Development of LRFD Resistance Factors for External Stability of MSE Walls

FDOT BDK75-977-22

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Scope of Work

- Perform centrifuge tests of MSE wall external stability which include soil variability
  - Sliding stability
  - Bearing capacity (flat ground)
  - Bearing capacity (on embankments)
- Analyze and validate test results
  - Calculate dead load factors: horizontal and vertical soil pressures
  - Quantify CV_Q, CV_R (load and resistance)
  - Quantify bias: λ_Q and λ_R for conventional methods
- Validate methods for bearing of walls on embankments
- Develop resistance factors (Φ) for stability cases tested

Background

Recommended factors, Loadings, Methods of analysis and permissible backfill

Wall Type	Load Factors[a]	Resistance Factors[a]	Loadings	Analysis	Reinforced Backfill Soil[c]
MSE	Vertical Earth: 1.35 Horizontal Earth: 1.5 Surcharge: 1.75	Bearing: 0.65 Sliding: 0.9-1.0 Overall: 0.65-0.75	Service I, Horizontal, Sloping, Broken Back	Vesic, Rankine[b], Meyerhoff, Modified Bishop, Simplified Janbu, Spencer	GW, SP, SW &SP GM, GC, SM & SC if <15% fines

[a] AASHTO LRFD (2009)
[b] Permitted in lieu of Coulomb analysis (FDOT SDG, 2009)
[c] FDOT SFH (2009)

Parametric Study

	Baseline Parameters		Range of Parameters		
	Mean (μ)	Coefficient of Variation (CV)	Mean (μ)	Coefficient of Variation (CV)	Distribution
Retained Soil ϕ	30°	10%	20° - 40°	5% - 20%	Lognormal
Retained Soil γ	105 pcf	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Foundation Soil ϕ	35°	10%	20° - 40°	5% - 20%	Lognormal
Foundation Soil γ	105°	5%	95 pcf - 120 pcf	5% - 20%	Lognormal
Surcharge q_s	250 psf	25%	NA	NA	Lognormal

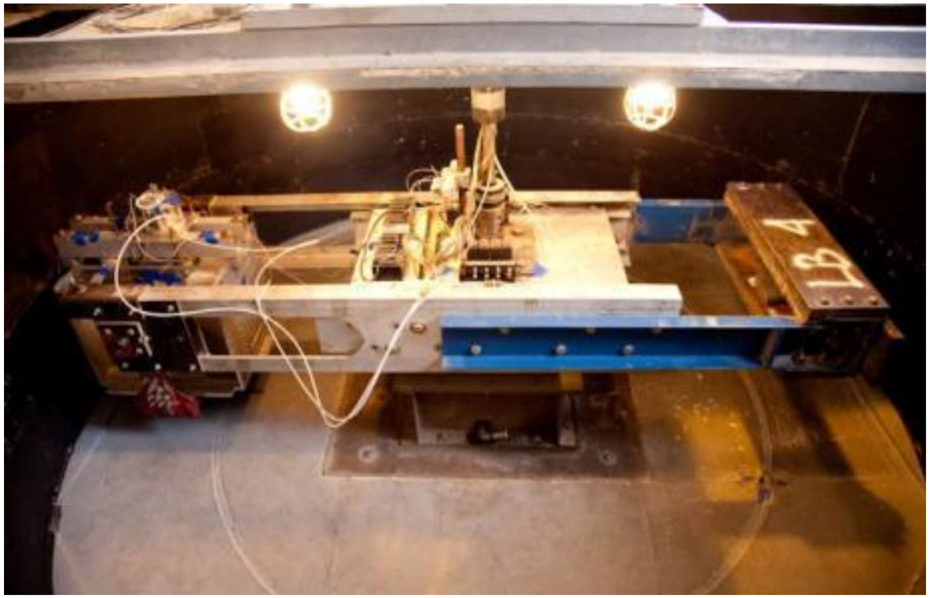
Greatest influence on P_f from:

- CV_ϕ and μ_ϕ of backfill and foundation soil;
- CV_γ backfill

Greatest influence on P_f from:

- CV_ϕ and μ_ϕ of foundation soil
- μ_ϕ of retained soil

Centrifuge Tests

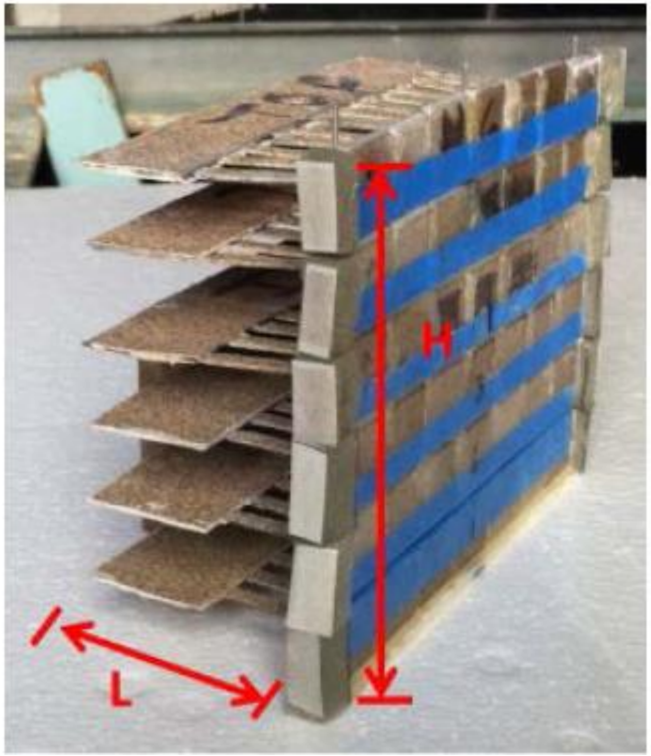


Scaling laws

Property	Prototype	Model
Acceleration (L/T^2)	1	N
Linear Dimensions (L)	1	1/N
Area (L^2)	1	$1/N^2$
Volume (L^3)	1	$1/N^3$
Mass (M)	1	$1/N^3$
Force (ML/T^2)	1	$1/N^2$
Unit Weight (M/L^2T^2)	1	N
Density (M/L^3)	1	1
Stress (M/LT^2)	1	1
Strain (L/L)	1	1
Moment (ML^2/T^2)	1	$1/N^3$

- 2.6 m diameter; 12.5 G-Ton capacity beam centrifuge
- Model heights up to 24 in, widths up to 20 in
- Hydraulic system for double acting pneumatic pistons
- 12-Channel wireless data acquisition

Bearing Stability Models



s_v = 0.78 inch

s_h = 0.47 inch

$\#rows$ = 6

w_r = 0.25 inch

t_r = 0.0125 inch

f'_y = 35,000 psi (reinforcement)

f'_y = 2,324 psi (connection)

H = 6 inches

L = 3 inches

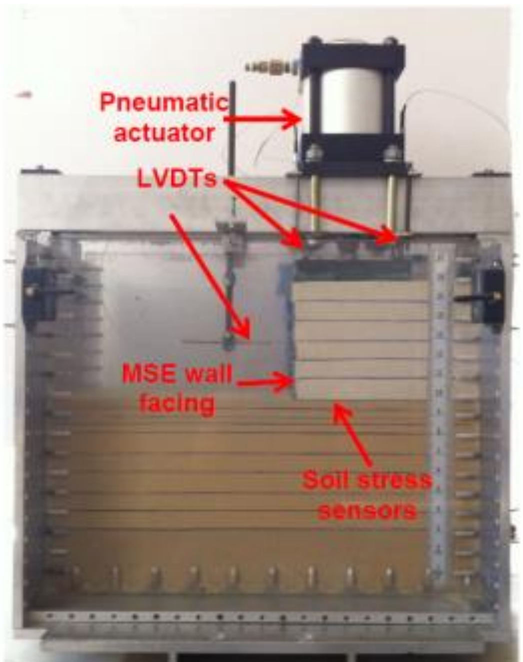
$\Phi_{pullout}$ = 0.90

$\Phi_{rupture}$ = 0.75

γ_{DL} = 1.35

Designed for stability against pullout and rupture failure

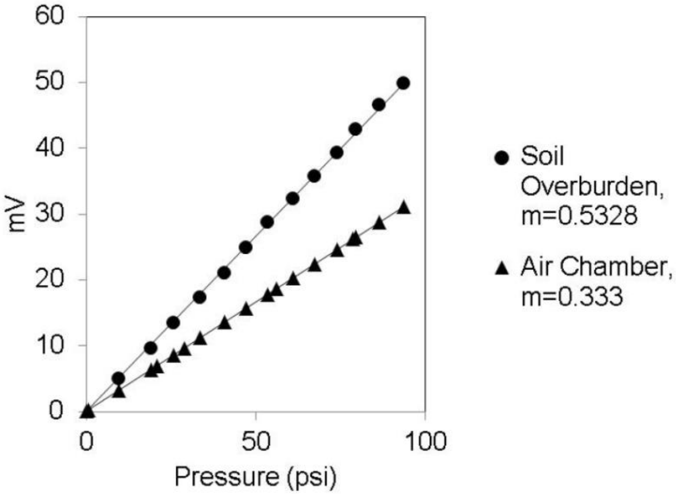
$CDR_{pullout, rupture}$ (capacity/demand) > 2



