil pressures (max.) distribution

(a) Stiff core, R ~ 0.3 for 7 THs



(b). Flexible core, R ~ 1.0 for 11 THs



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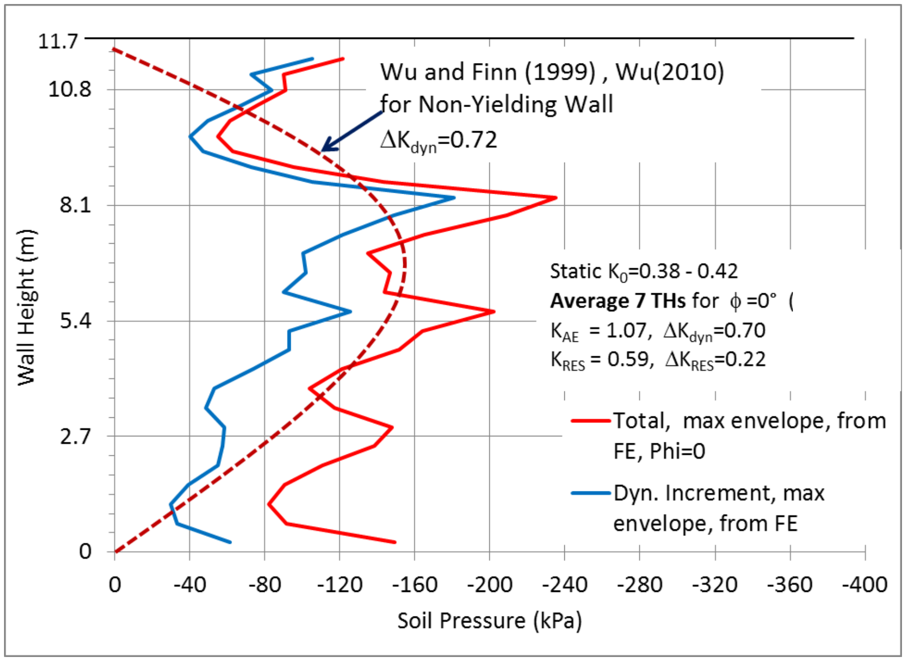
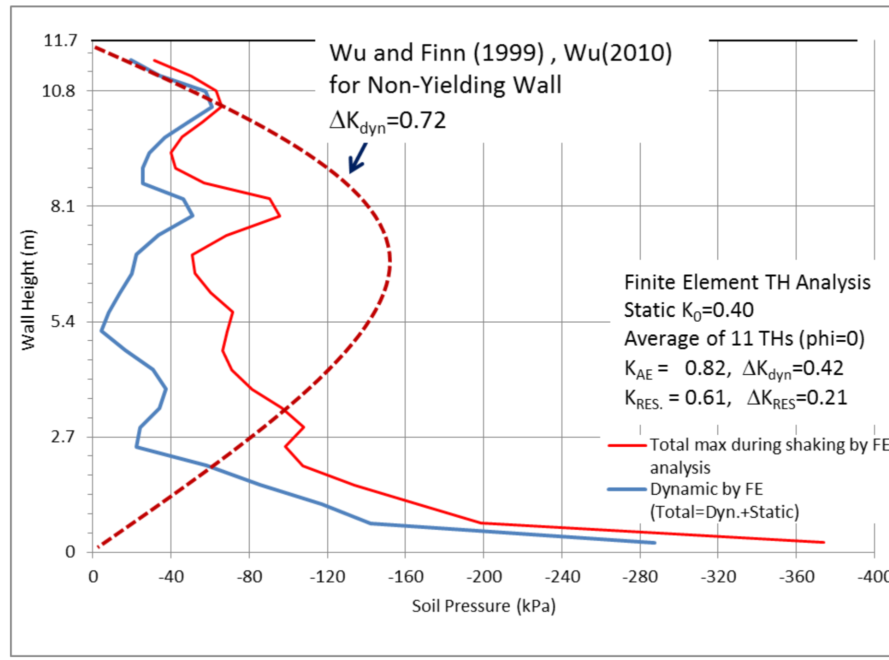
Finite Element (FE) Results compared to Rigid wall pressure with 100% PGA (0.36g)

Soil pressures (max. average)

