WVDOH IMPLEMENTATION 2006 AASHTO LRFD SECTION 10 INTERIMS

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Background

- Mid 2005
  - Comprehensive LRFD foundation policy
  - Specific to Section 10
    - Spread Footers
    - Drilled Shafts
    - Piling
- 80-90% Complete
- Recent ’06 interims
  - NCHRP 24-17, aka NCHRP 507

Current Policy/General Site

- State Policy:
  - *“All bridge foundations shall bear on competent rock”*
    - 70 to 80% of the state, rock within the first 15’-20’
    - 20 to 30%, 30’-50’ of overburden

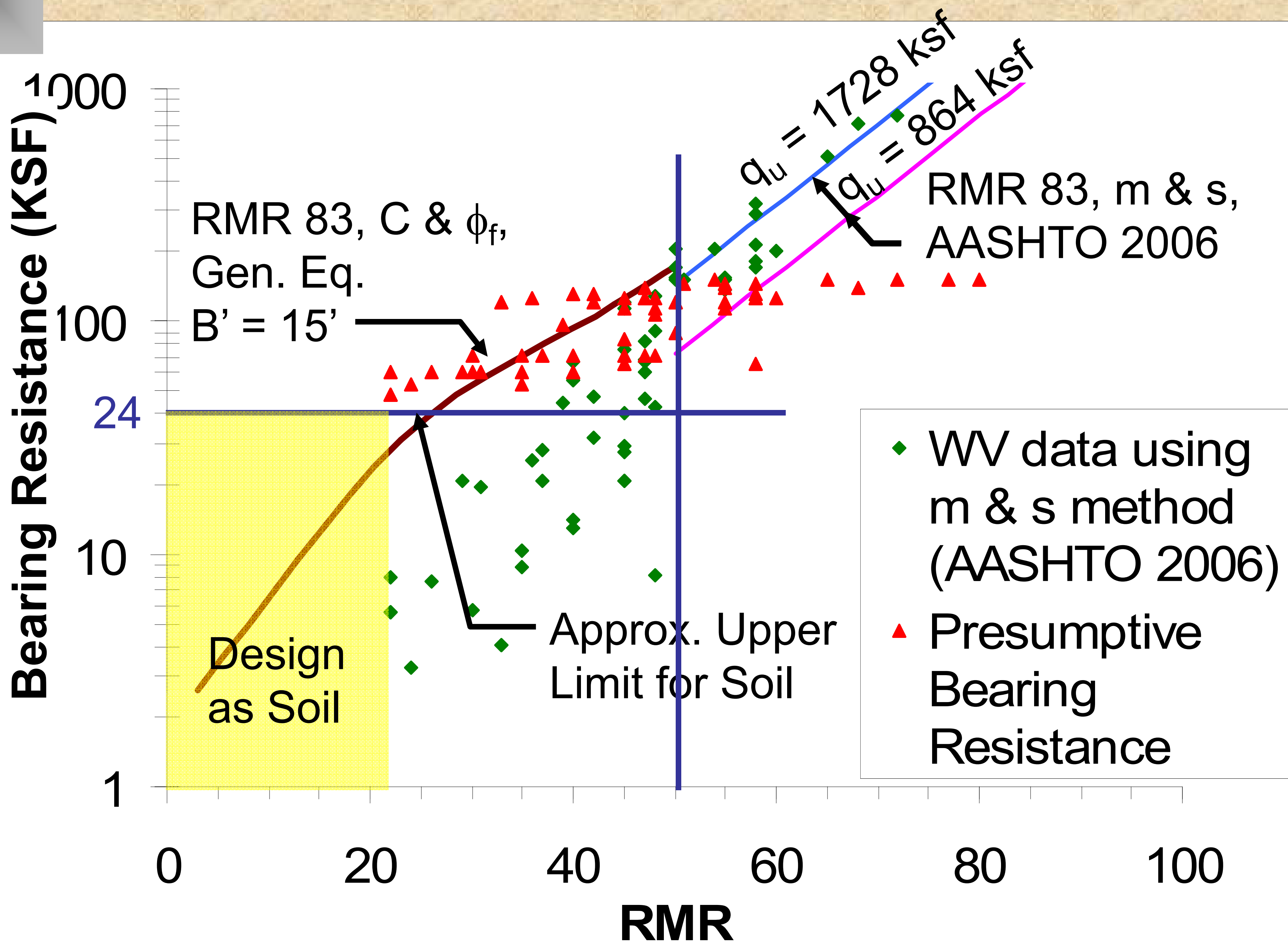
- Sedimentary Rock
    - shale
    - siltstone
    - claystone
    - sandstone

Highlights

- RMR 83
  - Uniform and comprehensive approach to characterize rock
  - RMR parameters latter used in the resistance equations
- Characterization Criteria for Rock
- Methods used to check Limit States
- Design equations
- Design examples
- dBase tracking system

RMR 83

- Transition from Uc/RQD/Presumptive methods and go towards RMR 83
  - Uniform and comprehensive approach to characterize rock
  - RMR values are latter used in the resistance equations
- Correlated previously developed geotechnical data and AASHTO presumptive values
- Taking a “three tier” approach w/ RMR
  - RMR>50
  - 20<RMR<50
  - RMR<20 (soil)



Bearing Resistance (KSF)
1000
100
24
10
1
0
20
40
60
80
100
RMR
q_u = 1728 ksf
q_u = 864 ksf
RMR 83, m & s, AASHTO 2006
RMR 83, C & ϕ_f, Gen. Eq. B' = 15'
Design as Soil
Approx. Upper Limit for Soil
WV data using m & s method (AASHTO 2006)
Presumptive Bearing Resistance

Spread Footings

Strength Limit State: *nominal bearing resistance*

- Nominal Bearing Resistance (q_n)

  [RMR > 50]

$$q_n = \left[\sqrt{s} + \sqrt{\left(m\sqrt{s} + s \right)} \right] U_c$$

eqn. 10.8.3.5.4c-2

m & s: use RMR value with Table 10.4.6.4-4

[20 < RMR < 50]

$$q_n = cN_{cm} + .5 g\gamma B N_{\gamma m}$$

eqn. 10.6.3.1.2a-1

$$c = 104 \times RMR$$

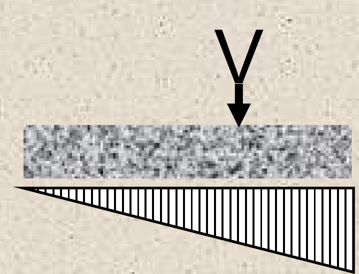
$$\phi_f = 5 + \frac{RMR}{2}$$

Spread Footings

Strength Limit State: *nominal bearing resistance*

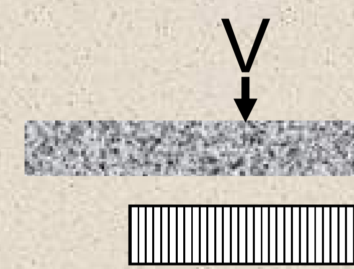
- Nominal Bearing Capacity cont…

[RMR > 50]



- (q_n) compared to the applied Strength Limit bearing stress from triangular/trapezoidal distribution

[20 < RMR < 50]



- Water table correction factors not required
- Generally, surcharge won't be used
- (q_n) compared to the applied Strength Limit bearing stress using equivalent B'

Spread Footings

Strength Limit State: *sliding 10.6.3.4*

- Sliding
  - Generally δ = 30-35°
  - Resistance Factor φ = 0.90
    - DL/LL > 3.0

Table C10.4.6.4-1. Typical ranges of friction angles for smooth joints in a variety of rock types (Modified after *Barton, 1976*; *Jaeger and Cook, 1976*)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

Spread Footings

Service Limit State: *Vertical Movement*

- Vertical Movement
  - Per 10.6.2.4.4 to calculate settlement,
  - Use 10.4.6.5 to calculate Em

$$\rho = q_0\left(1-\upsilon^2\right)\frac{B'I_p}{144E_m}$$

$$E_m(ksi) = 145\left(10^{\frac{RMR-10}{40}}\right)$$

Spread Footings

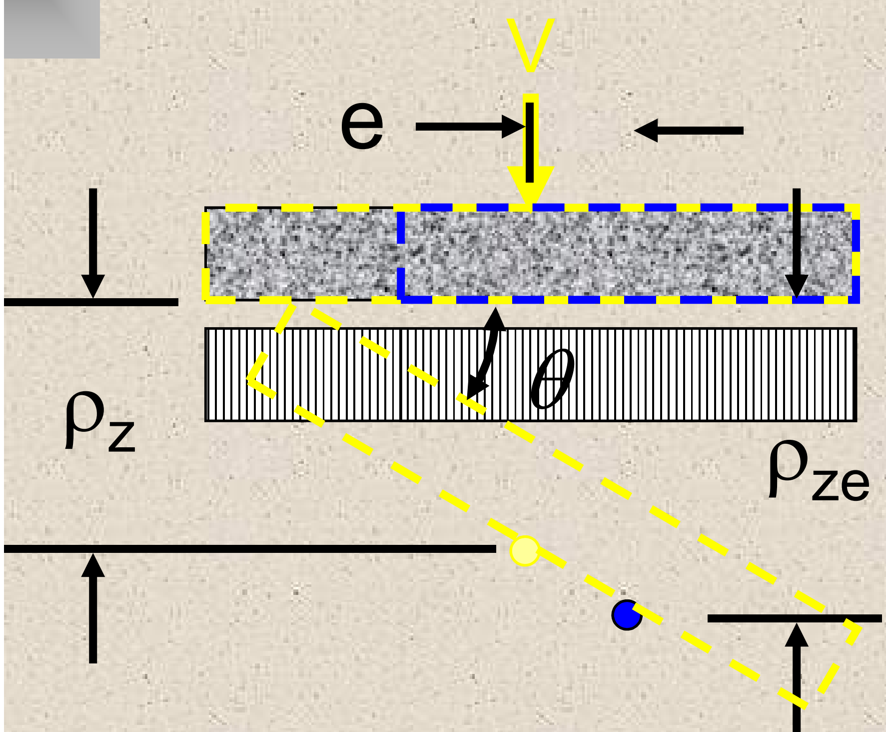
Service Limit State: *Rotational Movement*

- Rotational Movement

1. Compute displacement at the center of footing assuming a concentric loading (uniform load over entire footing area)

2. Compute displacement at the center of the effective footing area.

3. Compute rotation in each direction independently



Eccentric Displacement -

$$\rho_{ze} = \frac{\left(1-\nu^2\right) * V}{E_m \beta_z} \left(\frac{\sqrt{(B-2e_b)L}}{(B-2e_b)L} \right)$$

Center Displacement -

(AASHTO 10.6.2.4.2-1)