Soil Stress Sensors

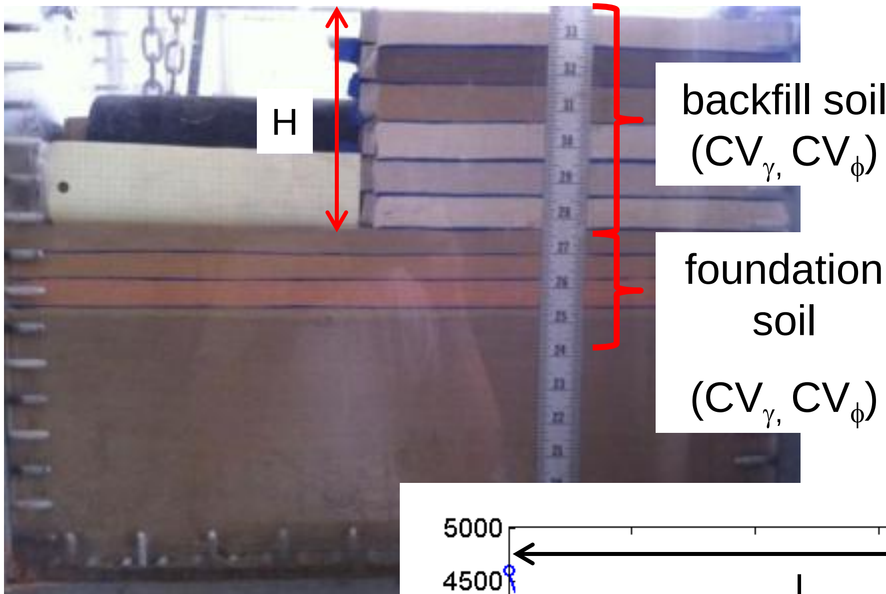
- Embedded sensor will influence measurements
- Factors must be satisfied for reliable output
- Stress sensor requires calibration for use in soil
- Performed in centrifuge utilizing increased G environments

Factor	Required ratios	Measured Ratios
Aspect ratio	T/D < 1/5 (Experimentation Station, 1944) < 1/10 (Dunnicliff, 1988)	1/7.5 – 1.5/7.5 < 1/5
Active diameter	$d/D_{50} > 10$	6/0.2 > 10
Sensor-soil stiffness ratio	> 0.5	28.5×10^3 ksi/(0.6 – 4 ksi) > 0.5
Active diameter/Deflection	$d/\Delta > 2000 - 5000$	6/0.002 > 2000