(a) Stiff core, R ~ 0.3 for 7 THs



(b). Flexible core, R ~ 1.0 for 11 THs



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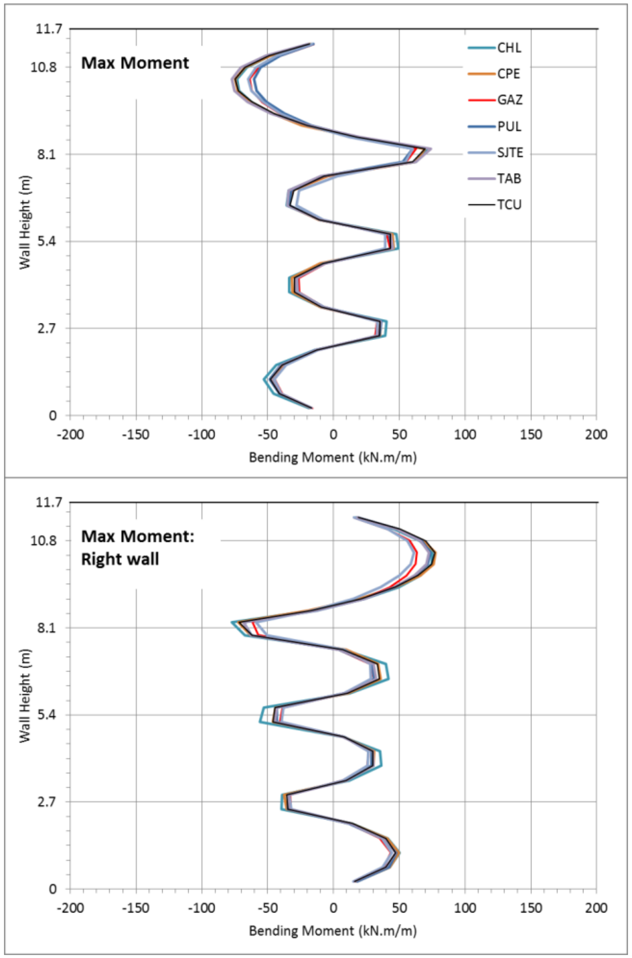
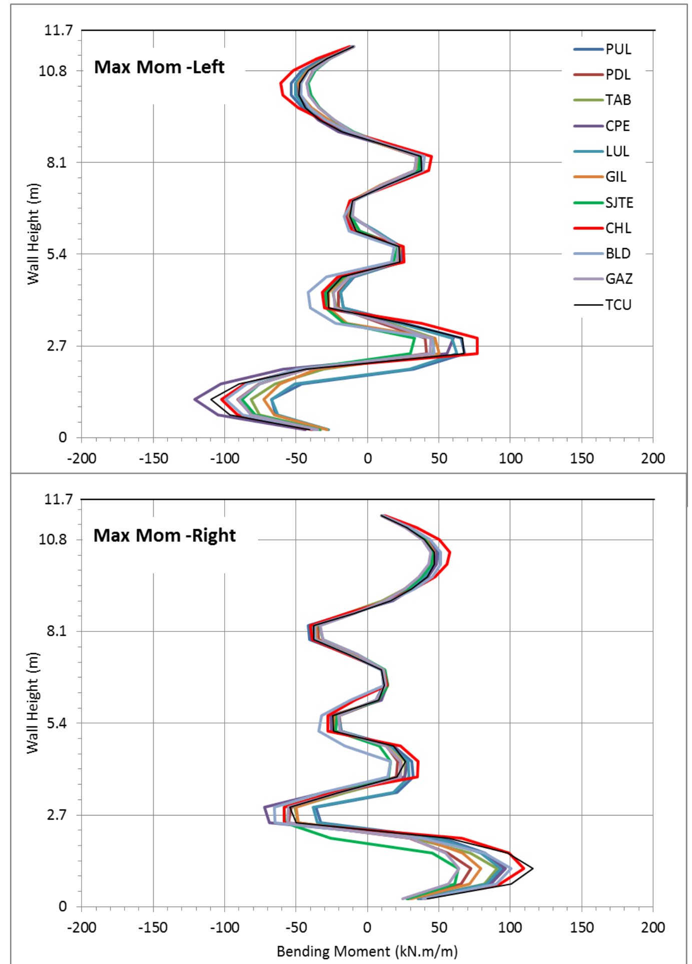
Finite Element Dyn. Analysis Results:

Bending moment (max.) distribution

(a) Stiff core, R ~ 0.3 for 7 THs



(b). Flexible core, R ~ 1.0 for 11 THs



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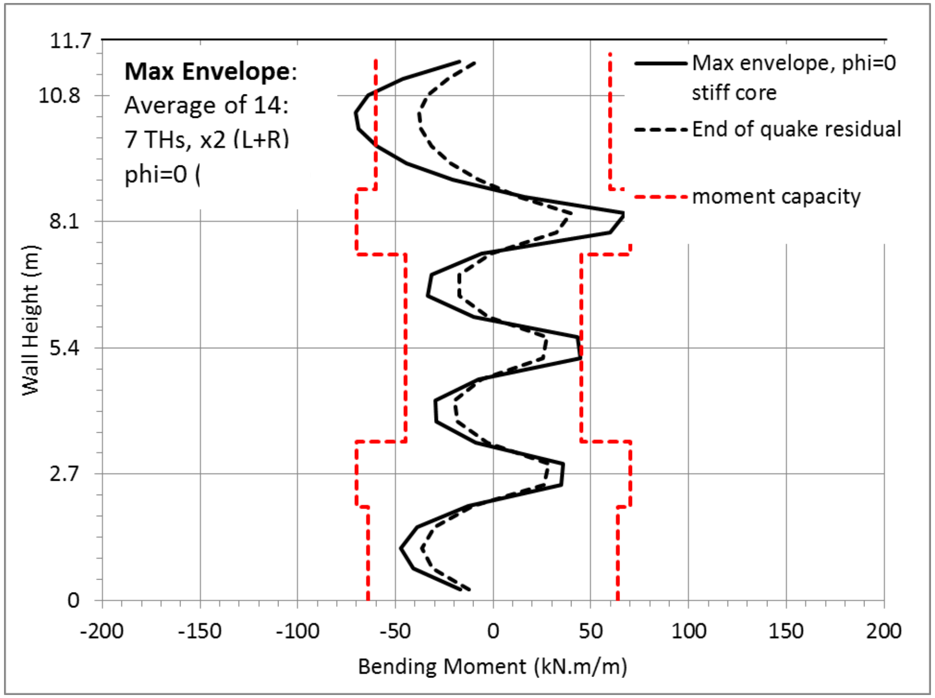
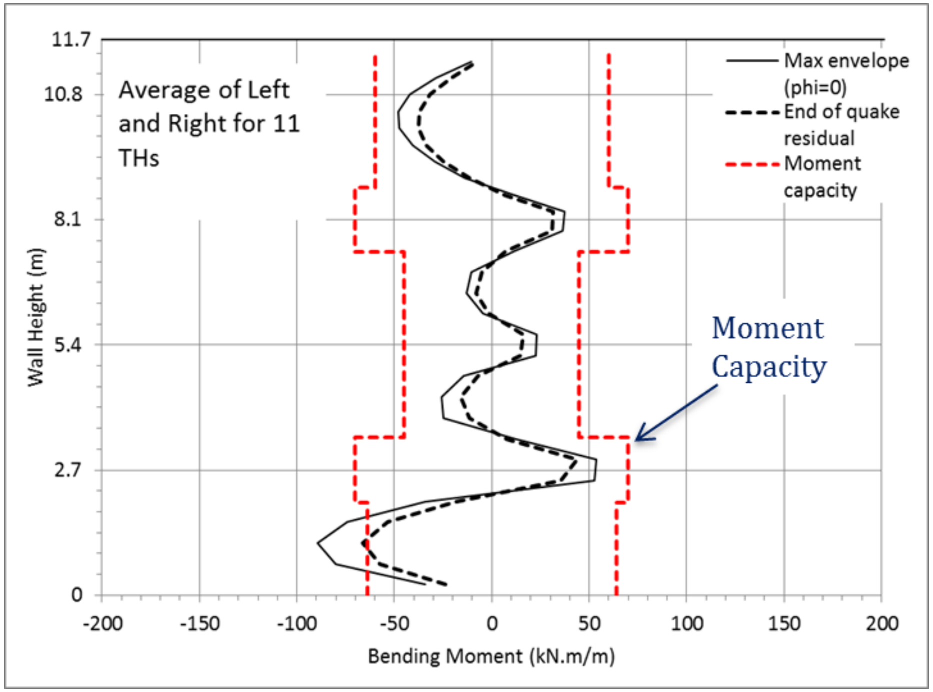
Finite Element Dyn. Analysis Results:

Bending moment (max. and residual average)

(a) Stiff core, R ~ 0.3 for 7 THs



(b). Flexible core, R ~ 1 for 11 THs

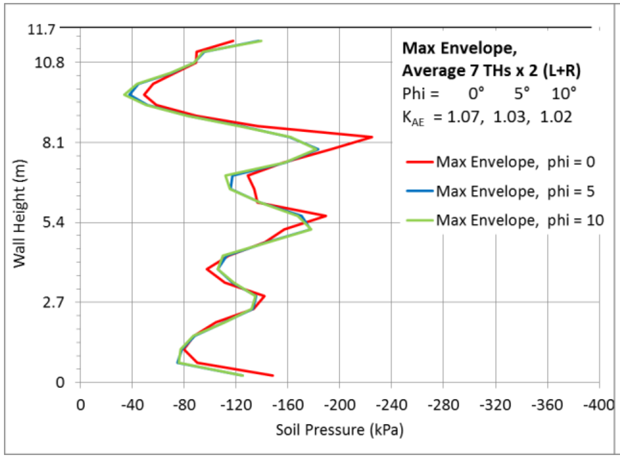


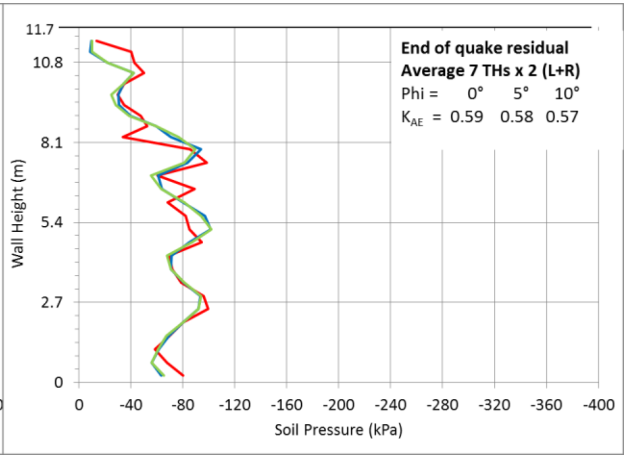
Note: Moment capacity is referred herein to provide a comparison to Taiebat et al. (2014); it has not been used in the finite element analyses.