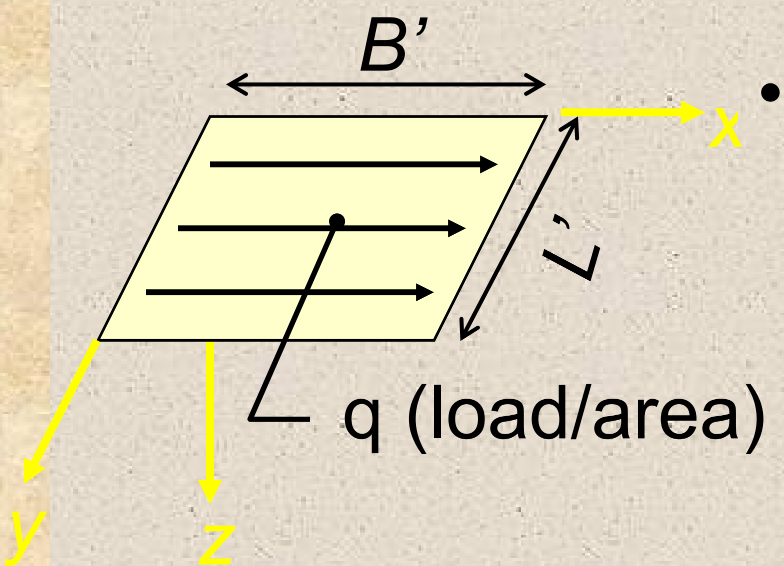
$$\rho_z = \frac{\left(1-\nu^2\right) * V}{E_m \beta_z} \left(\frac{\sqrt{BL}}{BL} \right)$$

Footing Rotation -

$$\theta = \tan^{-1}\left(\frac{\rho_{ze} - \rho_z}{e_b} \right)$$

Spread Footings

Service Limit State: *Horizontal Movement*



- Horizontal Movement

Horizontal displacement (Δx) at the center (B/2) of the rectangular loaded area is given by the following equation (Giroud, 1969)

$$\Delta x = \frac{1+\upsilon}{\pi E_m} q B' \left[2(1-\upsilon)\ln\frac{L'+\sqrt{B'^2+L'^2}}{B'} + \frac{L'}{B'}\ln\frac{B'+\sqrt{B'^2+L'^2}}{-B'+\sqrt{B'^2+L'^2}} \right]$$

Elastic modulus of rock mass

Drilled Shafts

Drilled Shafts

Strength Limit State

- **Combine both end bearing and side resistance**
- **Use soil-interaction for lateral resistance when site conditions permit**

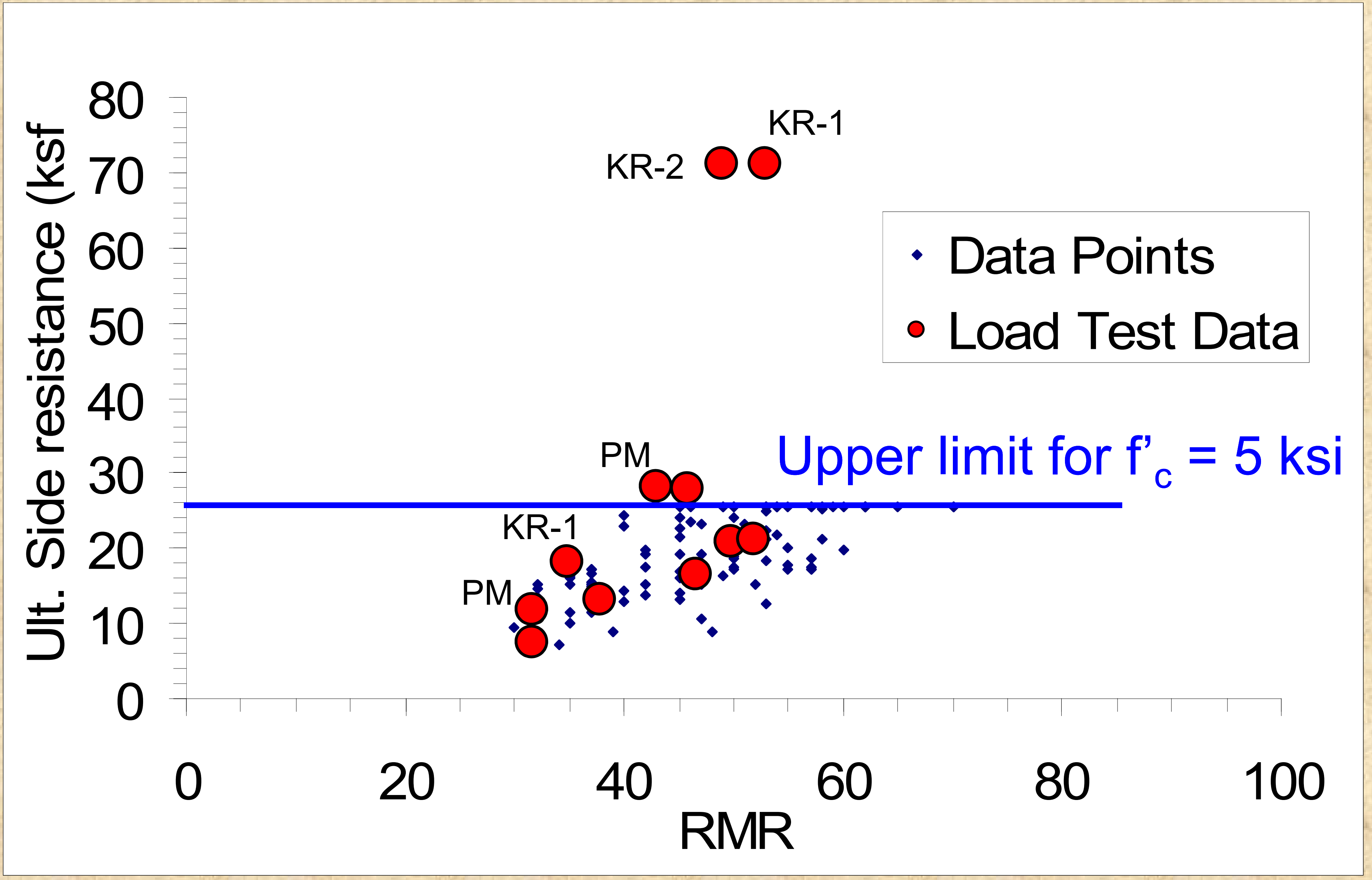
Drilled Shafts

Strength Limit State: *nominal side resistance*

- AASHTO LRFD 10.8.3.5.4b-1

$$q_s = 0.65\alpha_E p_a \sqrt{\left(q_u / p_a \right)} \le 7.8 p_a \sqrt{\left(f'_c / p_a \right)}$$

Side Resistance Correlation



Drilled Shafts

Strength Limit State: *nominal bearing resistance*

- Nominal End Bearing Resistance (q_n)

[RMR > 50]

$$q_n = \left[\sqrt{s} + \sqrt{\left(m\sqrt{s} + s\right)}\right] U_c$$

eqn. 10.8.3.5.4c-2

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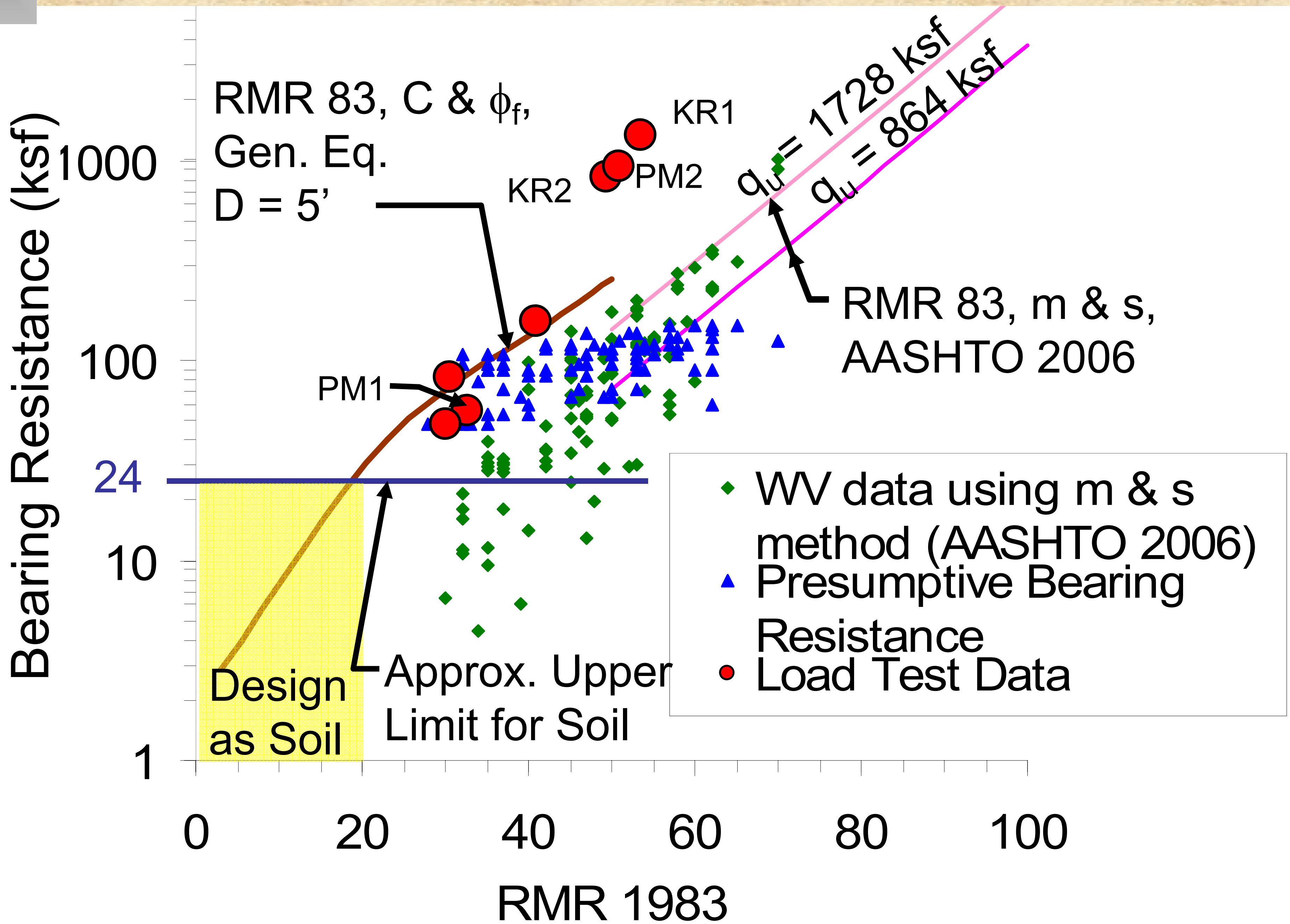
$$\phi_f = 5 + \frac{RMR}{2}$$


RMR 83, C & ϕ_f,
Gen. Eq.
D = 5'
KR1
KR2
PM2
PM1
q_u = 1728 ksf
q_u = 864 ksf
RMR 83, m & s,
AASHTO 2006
Bearing Resistance (ksf)
1000
100
24
10
1
Design
as Soil
Approx. Upper
Limit for Soil
WV data using m & s
method (AASHTO 2006)
Presumptive Bearing
Resistance
Load Test Data
0
20
40
60
80
100
RMR 1983