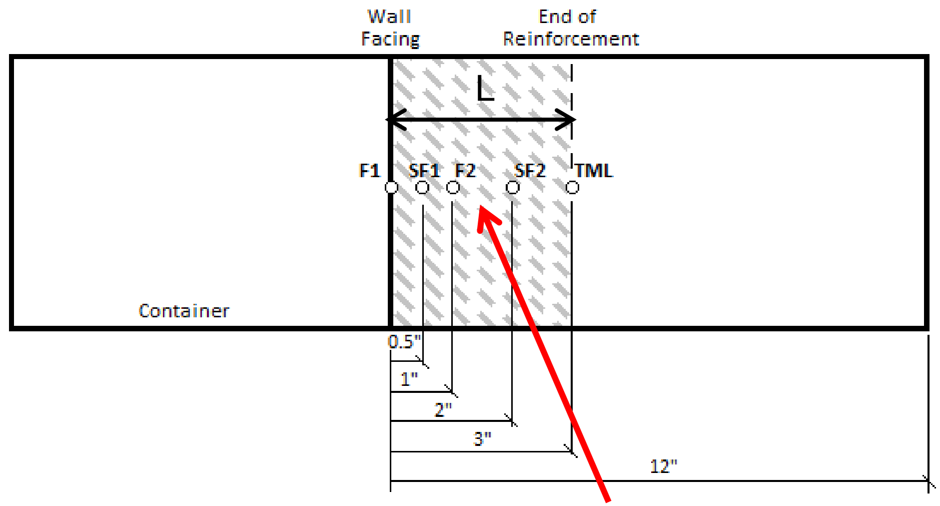


Bearing Stability Tests

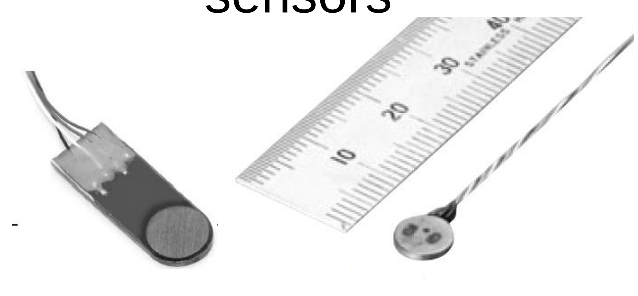
Side View



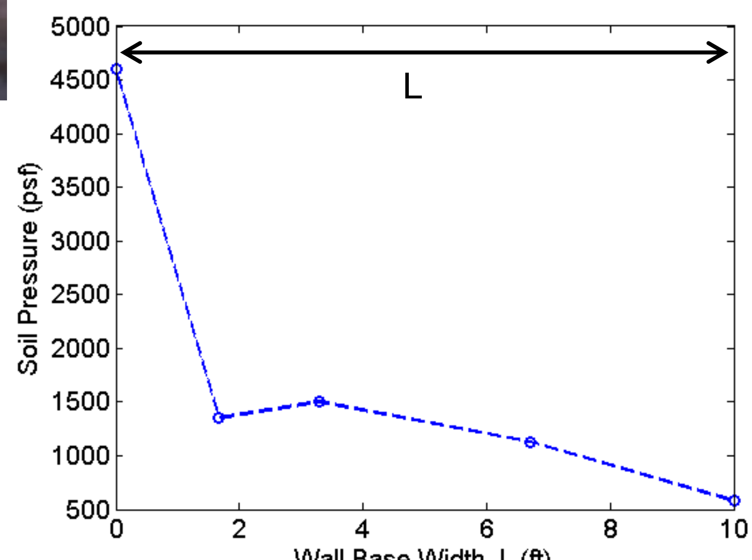
Plan View



Miniature soil pressure sensors



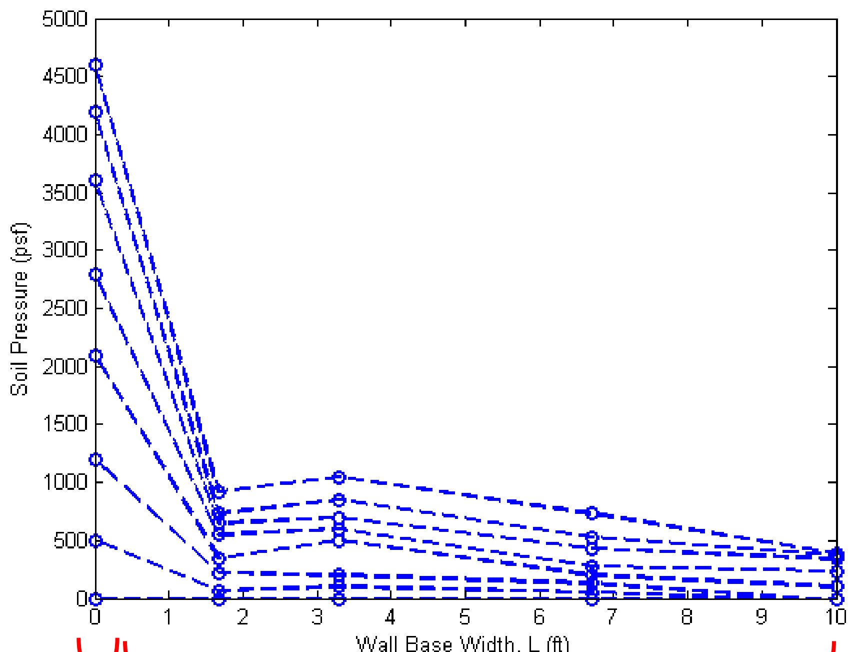
Measured soil pressure distribution

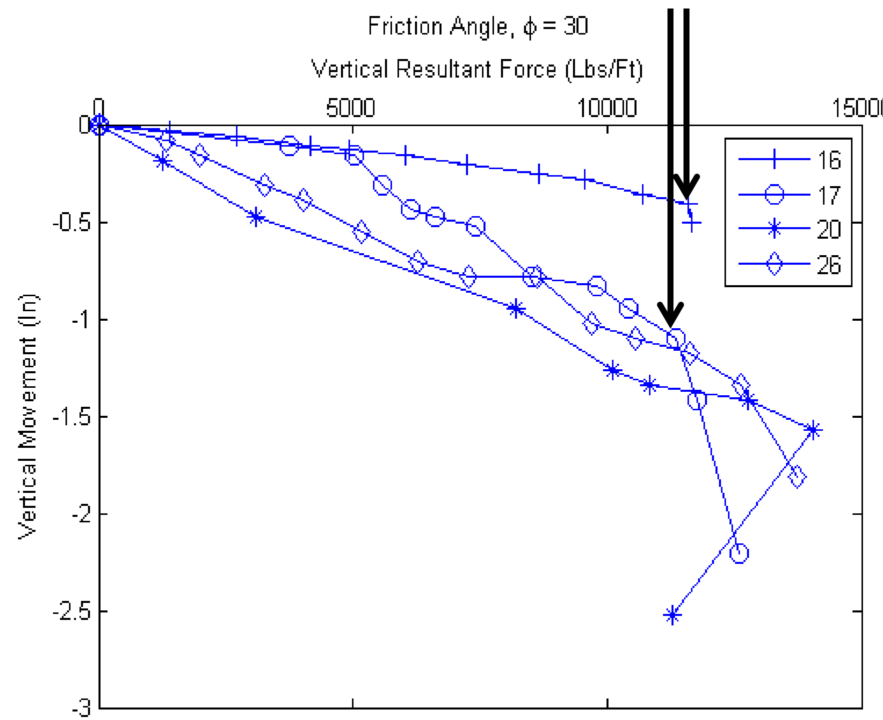


Bearing Stability Test Results

Load-Displacement curves

capacity



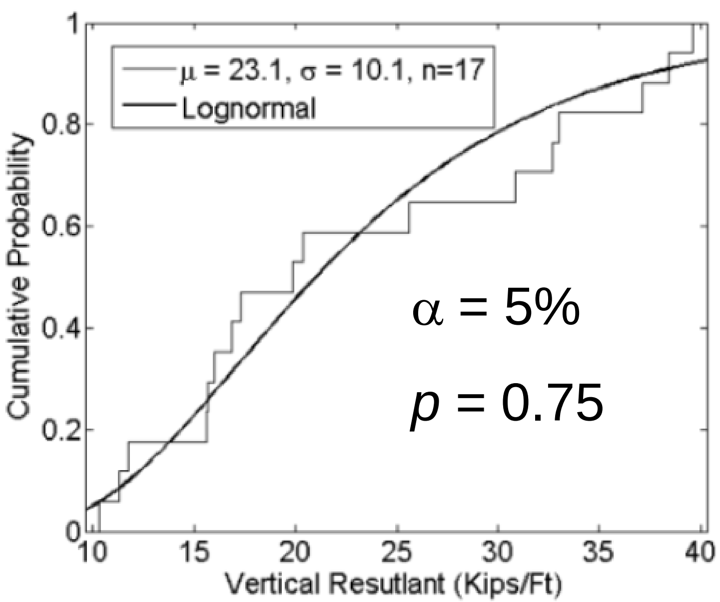


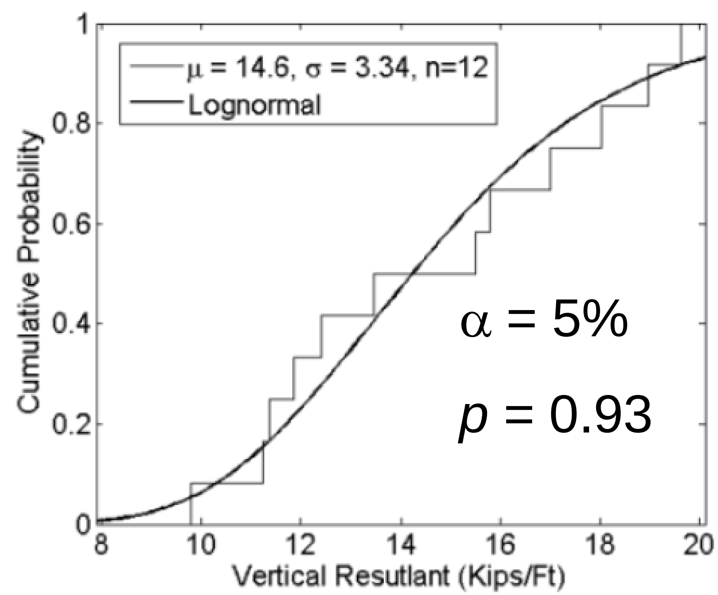
Rigid wall facing

Flexible reinforced soil

Bearing Stability Test Results

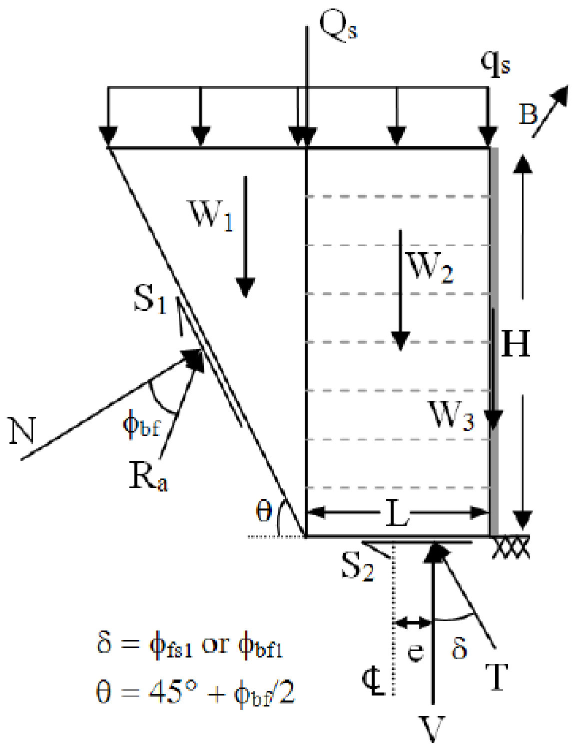
- $V_{measured}$ (capacity) grouped by $\mu_{\phi fs}$ (foundation soil)
- $\mu_{\phi fs}$ = 26° -30° and $\mu_{\phi fs}$ = 31° -33°
- K-S (Kolmogorov-Smirnov) fit test α = 5%

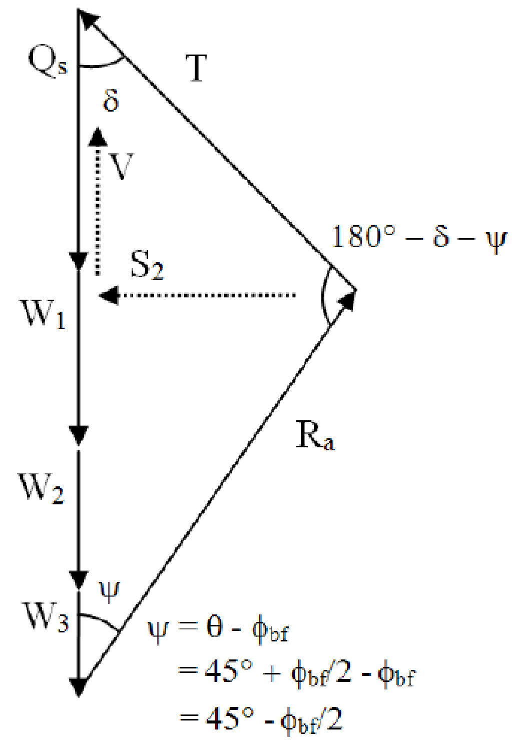




Validation-Force Equilibrium

- Failure of wall can be described by a planar rupture surface through backfill (observed in tests)
- $V_{calculated}$ from force polygon and measured weights



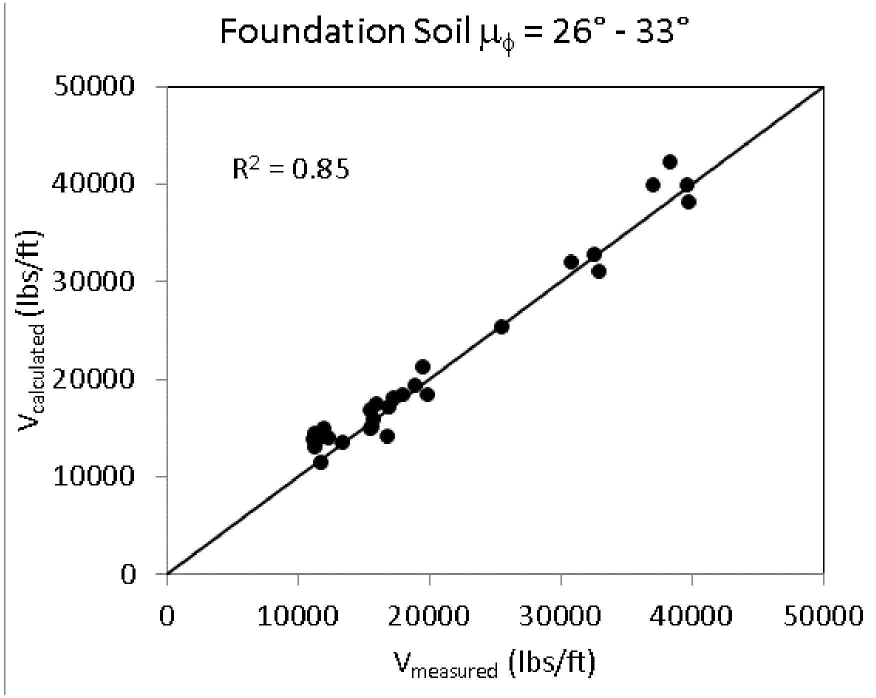


Of interest: S_2, V and e

$$V = \left[\frac{W_1 + W_2 + W_3 + Q_S}{\frac{\sin(\delta)}{\tan(\theta - \phi)} + \cos(\delta)} \right] \cos(\delta)$$