Kinematic interaction:

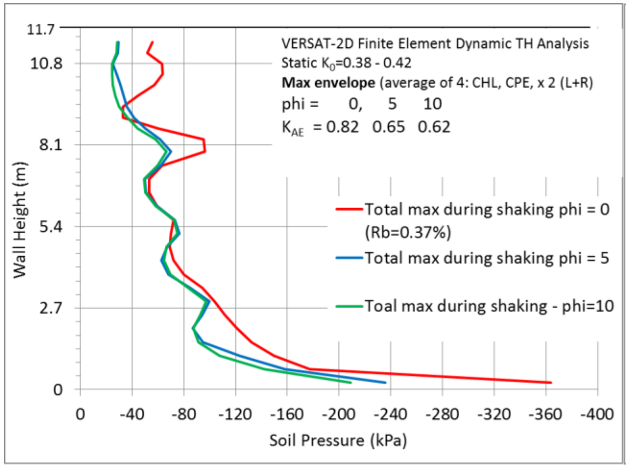
Effect of wall friction δ = 0°, 5°, 10° (also refer as Phi) on soil pressures (max. and residual average)

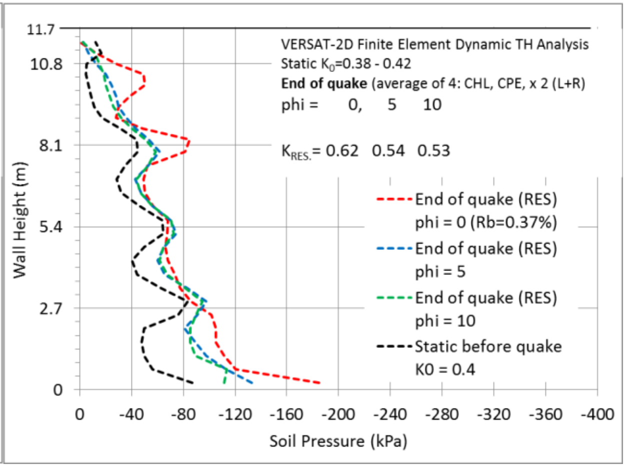
(a) Stiff core, R ~ 0.3 for 7 THs





(b). Flexible core, R ~ 1.0 for 11 THs (2 THs for Phi=5, 10)





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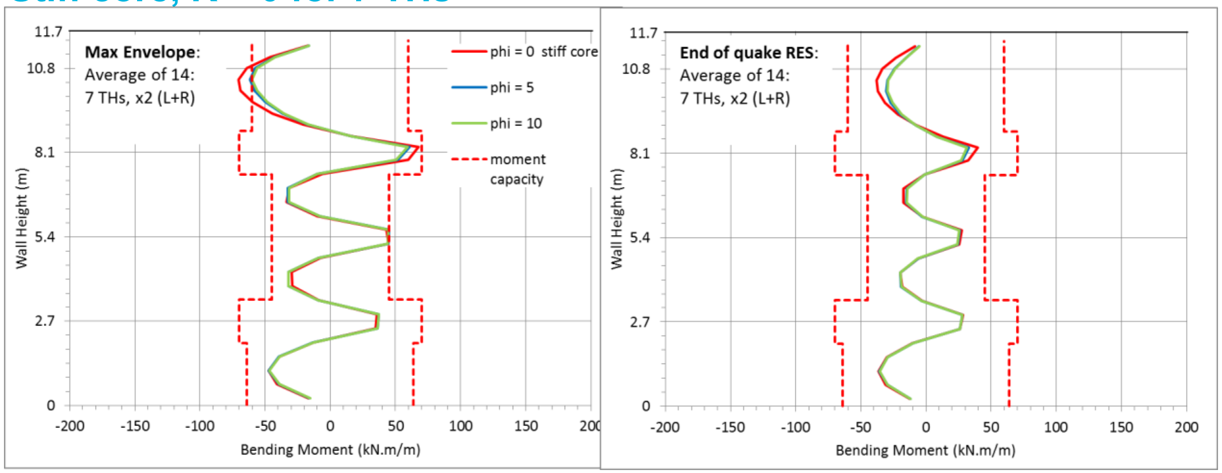
Kinematic interaction:

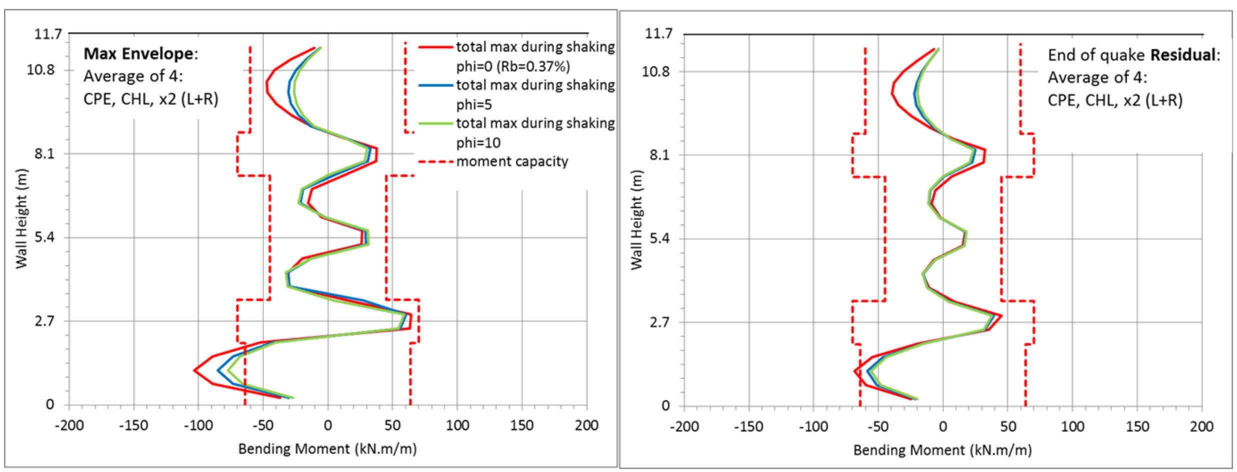
Effect of wall friction = 0°, 5°, 10° on soil pressures (max. and residual average)

(a) Stiff core, R ~ 0 for 7 THs



Note: Moment capacity is referred herein to provide a comparison to Taiebat et al. (2014); it has not been used in the finite element analyses.