Drilled Shafts

Strength Limit State: *combined side and end*

- **80:20, 70:30 "rules of thumb"**
- **FHWA-IF-99-025 (O'Neil & Reese) Appendix C**

$w_T = w_{T1} + \Delta w$ where (C.35)

$w_{T1} + \Delta w = F_3 \, (Q_T / \pi E_m B) - F_4 B$ where

$F_3 = a_1 \, (\lambda_1 B \, C_3 - \lambda_2 B \, C_4) - 4 \, a_3,$

$F_4 = \left[1 - a_1\left(\frac{\lambda_1 - \lambda_2}{D_4 - D_3}\right) B\right] a_2 \left(\frac{c}{E_m}\right)$

$a_1 = (1+\nu_{concrete}) \ln [5 \, (1-\nu) \, (D/B)] + a_2$ (C.39)

$a_2 = [(1-\nu_{concrete})(E_m/E_c) + (1+\nu)][1/(2 \tan \phi \tan \psi)]$ (C.40)

$\lambda_1 = [-\beta + (\beta^2 + 4\alpha)^{0.5}] / 2\alpha$ (C.41)

$\lambda_2 = [-\beta - (\beta^2 + 4\alpha)^{0.5}] / 2\alpha$ (C.42)

$\beta = a_3 \, [E_c/E_m] \, B$ (C.43)

$a_3 = [\nu_{concrete} / 2 \tan \psi] \, (E_m/E_c)$ (C.44)

$\alpha = a_1 \, (E_c / E_m) \, (B^2/4)$

$C_3 = D_3 / (D_4 - D_3)$

$C_4 = D_4 / (D_4 - D_3)$

$D_3 = [\pi(1-\nu^2) \, (E_m / E_b) + 4a_3 + a_1 \lambda_2 B] \exp (\lambda_2 D)$

$D_4 = [\pi(1-\nu^2) \, (E_m / E_b) + 4a_3 + a_1 \lambda_1 B] \exp (\lambda_1 D)$

$w_T = F_5 \, [Q_T / \pi \, E_m \, B] + F_6 \, B$ (C.53)

in which

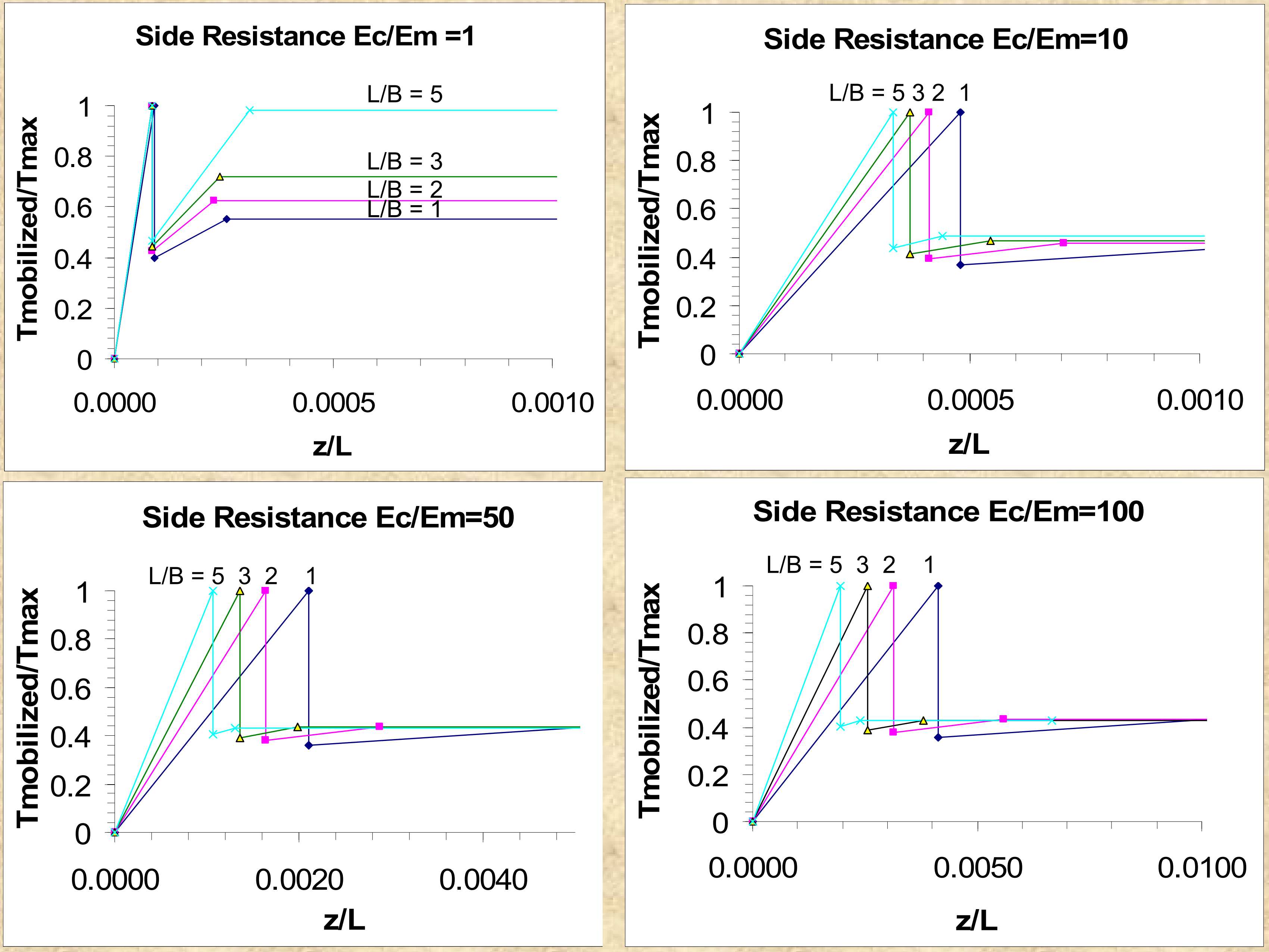
$F_5 = 4a_3 - a_1(\lambda_1 B \, C_5 - \lambda_2 B \, C_6)$ (C.54)

$F_6 = a_2 \, (c/E_m)$ (C.55)

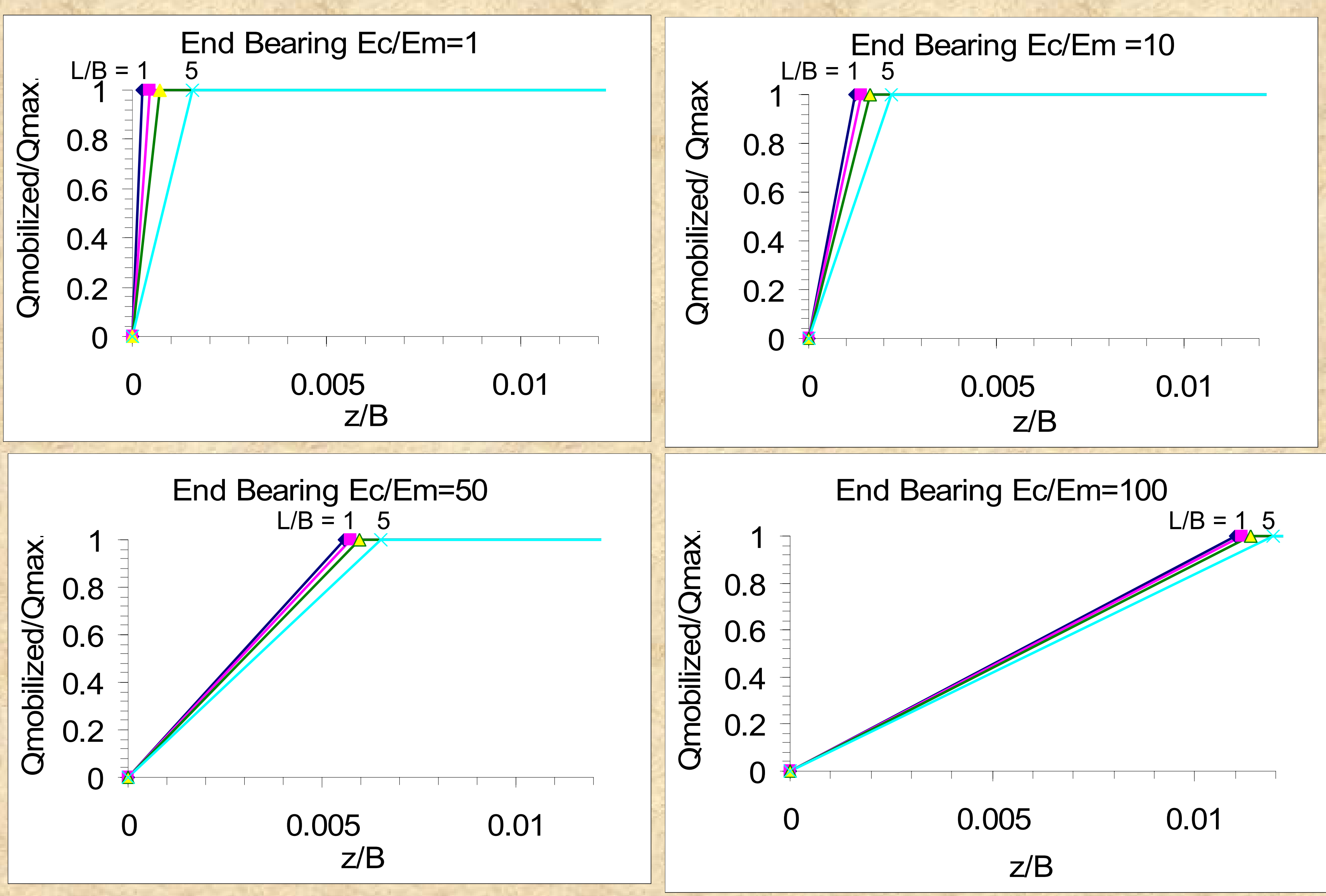
$C_5 = \exp [-\lambda_2 D] / \{\exp [-\lambda_1 D] - \exp [-\lambda_2 D]\}$ and (C.56)

$C_6 = \exp [-\lambda_1 D] / \{\exp [-\lambda_1 D] - \exp [-\lambda_2 D]\}$ (C.57)