Validation-Force Equilibrium

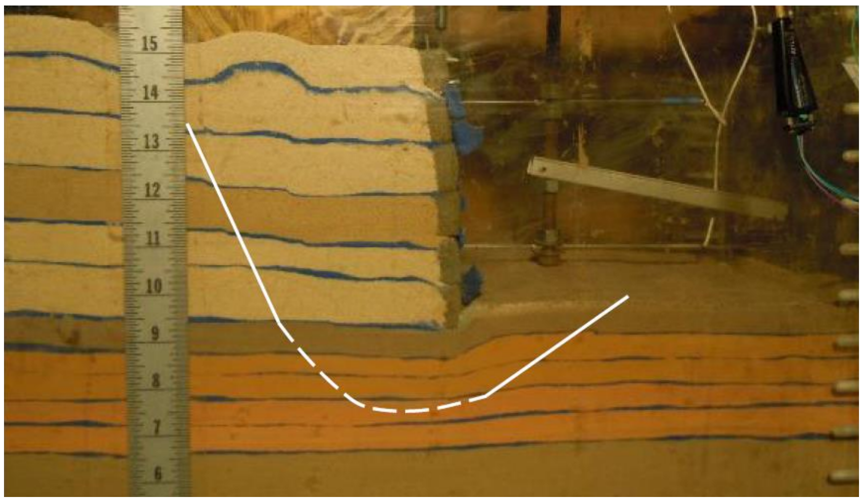
- Good correlation between $V_{measured}$ and $V_{calculated}$
- Model is accurate representation
- Useful for investigating S_2, eccentricity (e) and angle of inclination (δ)



- Backfill μ_γ and μ_ϕ range 93 pcf – 99 pcf and 28° and 33°

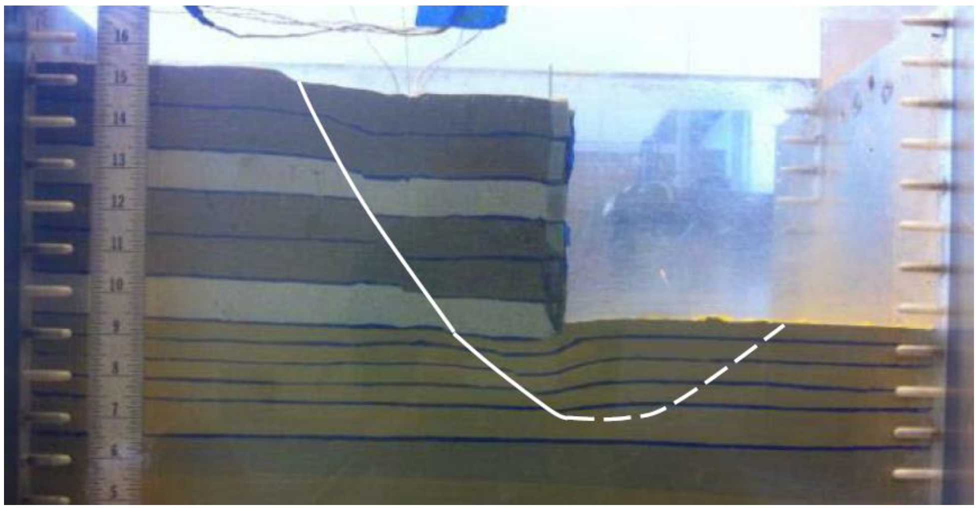
Effects of Load Inclination

- Inclined loads (T in MSE wall and wedge) reduce length of bearing rupture surface i.e., reduced capacity
- Sokolovski (1960) showed analytically for $\delta = 0^\circ - 20^\circ$ depth of rupture 0.78L to 0.3L and lateral extents 1.9L to 0.6L, respectively



Rupture surface in test 15 ($\delta = 30^\circ$ and $\mu_\phi = 28^\circ$): Dashed line is the estimated surface, Solid line is offset from observed surface

- Depth of rupture ≈ 0.5L
- Lateral extent ≈ 0.67L



Rupture surface in test 42 ($\delta = 25^\circ$ and $\mu_\phi = 28^\circ$): Dashed line is the estimated surface, Solid line is offset from observed surface

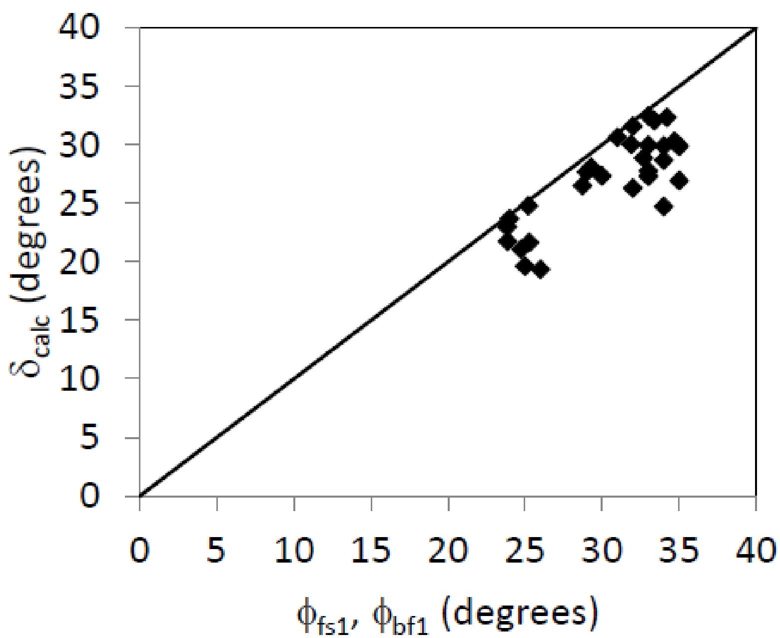
- Depth of rupture ≈ 0.7L
- Lateral extent > 0.67L

Angle of Load Inclination, δ

- Horizontal force equilibrium based on force polygon gives S_2

$$S_2 = (W_1 + W_2 + W_3 + Q_S - V_{meas})\tan(\theta - \phi)$$

- Using $V_{measured}$ and $\tan^{-1}(S_2/V_{measured})$, δ is back calculated for comparison to the smaller of ϕ_{fs} and ϕ_{bf} at interface between foundation soil and backfill

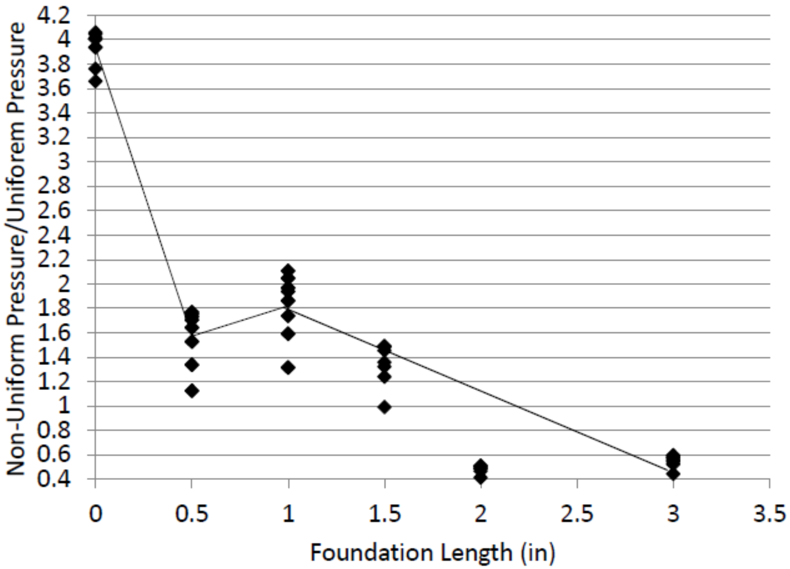


- Solid line is upper bound limit ($\delta = \phi$)
- All tests show $\delta_{calculated} < \phi_{fs}$ or ϕ_{bf}