(b). Flexible core, R ~ 1 R ~ 1 for 11 THs (2 THs for Phi=5, 10)



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Comparison of Analysis Results:

Finite Element Dynamic (VERSAT)

versus

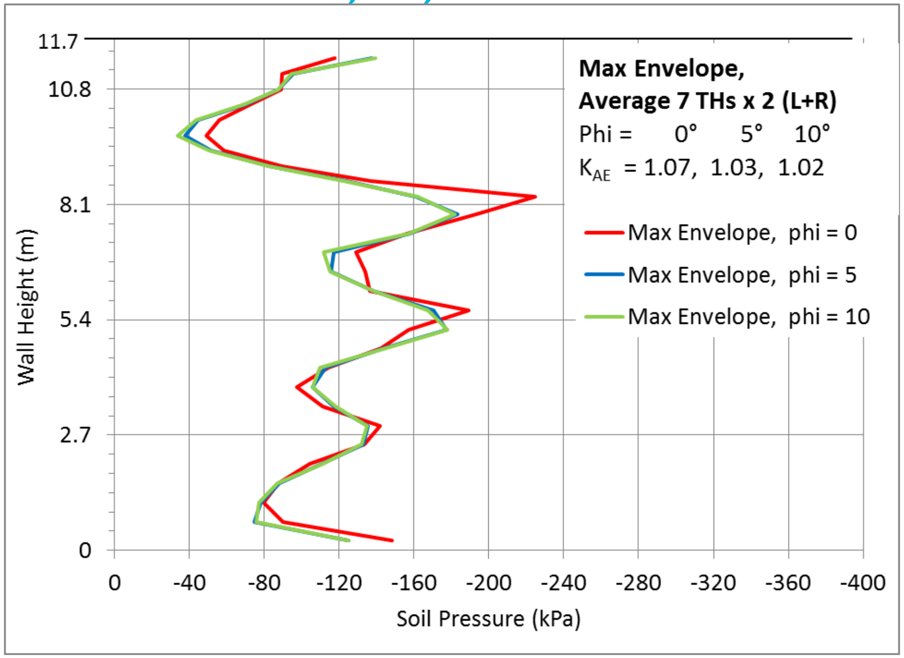
Finite Difference Dynamic (FLAC)

Comparison of Analysis Results:

Finite Element Dynamic (VERSAT):

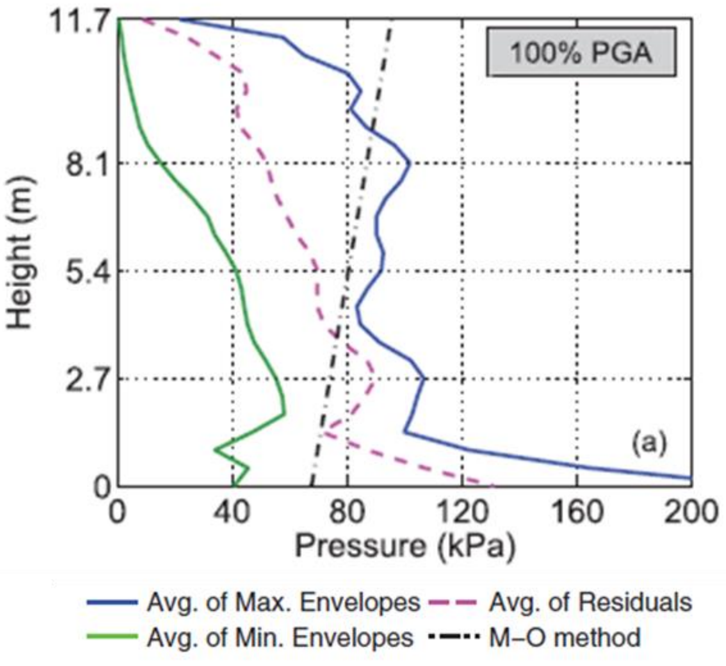
7 THs scaled to NBC(2015) 0.36g

For stiff core case, i.e., R = 0.3



Finite Difference Dynamic (FLAC):