Recommendation for T-z plots

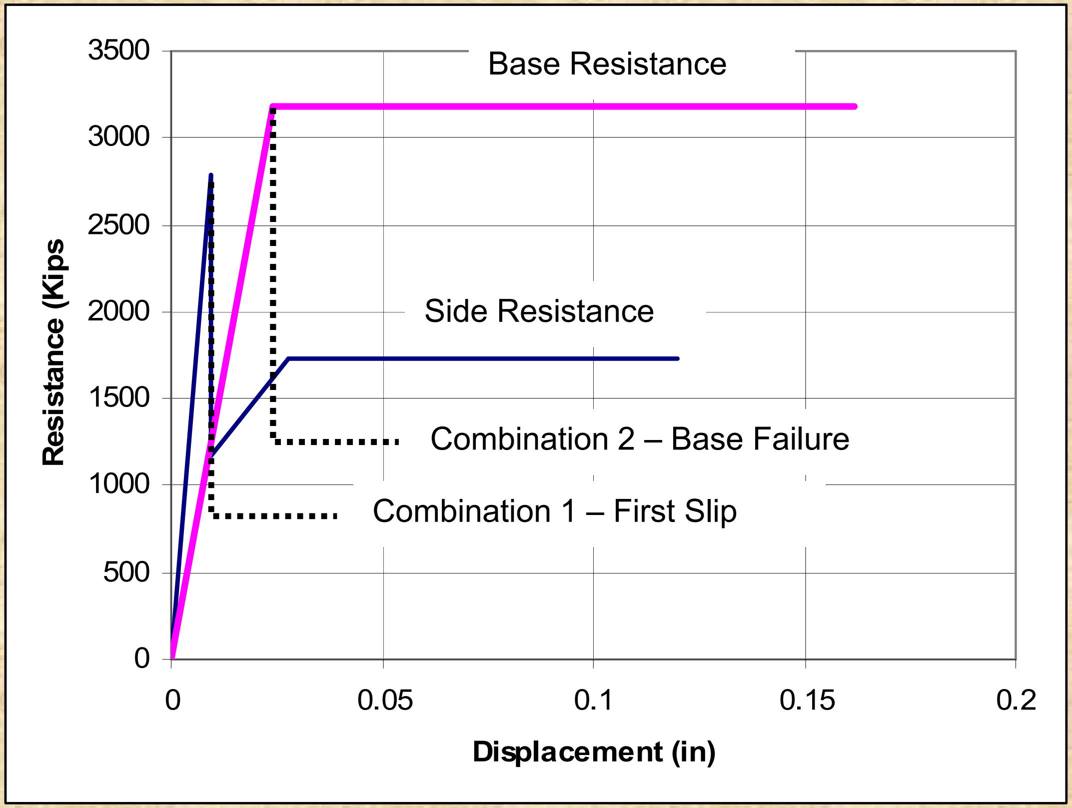


Recommendation for Q-z plots



Drilled Shafts

Strength Limit State: *combined side and end*



Resistance
&
Displacement Relation
(4.5' dia. x 10' long rock socket)

Side Control

Rs = 2784
Rp = 1200
R_T = 3984 kips

End Control

Rs = 1600
Rp = 3180
R_T = 4780 kips

P-y curves

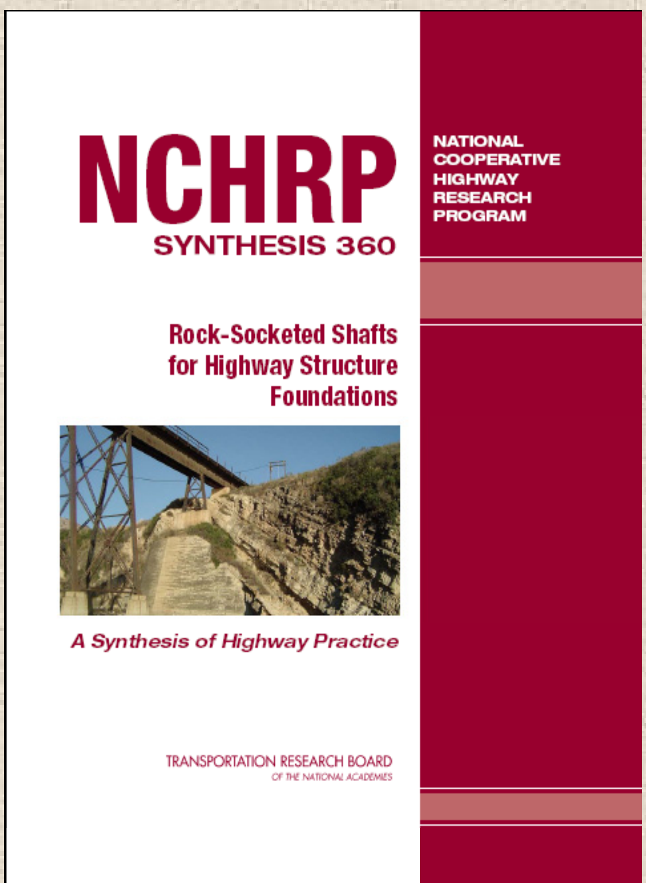
Service Limit State: *lateral*

- P-y in soil

- P-y in rock
  - P-y relationship proposed by Liang
  - Correlation if applicable to WV foundation material

P-y curves

NCHRP Synthesis 360 (2006)

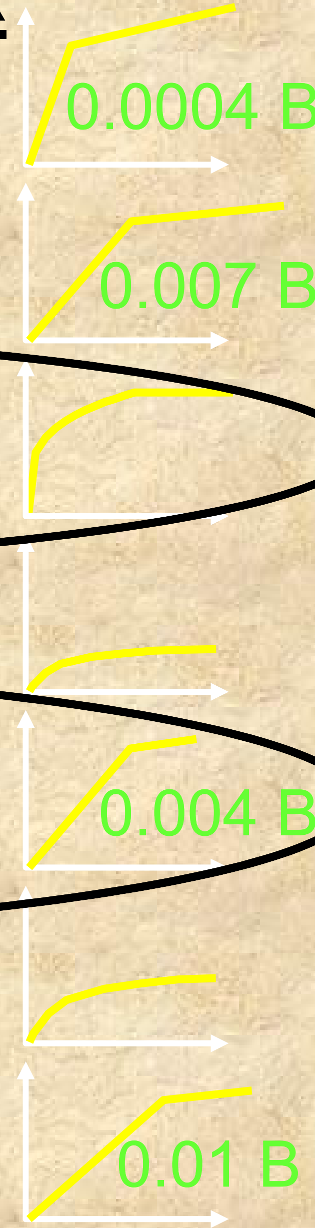


Methods of Analysis

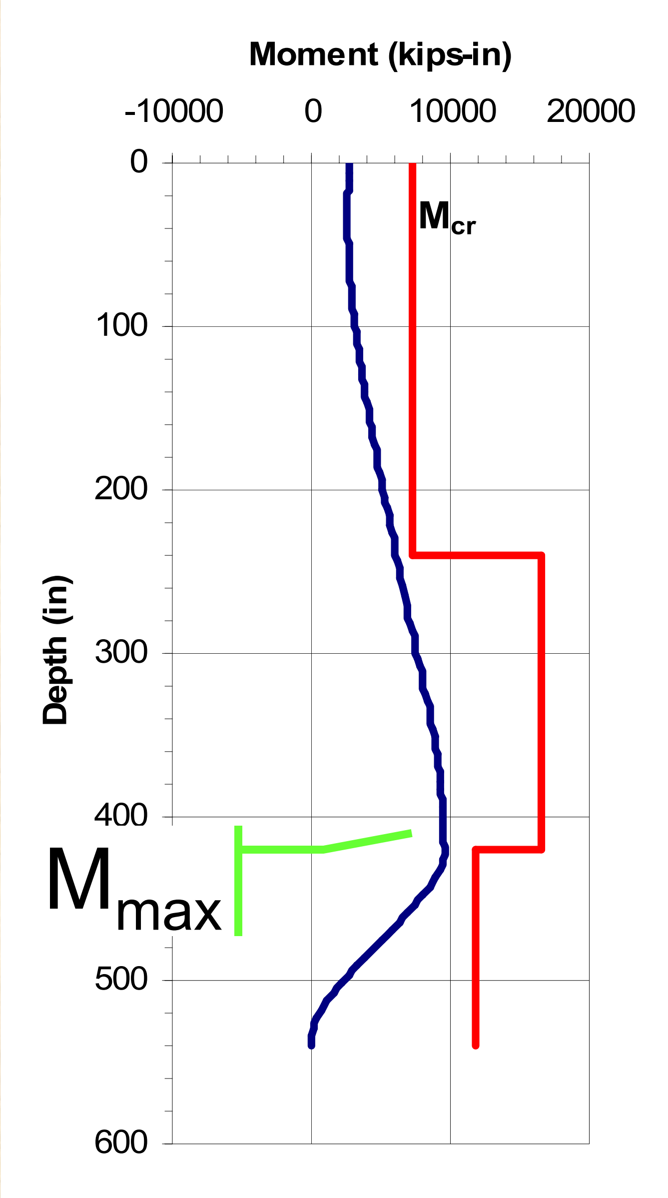
P-y analysis = 29
COM624 = 17
FB-Pier = 8
L-Pile = 23
Reese 1997 criteria = 25
States Responding = 33

P-y response curves for Rock

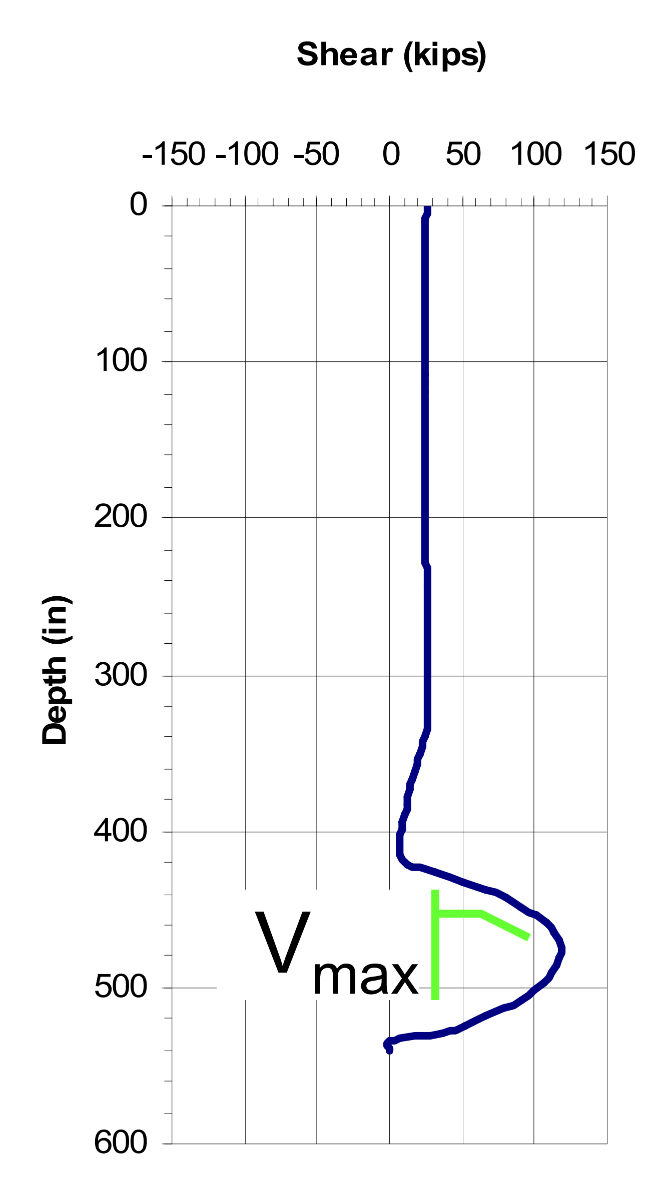
1978	Vuggy Limestone	Nyman Reese
1987	Sandstone & Shale	KDOT
1997	Weak Rock	Reese
2002	Rock	Gabr (NC)
2004	Florida Limestone	McVay
2006	Rock	Liang (OH)
2006	Basalt	WA
2006	All	Vu (WY)