Vertical Dead Load Factor, γ_{DL}

- Vertical dead load from vertical earth pressure $\gamma_{DL} = \gamma_{EV}$
- Load factor calculated with bias (λ) and CV of load (Nowak, 1995) and n = 2 (AASHTO, 2009)

$$\gamma = \lambda(1 + nCV)$$



$$\frac{\sigma'_{nu}}{\sigma'_V} = 0.49(x) + 1.3 \quad \text{for } 0.5 < x \leq 1 \text{ inch}$$

$$\frac{\sigma'_{nu}}{\sigma'_V} = -0.61(x) + 2.4 \quad \text{for } 1 < x \leq 3 \text{ inch}$$

$$\sigma'_V = \gamma'_s z$$

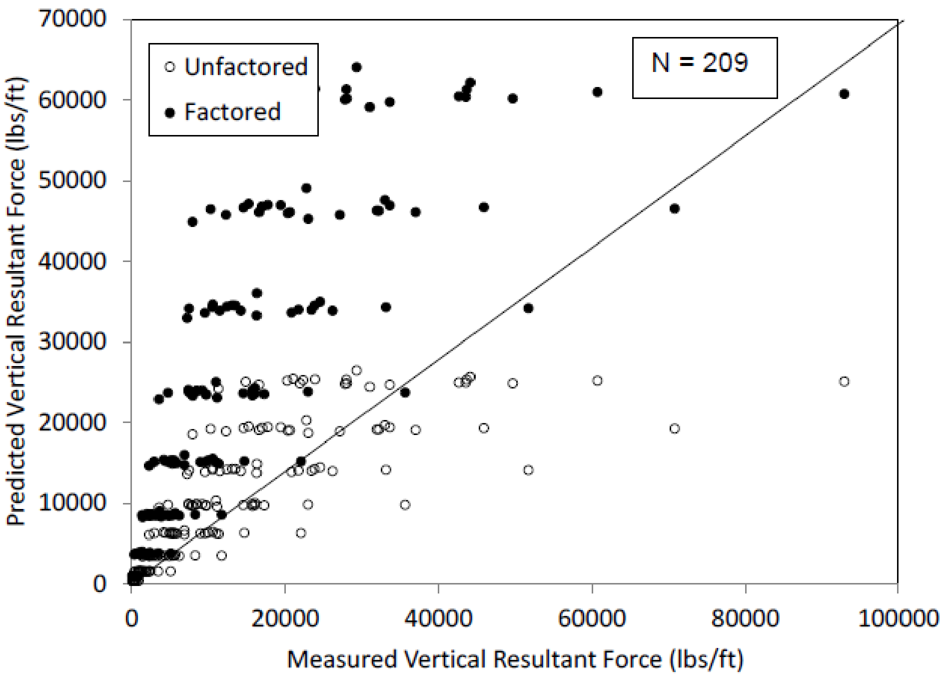
γ_s' = soil's effective unit weight

z = depth of overburden (H)

- For λ = 0.96, CV = 0.42 γ_{EV} = 1.80
- Bathurst et. al. proposed γ_{EV} = 1.75 from 34 tests on full scale MSE walls

Vertical Dead Load Factor, γ_{DL}

- Factoring the predicted load (applied vertical resultant) with γ_{EV} = 1.80 brings almost all points above 1:1 line



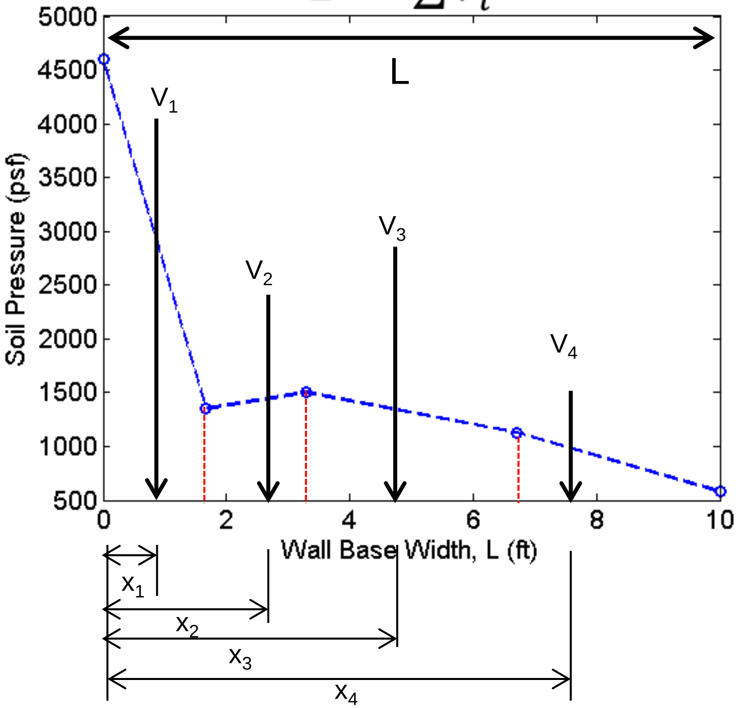
Eccentricity, e

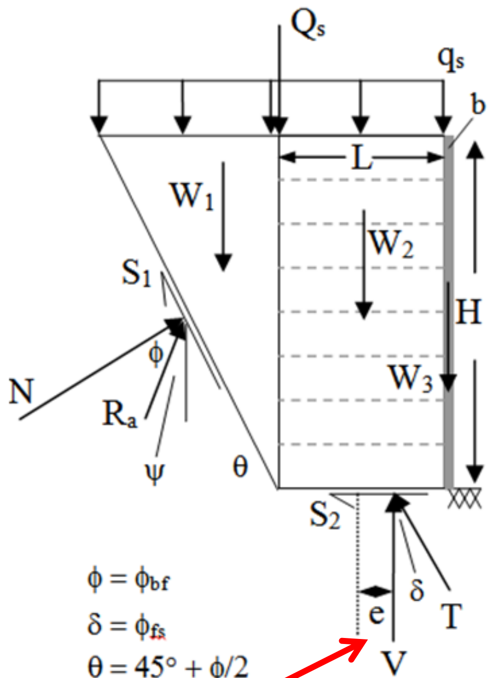
- Capacity equation: $q_u = \frac{1}{2}\gamma L' N_\gamma i_\gamma$

  where L' = L-2e

Calculated from measured soil pressure distributions

$$e = \frac{L}{2} - \frac{\sum V_i x_i}{\sum V_i}$$





$$e = \frac{L}{2} - \frac{M_r - M_o}{V}$$

Calculated from estimated moments and resultant vertical force

R^2 = 0.62 between measured and predicted

Bearing Capacity Prediction

- Bearing capacity equation

$$q_u = \frac{1}{2}\gamma L' N_\gamma i_\gamma$$

- Evaluation with 7 soil self weight factors, N_γ :

- Meyerhof's

$$N_\gamma = (N_q - 1)\tan(1.4\phi)$$

- Hansen's

$$N_\gamma = 1.5(N_q - 1)tan(\phi)$$

- Vesic's

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

- Salgado's

$$N_\gamma = (N_q + 1)tan(1.32\phi)$$

- Eurocode 7 (2005)

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

- Michalowski (1997)

$$N_\gamma = e^{(0.66+1\tan(\phi))}tan(\phi)$$

- Bolton et. al.(1993)

$$N_\gamma = (N_q - 1)tan(1.5\phi)$$

where

$$N_q = e^{\pi\tan\phi}tan^2\left(45° + \frac{\phi}{2}\right)$$

Bearing Capacity Prediction

- Evaluation of 4 methods of load inclination factor, i_γ:

- Hansen's

$$i_\gamma = \left(1 - \frac{0.7S_2}{V}\right)^\eta$$

$2 \leq \eta \leq 5$ (Bowles, 1996)

η = 2 herein

- Muhn's

$$i_\gamma = (1 - \tan(\delta))^\eta$$

η = 1

- Vesic's

$$i_\gamma = \left(1 - \frac{S_2}{V}\right)^{m+1}$$

m = (2+L/B)/(1+L/B)

where

L = foundation width

B = unit length

for these tests, m =1.09

- New

$$\frac{V_{meas}}{V_{qu_{pred}}} = \left(1 - \frac{S_2}{V}\right)^\eta$$

$$i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.08}, \quad 26 < \phi_{found} < 30$$

$$i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.55}, \quad 31 < \phi_{found} < 33$$