7 THs SP matched to NBC(2005) 0.46g



Source: Taiebat et al. (2014) CGJ Vol.51

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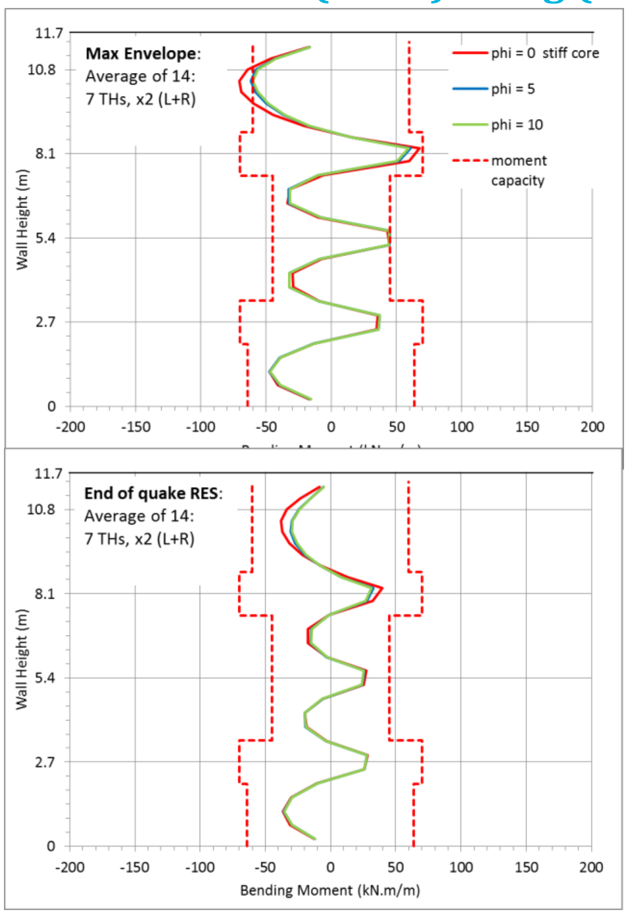
Comparison of Analysis Results:

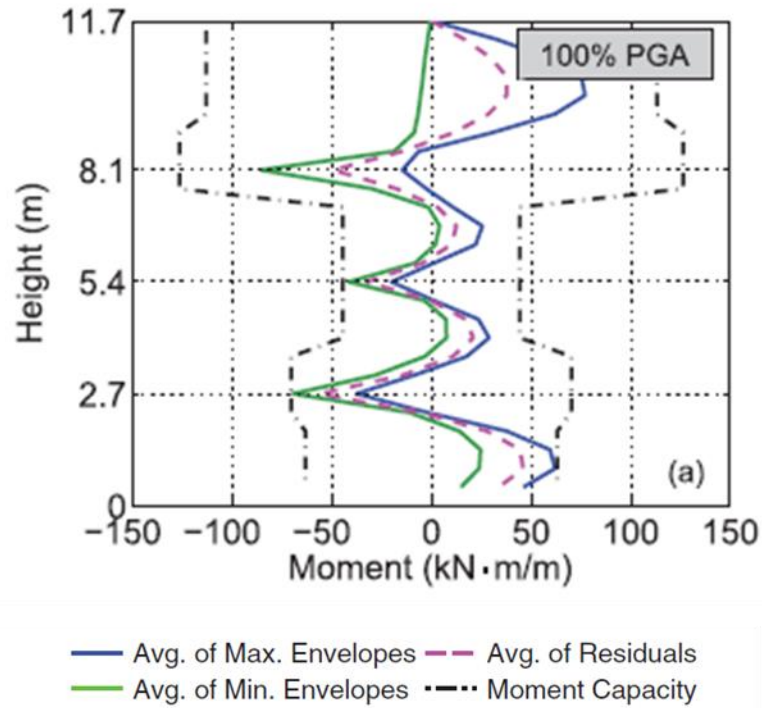
Finite Element Dynamic (VERSAT):

7 THs scaled to NBC(2015) 0.36g (for R=0.3)



Finite Difference Dynamic (FLAC):

7 THs SP matched to NBC(2005) 0.46g



Source: Taiebat et al. (2014) CGJ Vol.51

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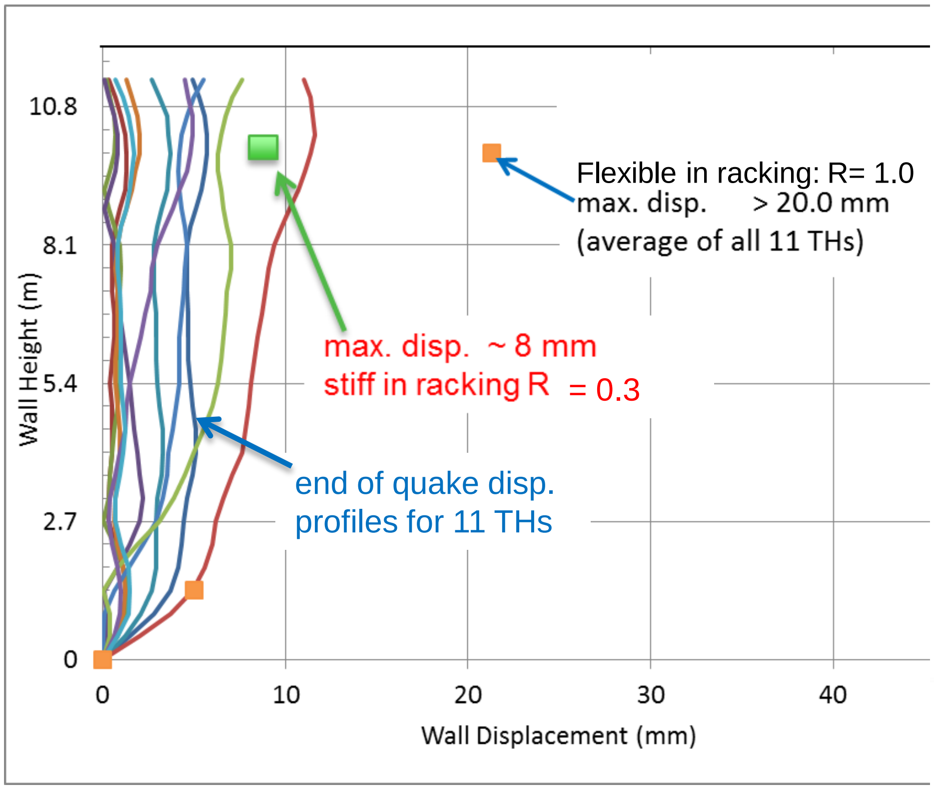
Comparison of Analysis Results:

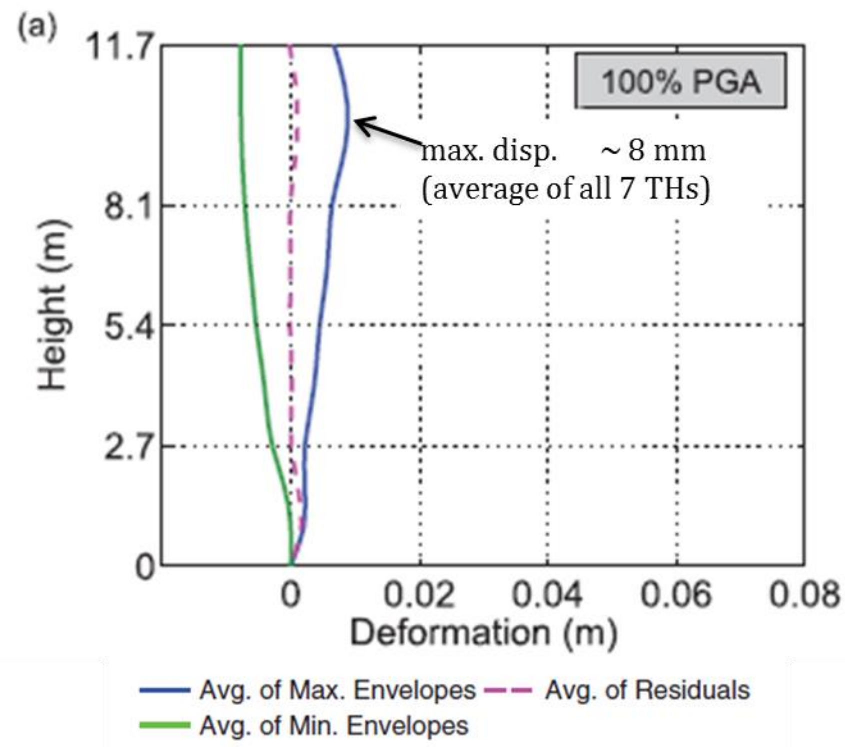
Finite Element Dynamic (VERSAT):

11 THs (7 THs) scaled to NBC(2015) 0.36g



Finite Difference Dynamic (FLAC):

7 THs SP matched to NBC(2005) 0.46g



Source: Taiebat et al. (2014) CGJ Vol.51

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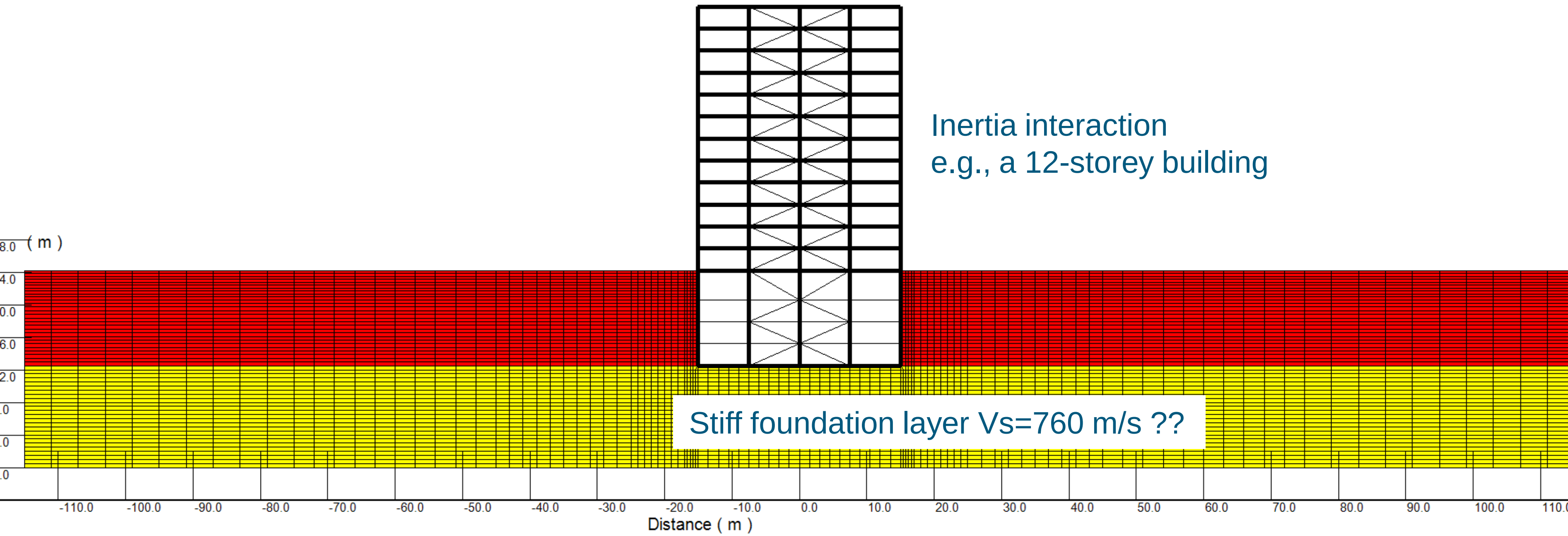
Other major factors to consider:

- Inertia interaction – can increase loads ??
- Stiffer foundation layer with Vs=760 m/s; can increase ground accelerations, and seismic soil pressures ??