P-y Results

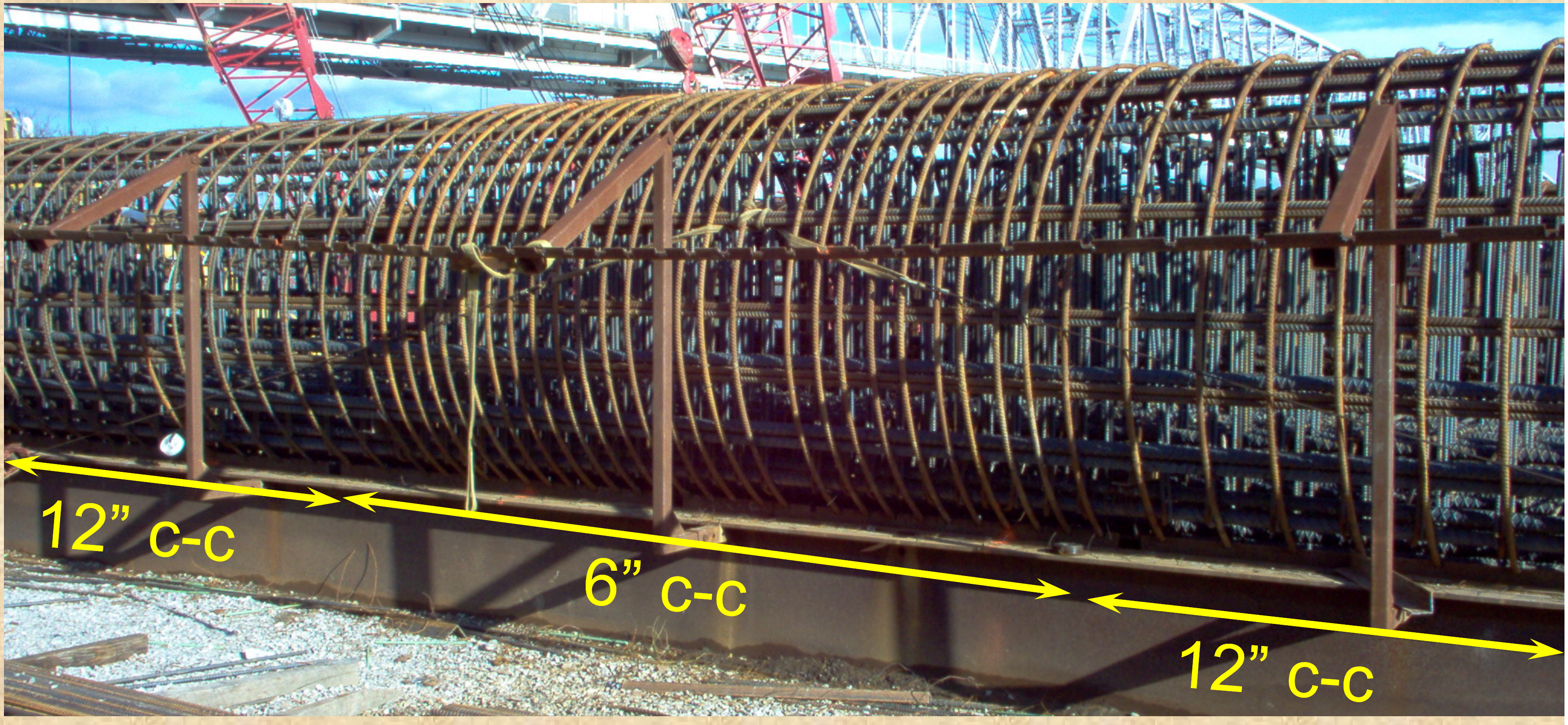


Moment (kips-in)
-10000
0
10000
20000
0
100
200
300
400
500
600
Depth (in)
M_{cr}
M_{max}



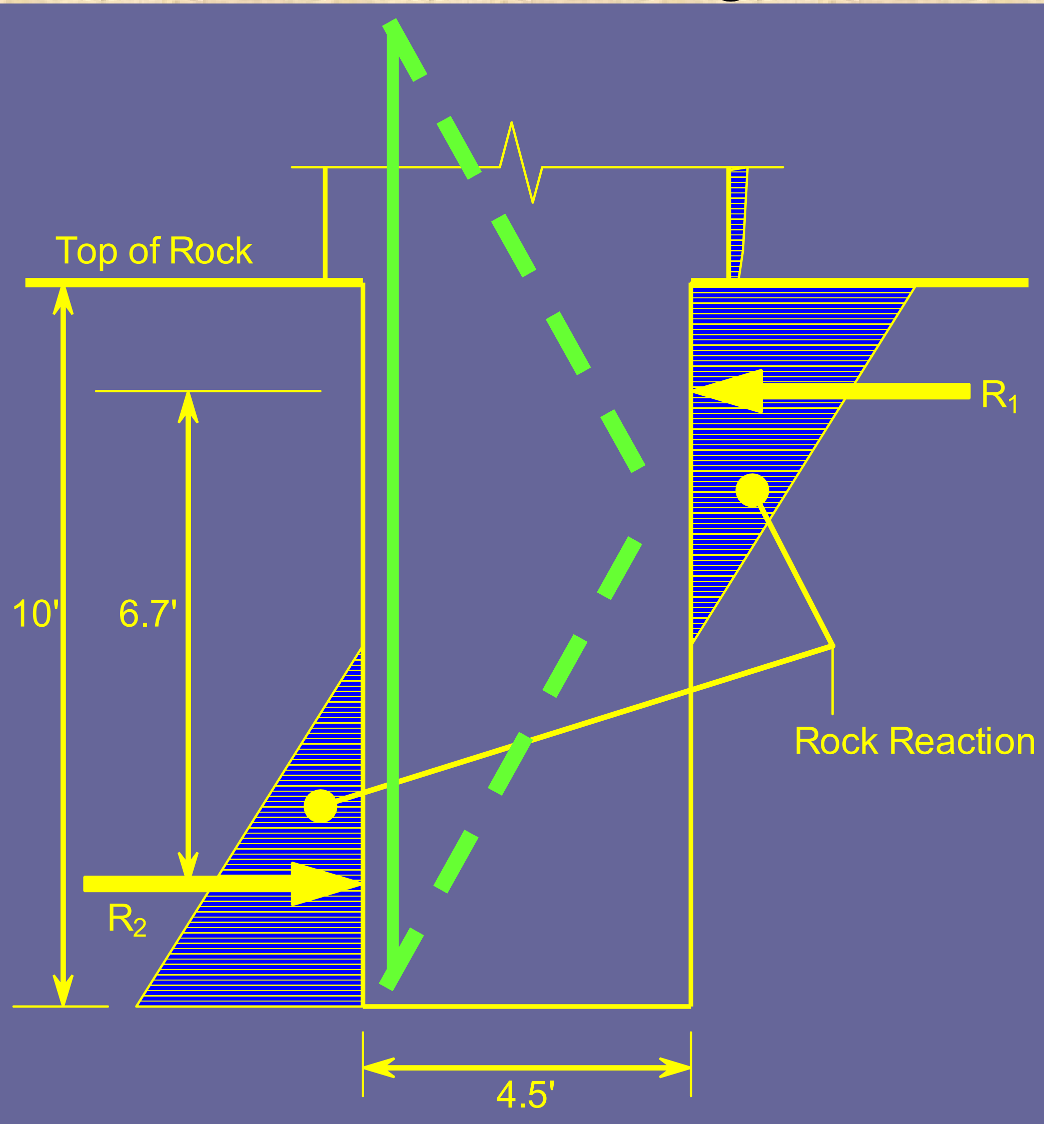
Shear (kips)
-150 -100 -50 0 50 100 150
0
100
200
300
400
500
600
Depth (in)
V_{max}