LRFD Φ for Bearing Stability

$$\Phi = \frac{\lambda_R \cdot \sqrt{\frac{(1+CV_Q^2)}{(1+CV_R^2)}}(\gamma_D \cdot q_D + \gamma_L \cdot q_L)}{(\lambda_D \cdot q_D + \lambda_L \cdot q_L) \cdot e^{\beta_T \sqrt{\ln[(1+CV_R^2)(1+CV_Q^2)]}}}$$

$$CV_Q^2 = \frac{q_D^2 \cdot E[\lambda_D]^2 \cdot CV_D^2 + q_L^2 \cdot E[\lambda_L]^2 \cdot CV_L^2}{q_L^2\left(\frac{q_D^2}{q_L^2} E[\lambda_D]^2 + 2\frac{q_D}{q_L} E[\lambda_D] \cdot E[\lambda_L] + E[\lambda_L]^2\right)}$$

Summary statistics and factors used in estimating Φ

CV_D	CV_L	q_D (lbs/ft)	q_L (lbs/ft)	γ_D	γ_L	λ_D	λ_L
0.42	0.42	32,314	1,144	1.80	1.75	0.96	1.2

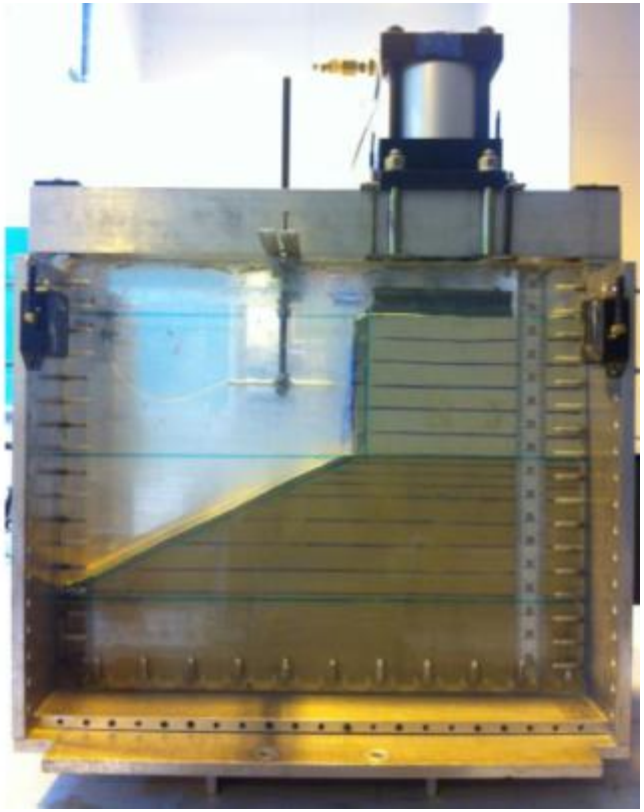
resistance factors (Φ) for $\mu_{\phi fs}$ = 26° - 30° using the new i_γ

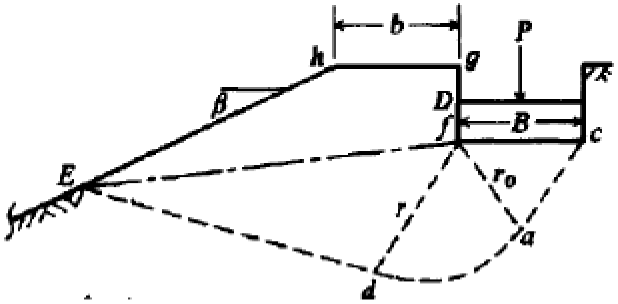
		Meyerhof	Hansen	Vesic	Salgado	Euro7	Michalowski	Bolton
CV_R		0.450	0.444	0.433	0.437	0.444	0.441	0.452
λ_R		1.93	1.98	1.29	1.82	1.48	1.39	1.75
P_f =	Φ	0.986	1.020	0.682	0.956	0.765	0.721	0.889
1%	Φ/λ_R	0.510	0.516	0.528	0.524	0.516	0.520	0.508
P_f =	Φ	0.668	0.694	0.467	0.652	0.520	0.491	0.602
0.1%	Φ/λ_R	0.346	0.351	0.361	0.358	0.351	0.354	0.344

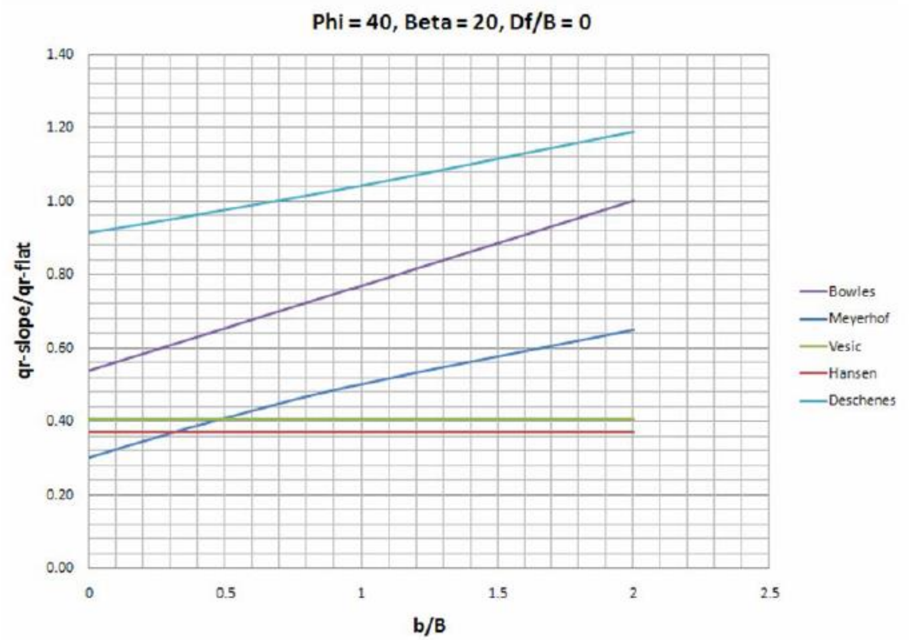
resistance factors (Φ) for $\mu_{\phi fs}$ = 31° - 33° using the new i_γ

		Meyerhof	Hansen	Vesic	Salgado	Euro7	Michalowski	Bolton
CV_R		0.440	0.436	0.431	0.434	0.436	0.436	0.442
λ_R		1.71	1.80	1.23	1.70	1.35	1.27	1.53
P_f =	Φ	0.890	0.945	0.652	0.896	0.709	0.667	0.793
1%	Φ/λ_R	0.521	0.525	0.530	0.527	0.525	0.525	0.519
P_f =	Φ	0.607	0.645	0.447	0.613	0.484	0.455	0.540
0.1%	Φ/λ_R	0.355	0.359	0.363	0.360	0.359	0.359	0.353

Tests of MSE Walls on Embankments





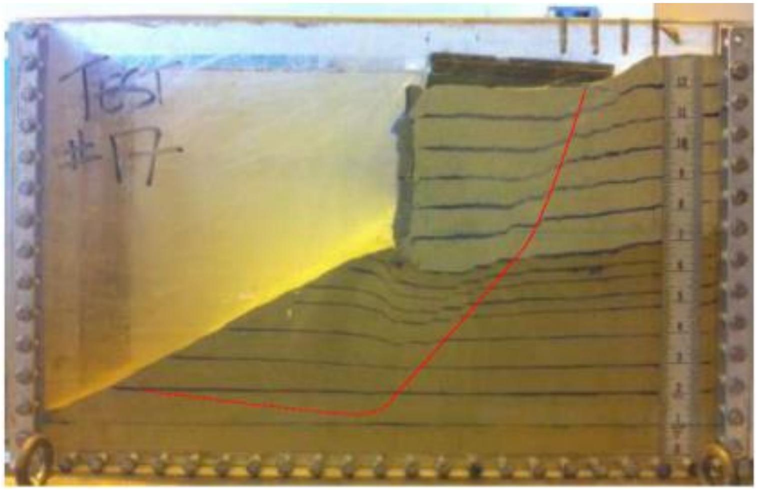


Tests of MSE Walls on Embankments

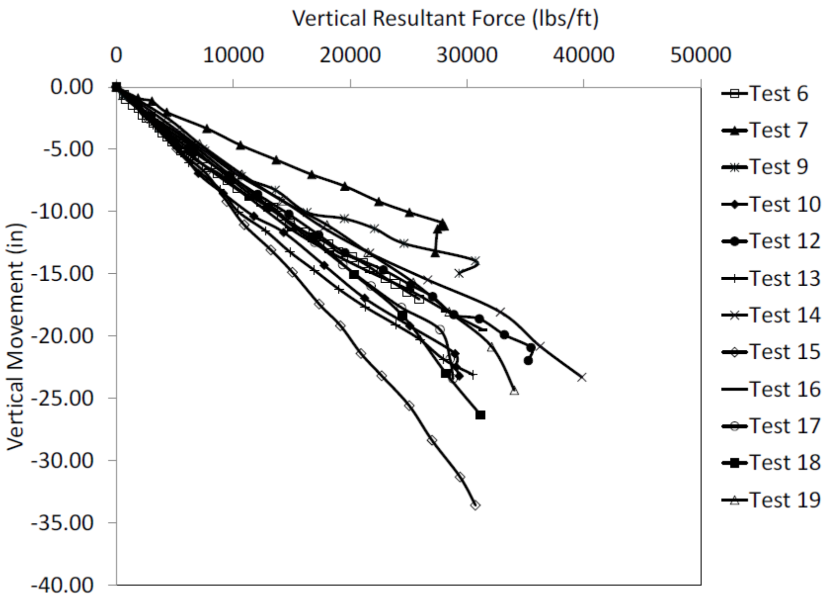
- Models with $\mu_{\phi fs}$ = 26° -27° being evaluated for CV_R
- Previous tests had $\mu_{\phi fs}$ > 32 and did not result in bearing failures and had moisture contents