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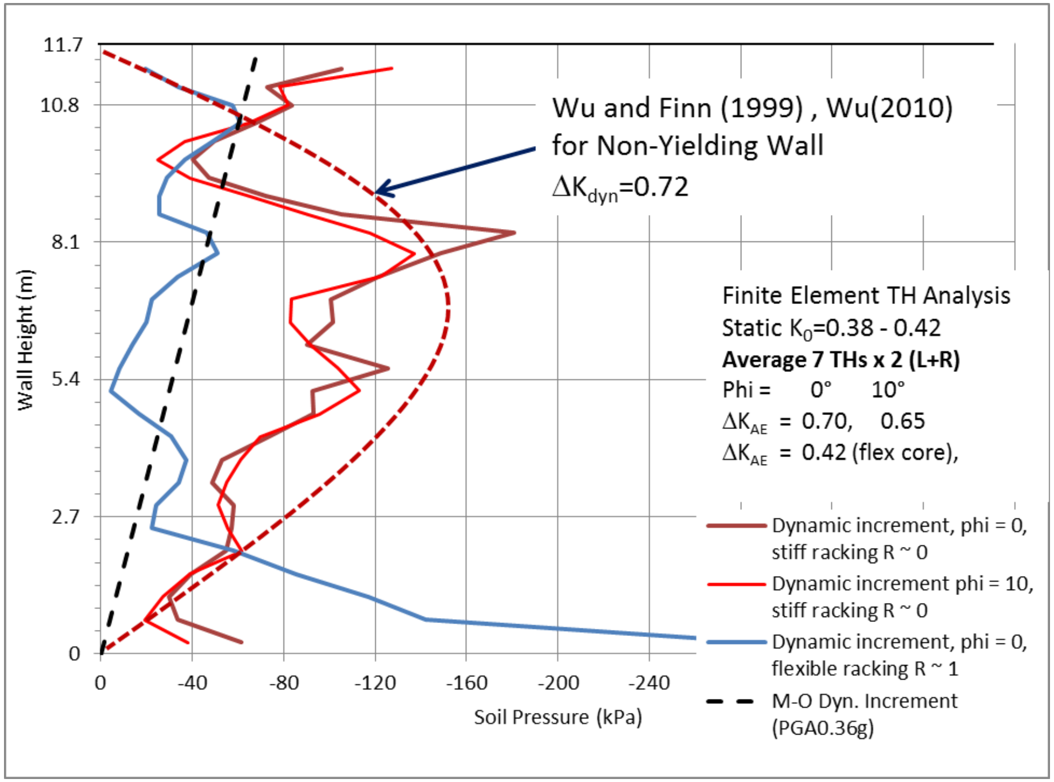
Design Consideration (for discussion):

Based on Finite Element Dyn. Analyses for NBC Vancouver Site (0.36g)

Wall friction angle, δ, is referred as "phi" in this plot.



The design seismic pressures on basement wall depends on racking stiffness of the system:

- The racking stiffness can be determined by a frame push-over analysis (see Slide #5);
- The racking coefficient, R, is then determined from free field soil deformations.
- Dynamic force increment K_{AE} decreases from 0.72 to 0.32 as R ~ 0.3 (stiff) to R ~ 1 (flex)
- The distribution of the seismic pressures vary with R; the centre of force moves downwards as the R increases, i.e., from stiff to flex.

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Conclusion Remarks:

1. The racking coefficient R can be determined (Slide 5) from racking stiffness (frame push over analysis) and the free field soil displacement. The effect of racking stiffness should be taken into account by Engineers while providing the seismic soil pressures.
2. The finite element dynamic analyses, using NBC2015 with PGA of 0.36g, predicted that dynamic force increment ΔK_{AE} decreases from 0.72 to 0.32 as the racking coefficient R = 0.3 (stiff) increases to R =1 (flex); The distribution of the seismic pressures also vary with R; the centre of force moves downwards as the R increases, i.e., from a stiff to flex system.
3. For a stiff core basement (e.g., R=0.3 in this study), the assumed M-O invert triangular distribution of dynamic soil pressure is not applicable for deep basement walls (such as 11.7 m, 13.1 m, and 17.1 m deep walls referenced); and M-O force with 60% PGA (even with 100% PGA) can significantly underestimate the total seismic thrust.
4. Finite Element dynamic analyses (VERSAT) and Finite Difference analysis (FLAC) may have predicted some significantly different structural response on wall bending moments and on seismic soil pressures. More detailed comparisons could be made using the same quake levels (e.g., level for NBC2005 with PGA of 0.46 g) – This provide us a good example on how critical it is to check numerical solutions!!
5. The inertia from the superstructure building and the foundation soil stiffness variation (Vs=200 m/s vs. others) can also have impacts to the seismic soil pressures on the basement wall; more studies could be conducted to investigate. Yield in bending of the wall can also impact the pressure distribution when high seismic level loads apply.

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