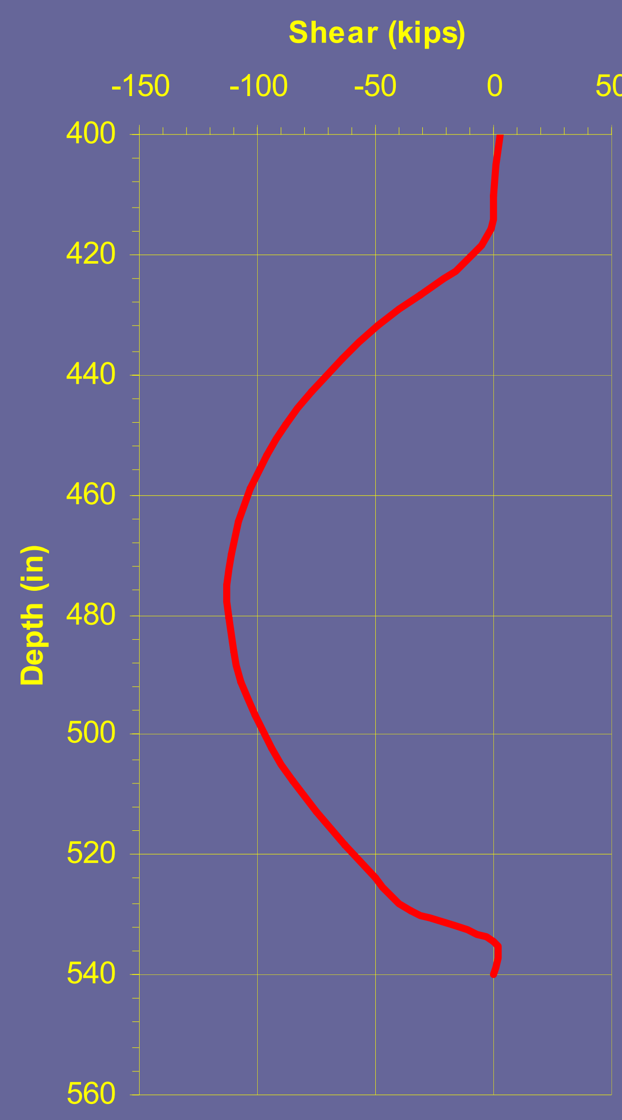
P-y Results



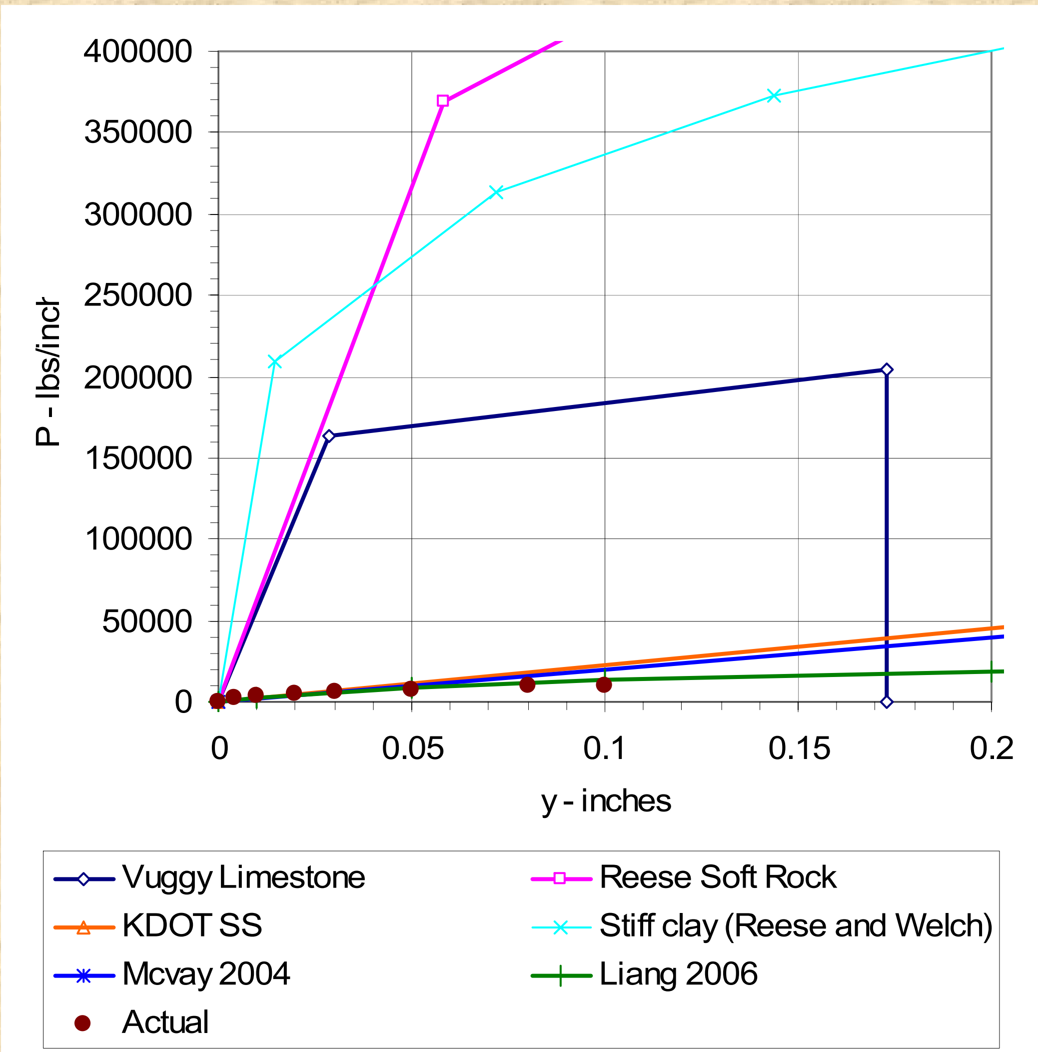
High Shear

Strut and Tie Model for Structural Design of Shaft (AASTHO 5.6.3)