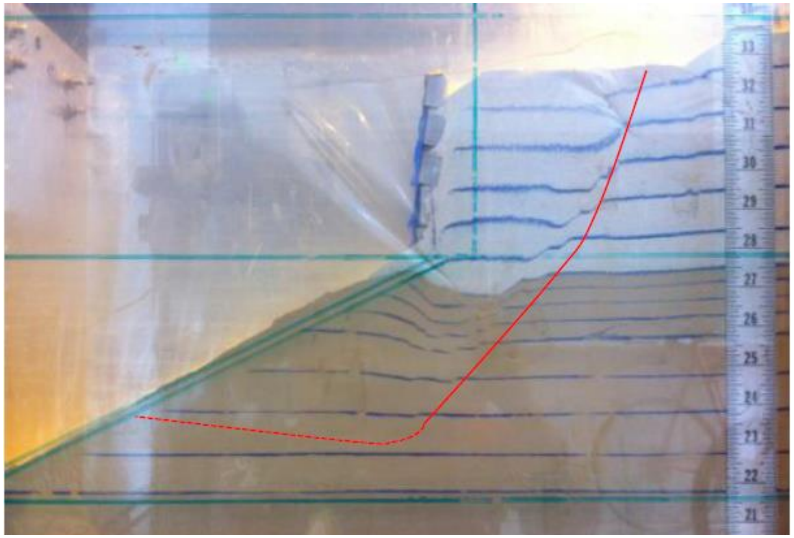
Test	Backfill				Embankment (Foundation Soil)			
	Unit Weight (pcf) Mean (μ)	Unit Weight CV (%)	Friction Angle (°) Mean (μ)	Friction Angle CV (%)	Unit Weight (pcf) Mean (μ)	Unit Weight CV (%)	Friction Angle (°) Mean (μ)	Friction Angle CV (%)
1	95	7	30	7	100	5	32	5
2	94	8	28	12	105	5	34	5
3	103	6	36	5	105	5	33	5
4	98	1	37	2	107	2	34	2
5	97	1	37	2	107	2	34	2
6	98	2	38	5	86	1	26	1
7	99	13	38	2	90	5	27	4
8	98	1	38	2	86	3	26	2
9	98	1	38	2	85	2	26	2
10	97	1	37	3	82	2	26	2
11	97	1	37	2	90	2	27	2
12	97	1	37	3	82	2	26	2
13	98	1	38	3	85	1	26	1
14	95	1	35	1	83	3	26	2
15	95	1	35	1	83	2	26	2
16	96	1	36	4	84	10	26	3
17	97	2	37	4	82	2	26	2
18	98	4	38	9	82	4	26	4
19	97	2	37	4	85	3	26	2

Observed Load-Displacement and Failures



Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 17



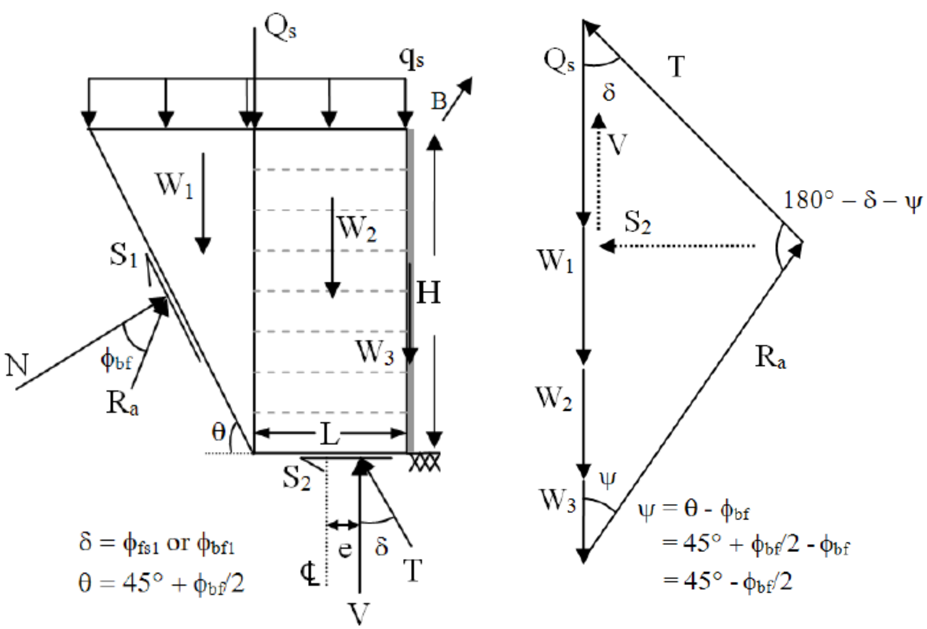


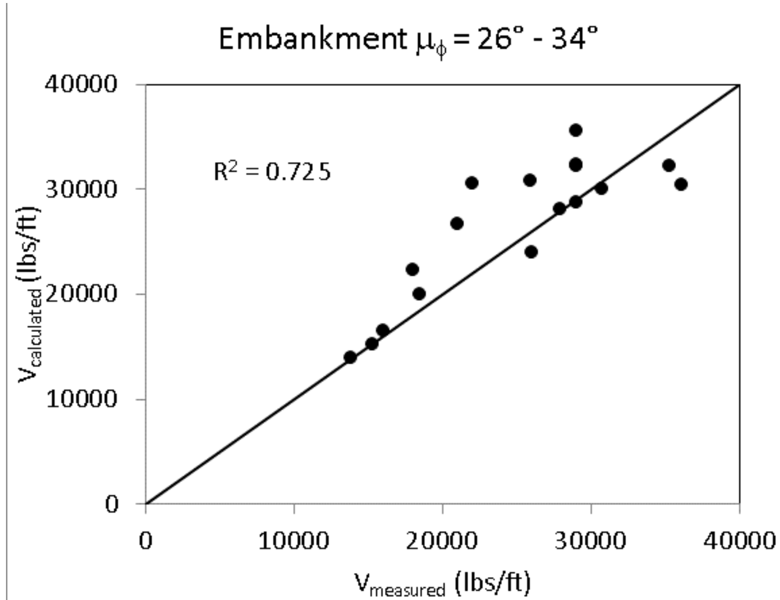
Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 10

Observed rupture on surface of embankment in Test 10

Validation-Force Equilibrium

- Correlation between $V_{measured}$ and $V_{calculated}$





$$V = \left[\frac{W_1 + W_2 + W_3 + Q_S}{\frac{\sin(\delta)}{\tan(\theta - \phi)} + \cos(\delta)}\right] \cos(\delta)$$

Bearing Capacity Prediction

- Bearing capacity equation

$$q_{u_{pred}} = \frac{1}{2}\gamma L' N'_\gamma i_\gamma$$

- Evaluation with 3 modified soil self weight factors, N_γ' :

- Bowles (1996)

$$N_\gamma' = \frac{N_\gamma}{2} + \frac{N_\gamma}{2}\left[R + \frac{b}{2L}(1-R)\right]$$

where R is ratio of K_{pmin}/ K_{pmax} = K(-β)/K(+β)