Dayton Load test at 3' depth



Input Rock Properties

Reese & Vuggy LS

q_u = 5668 psi

E_m = 38142 psi

γ_r = 0.038 pci

RQD = 8

Liang 2006

q_u = 5668 psi

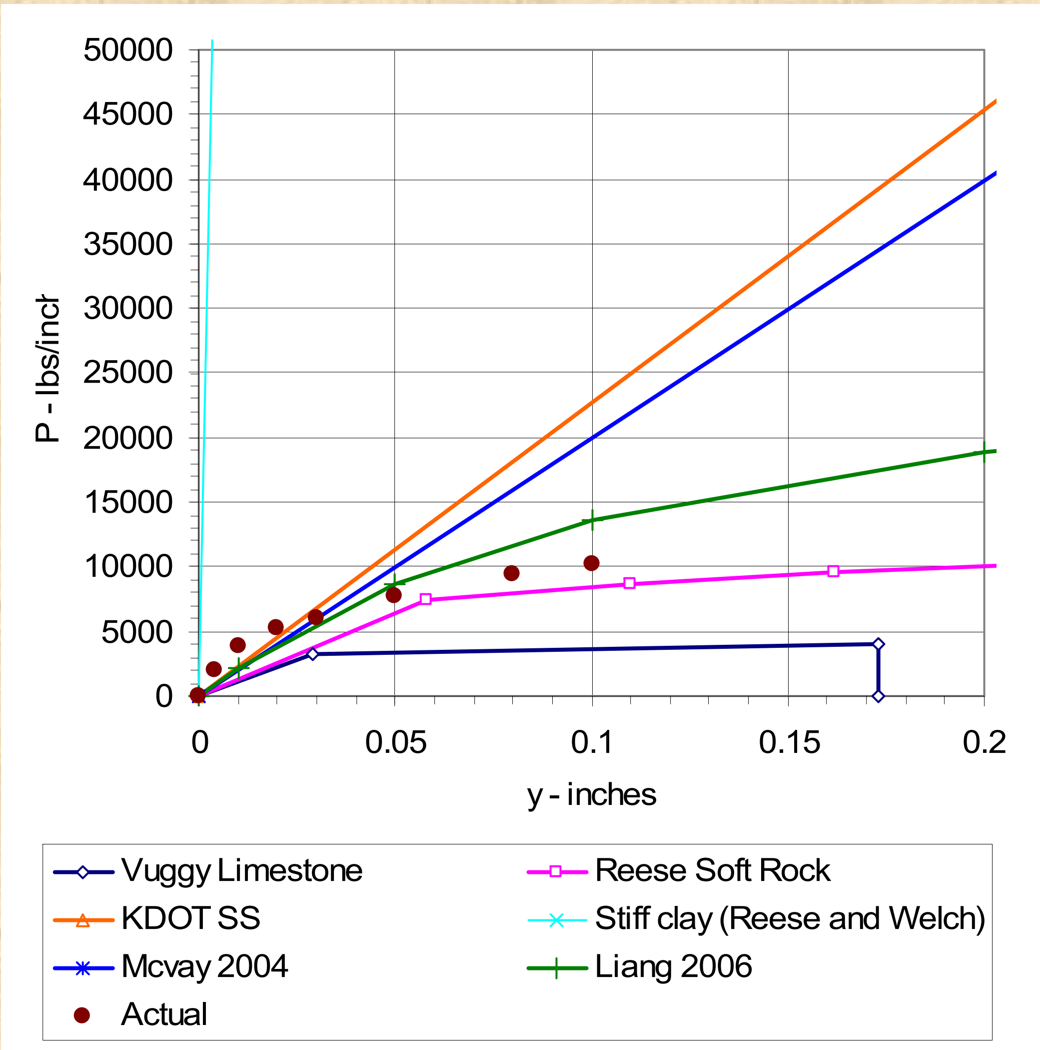
E_i = 590000 psi

γ_r = 0.038 pci

RMR/GSI = 40

m_i = 6

Dayton Load test at 3' depth



Input Rock Properties

Reese & Vuggy LS

q_u = 5668 psi x 0.02

E_m = 38142 psi x 0.02

γ_r = 0.038 pci

RQD = 8

Liang 2006

q_u = 5668 psi

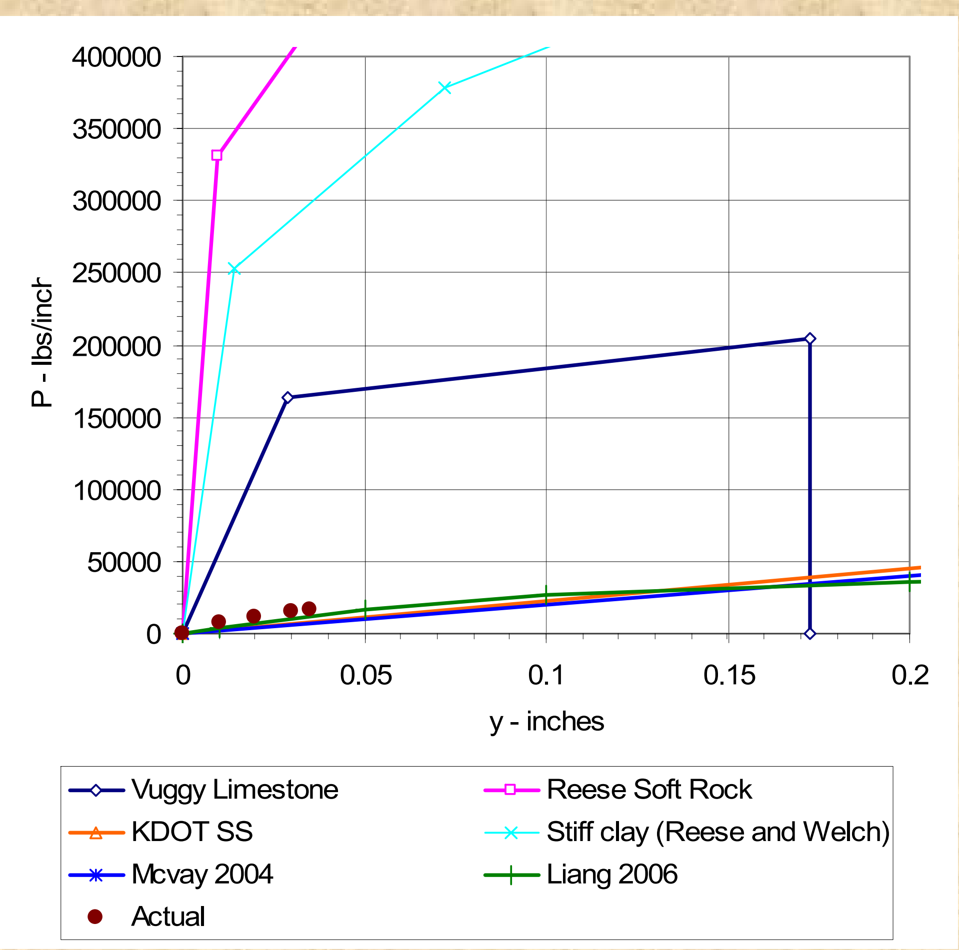
E_i = 590000 psi

γ_r = 0.038 pci

RMR/GSI = 40

m_i = 6

Dayton Load test at 11’ depth



Input Rock Properties

Reese & Vuggy LS

q_u = 5668 psi

E_m = 98102 psi

γ_r = 0.038 pci

RQD = 8

Liang 2006

q_u = 5668 psi

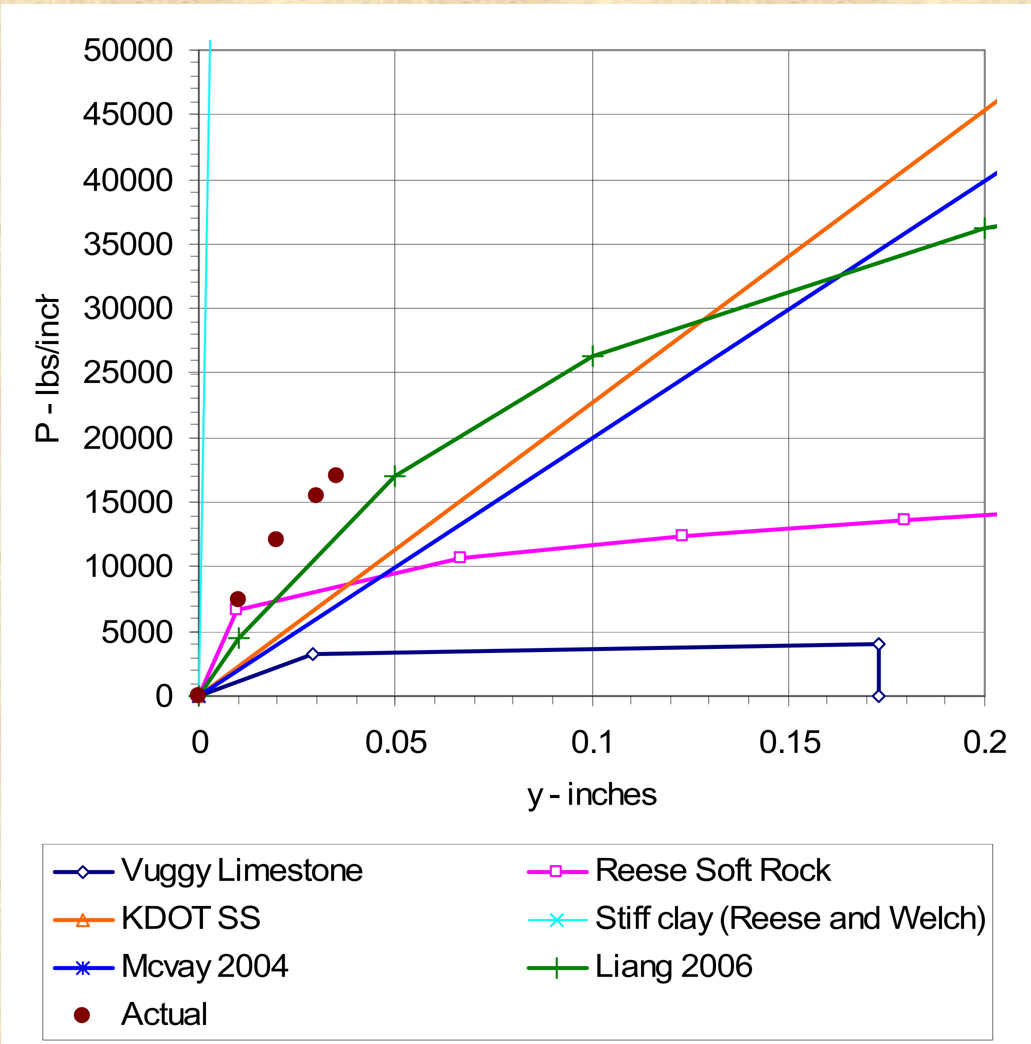
E_i = 590000 psi

γ_r = 0.038 pci

RMR/GSI = 61

m_i = 6

Dayton Load test at 11’ depth



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RQD = 8

Liang 2006

q_u = 5668 psi

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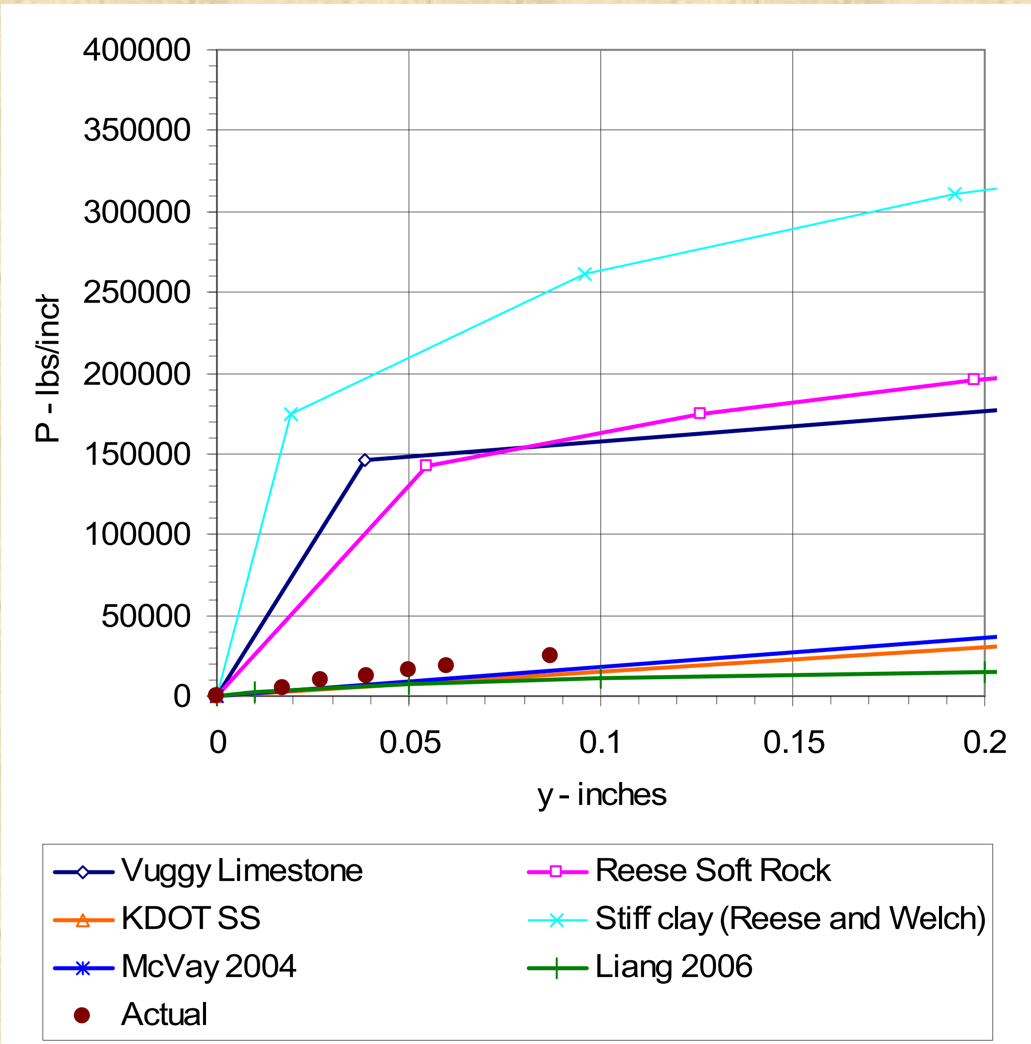
RMR/GSI = 61