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

- Hansen's

$$g_\gamma = (1 - 0.5\tan\beta)^5$$

- Vesic's

$$g_\gamma = (1 - \tan\beta)^2$$

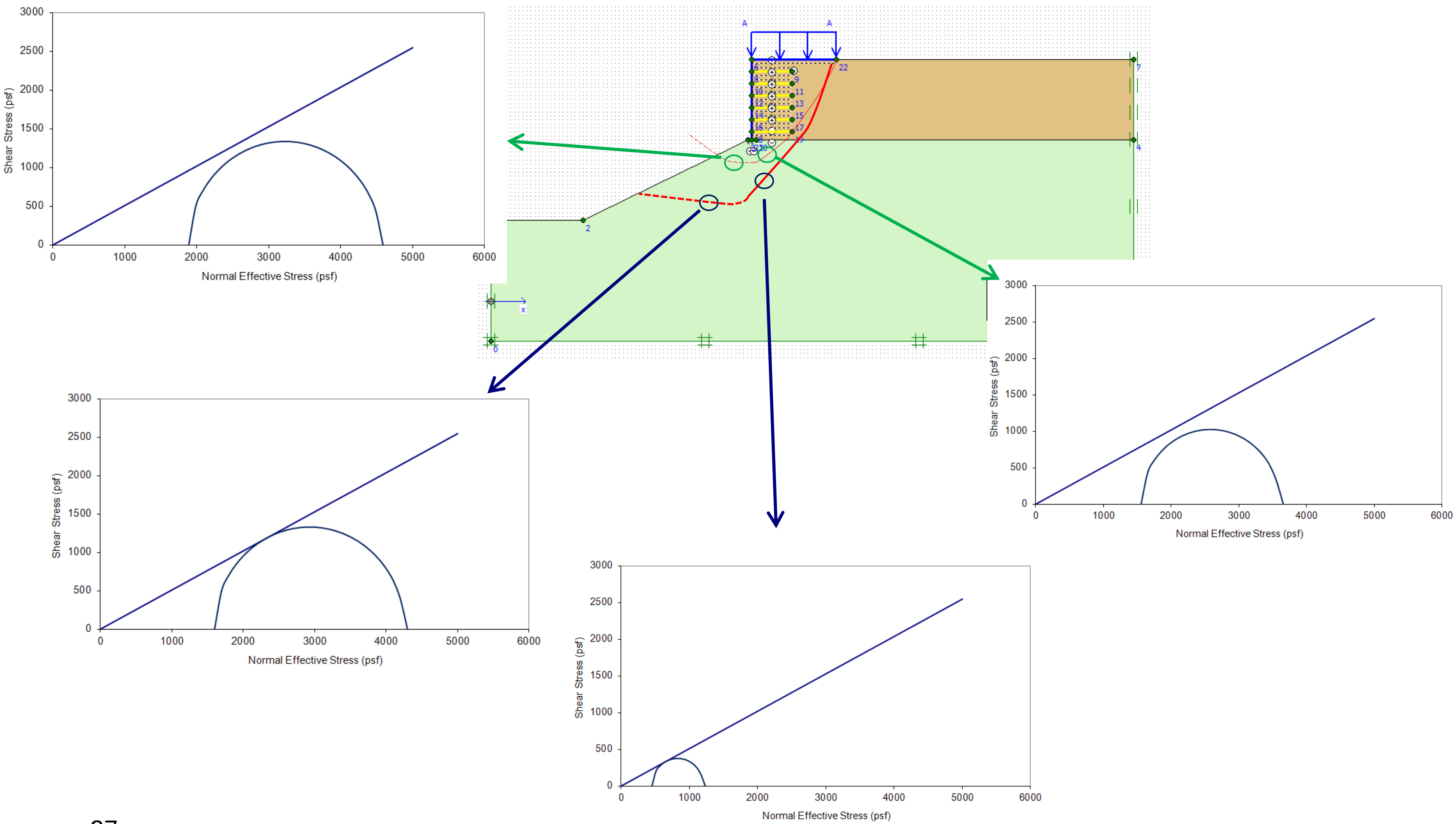
where $N'_\gamma = N_\gamma g_\gamma$

Bearing Capacity Prediction

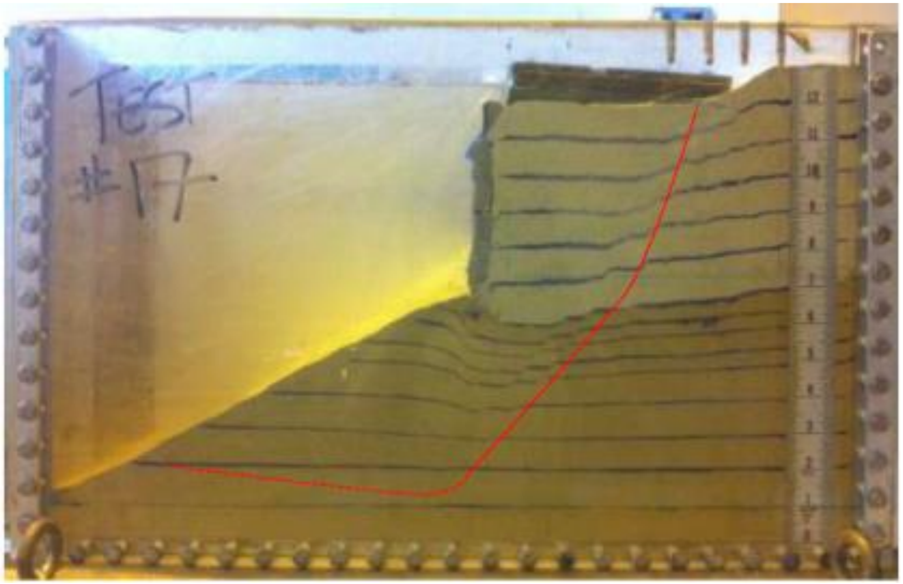
Test	Mean Unit Weight (lbs/ft^2)	Mean Friction Angle (°)	$V_{u\ Hansen}$ (kips/ft)	$V_{u\ Vesic}$ (kips/ft)	$V_{u\ Bowles}$ (kips/ft)	V_{meas} (kips/ft)
1	100	32	2.0	2.3	3.0	13.8
2	105	34	3.6	5.4	7.2	18.5
6	86	26	3.7	2.3	7.5	25.9
7	90	27	4.5	2.8	9.2	27.9
9	86	26	3.5	2.0	7.2	30.7
10	85	26	3.6	2.3	7.4	29.0
11	90	27	3.5	5.8	9.0	29.0
12	82	26	3.5	2.2	7.2	28.5
13	85	26	3.6	2.3	7.4	28.2
14	83	26	3.4	5.7	7.0	32.9
15	83	26	3.4	5.7	7.0	24.5
16	84	26	3.3	5.5	6.8	18.0
17	82	26	3.4	5.7	6.9	28.0
18	82	26	2.9	5.0	5.6	25.0
19	85	26	4.1	6.8	8.3	32.0

- Predictions based on Bowles method use Hansen's N_γ and i_γ, giving the lowest bias (λ) = 3.4
- Vesic's gives bias (λ) = 4.3
- Hansen's gives bias (λ) = 7.2
- If a bearing capacity problem, all methods highly over conservative
- Extents of rupture surfaces suggest failures exhibiting a deeper rupture due to the combined shear on vertical and horizontal plane from slopes and MSE wall

Plaxis Analysis: MSE Walls on Embankments



Observed Rupture Surfaces



Observed rupture surface (solid line) and estimated rupture surface (dashed line) in Test 17

Observed rupture on surface of embankment in Test 10

Conclusions and Recommendations

Sliding Stability

- Resistance factors (Φ)determined for wall heights of 8 ft, 11 ft and 14 ft, L/H = 1, and backfill properties of μ_ϕ = 31.5° CV$_\phi$ = 11%
- Φ ranged from 0.74 – 0.94 using Rankine's loading and 0.62 – 0.67 using Coulomb's loading
- Remaining work includes calculating horizontal load factor (γ_{EH}) for influence on Φ's

Bearing Stability

- Recommend Vertical load factor (γ_{EV}) = 1.80 be used based on 209 measurements of vertical force. Current practice γ_{EV} = 1.35 (AASHTO, 2009).
- Observed rupture surfaces supported the use of load inclination factors, i_γ
- Recommended $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.08}$, $26 < \phi_{found} < 30$ $i_\gamma = \left(1 - \frac{S_2}{V}\right)^{1.55}$, $31 < \phi_{found} < 33$
- Resistance factors (Φ)determined for wall height 20 ft with L/H = 0.5, and foundation soil properties of μ_ϕ = 26° - 30° and 31° - 33° with CV$_\phi$ = 5%
- For μ_ϕ = 26° - 30° Φ = 0.47 and for 31° - 33° Φ = 0.45

Conclusions and Recommendations

MSE Walls on Embankments

- Centrifuge tests of MSE wall on embankment ongoing
- 14 tests exhibited failure – 12 tests with $\mu_{\phi fs}$ = 26° -27°
- Tests with $\mu_{\phi fs}$ = 26° -27° exhibited deeper rupture surfaces due to the combined shear on vertical and horizontal plane from slopes and MSE wall
- Bearing capacity prediction methods which account for ground inclination, g_{γ}, (Bowles, Meyerhof, Hansen, and Vesic) are highly over conservative
- Current methods (Bowles, Meyerhof, Hansen, and Vesic) lead to bias, λ, > 3
- Tests suggest bearing capacity of MSE walls on embankments not an issue (validated with Plaxis model)
- Stability analysis should look at equilibrium for deeper ruptures

Publications

- Wasman, S.J., McVay, M.C., Bloomquist, D., Lai, P., Jones, L. and Herrera, R., "Determination of LRFD Vertical Load and Resistance Factors for Bearing Capacity of Mechanically Stabilized Earth Walls in Granular Soils", *Geotextiles and Geomembranes*, *(In Preparation)*

Thank You!

Questions?