m_i = 6

Pomeroy Mason #2 at 0.5' depth



Input Rock Properties

Reese & Vuggy LS

q_u = 3797 psi

E_m = 23885 psi

γ_r = 0.059 pci

RQD = 44

Liang 2006

q_u = 3797 psi

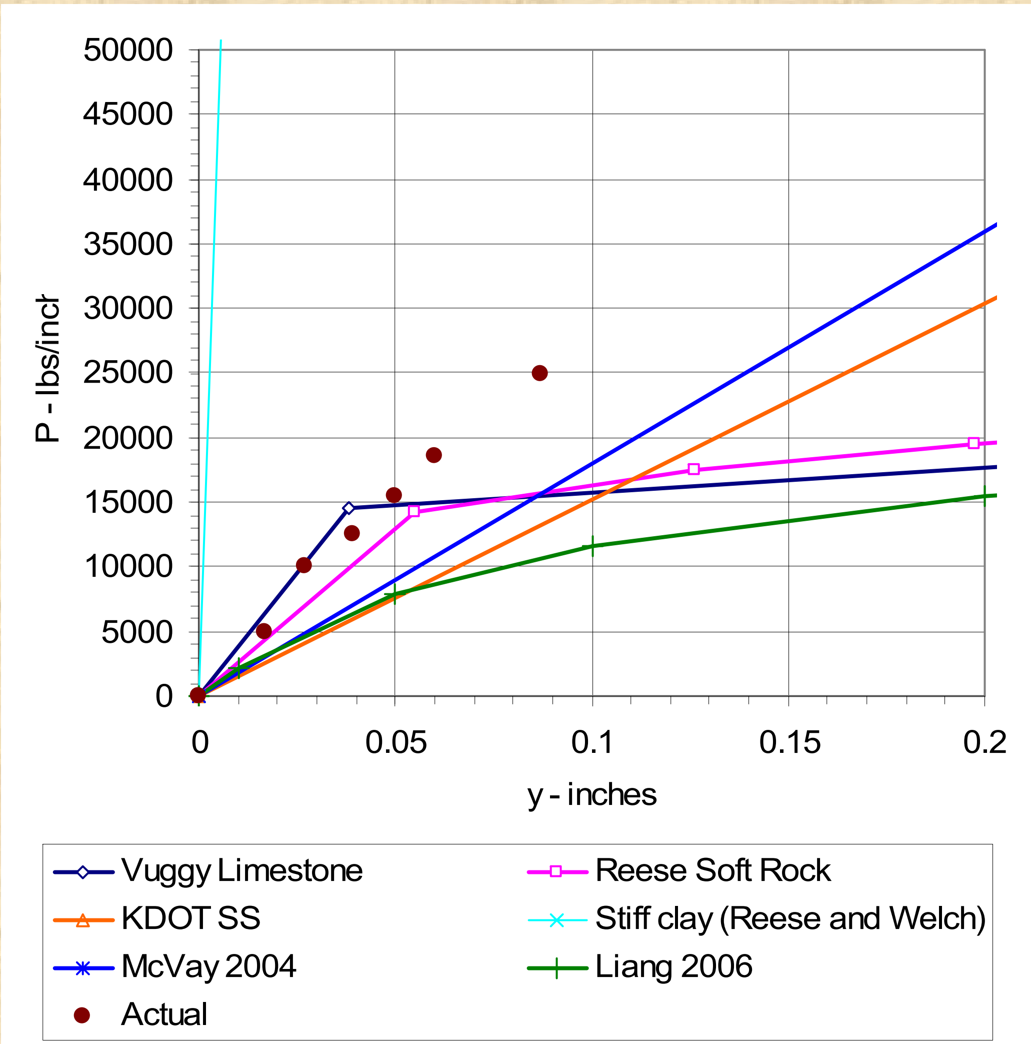
E_i = 914888 psi

γ_r = 0.059 pci

RMR/GSI = 42

m_i = 6

Pomeroy Mason #2 at 0.5' depth



Input Rock Properties

Reese & Vuggy LS

q_u = 3797 psi x 0.1

E_m = 23885 psi x 0.1

γ_r = 0.059 pci

RQD = 44

Liang 2006

q_u = 3797 psi

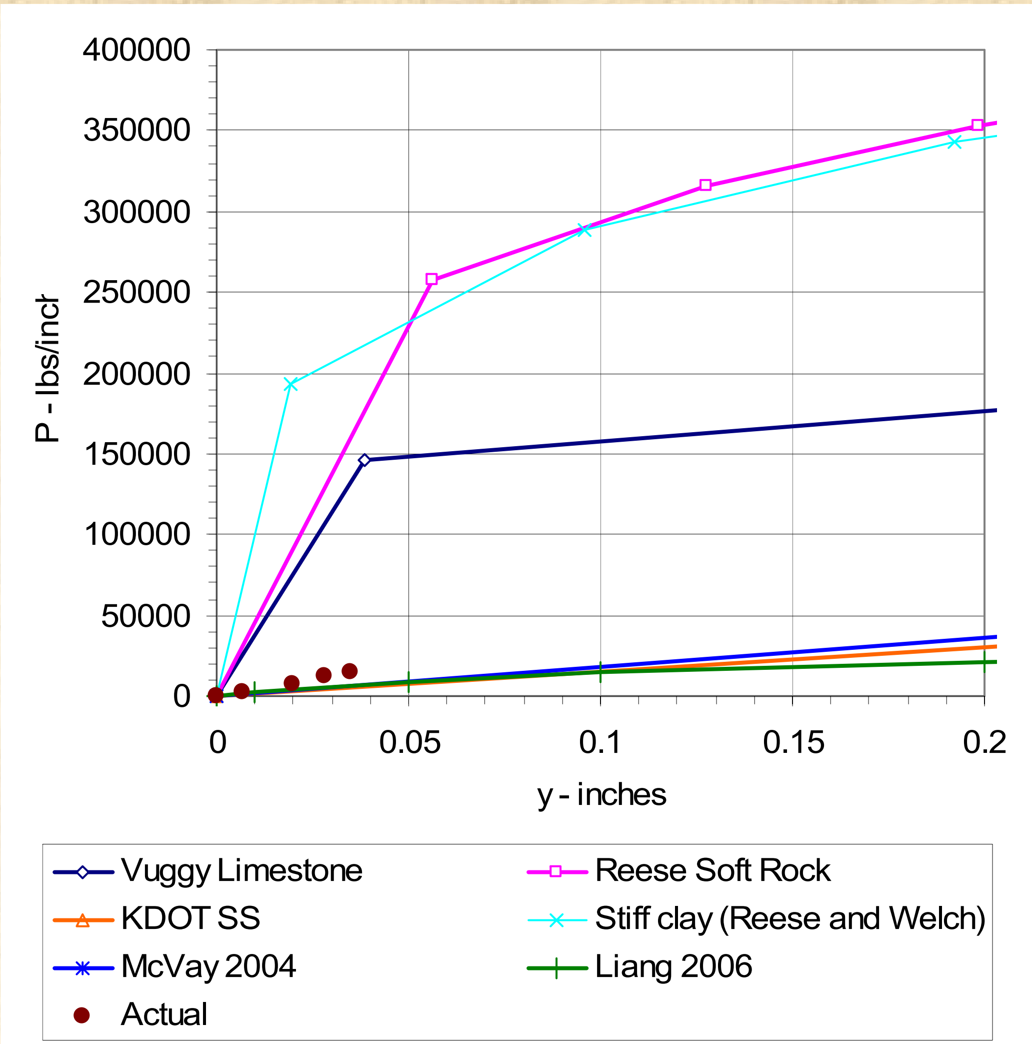
E_i = 914888 psi

γ_r = 0.059 pci

RMR/GSI = 42

m_i = 6

Pomeroy Mason #2 at 5.5' depth



Input Rock Properties

Reese & Vuggy LS

q_u = 3797 psi

E_m = 23885 psi

γ_r = 0.059 pci

RQD = 44

Liang 2006

q_u = 3797 psi

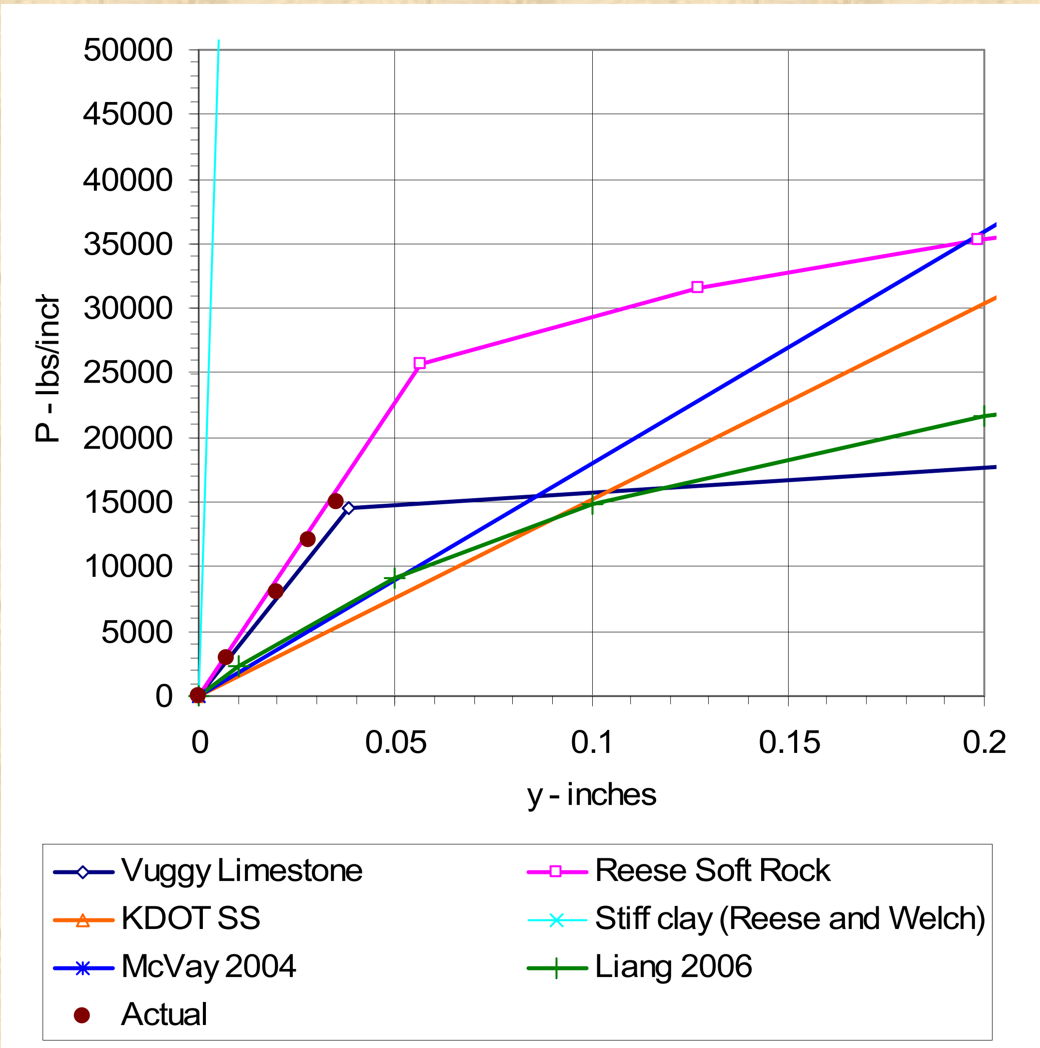
E_i = 914888 psi

γ_r = 0.059 pci

RMR/GSI = 42

m_i = 6

Pomeroy Mason #2 at 5.5’ depth



Input Rock Properties

Reese & Vuggy LS

q_u = 3797 psi x 0.1

E_m = 23885 psi x 0.1

γ_r = 0.059 pci

RQD = 44

Liang 2006

q_u = 3797 psi

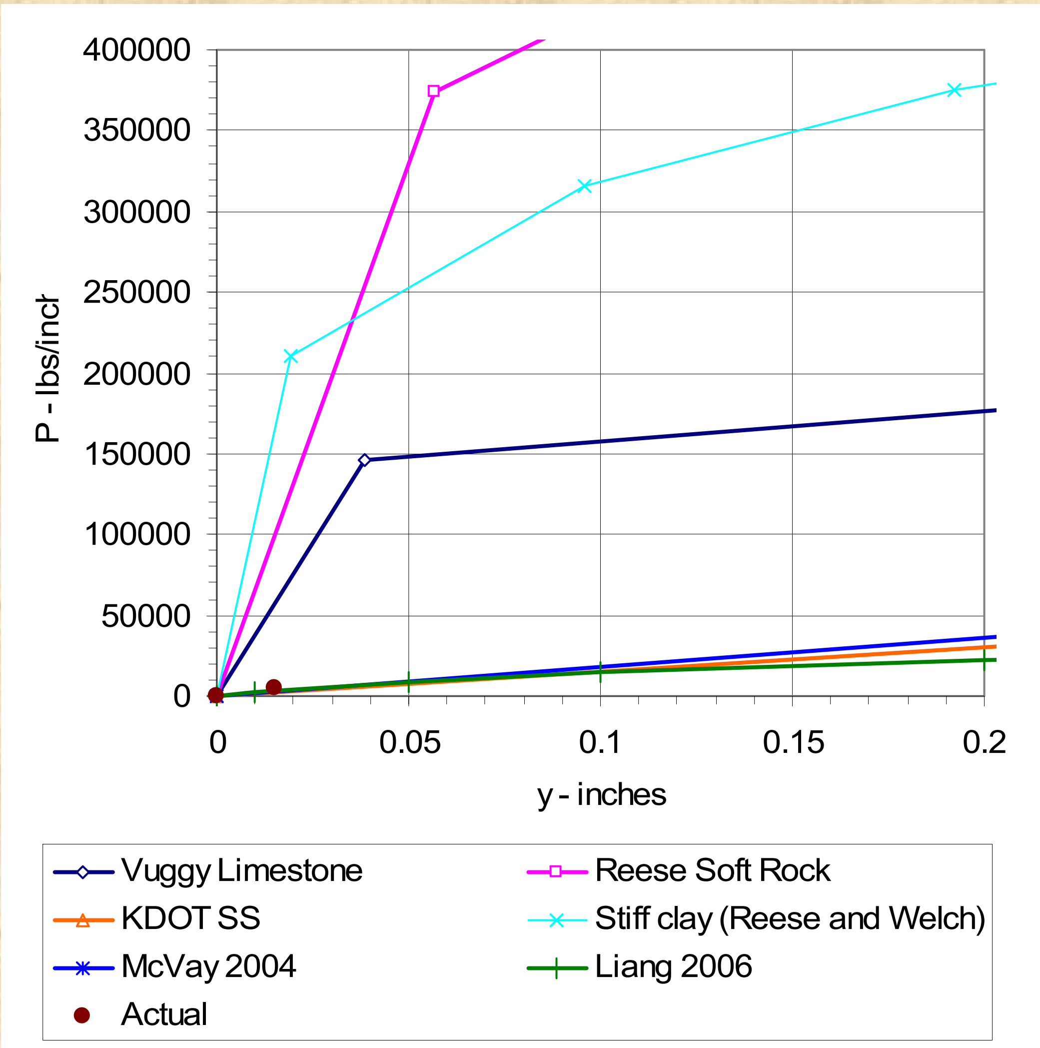
E_i = 914888 psi

γ_r = 0.059 pci

RMR/GSI = 42

m_i = 6

Pomeroy Mason #2 at 10.5' depth



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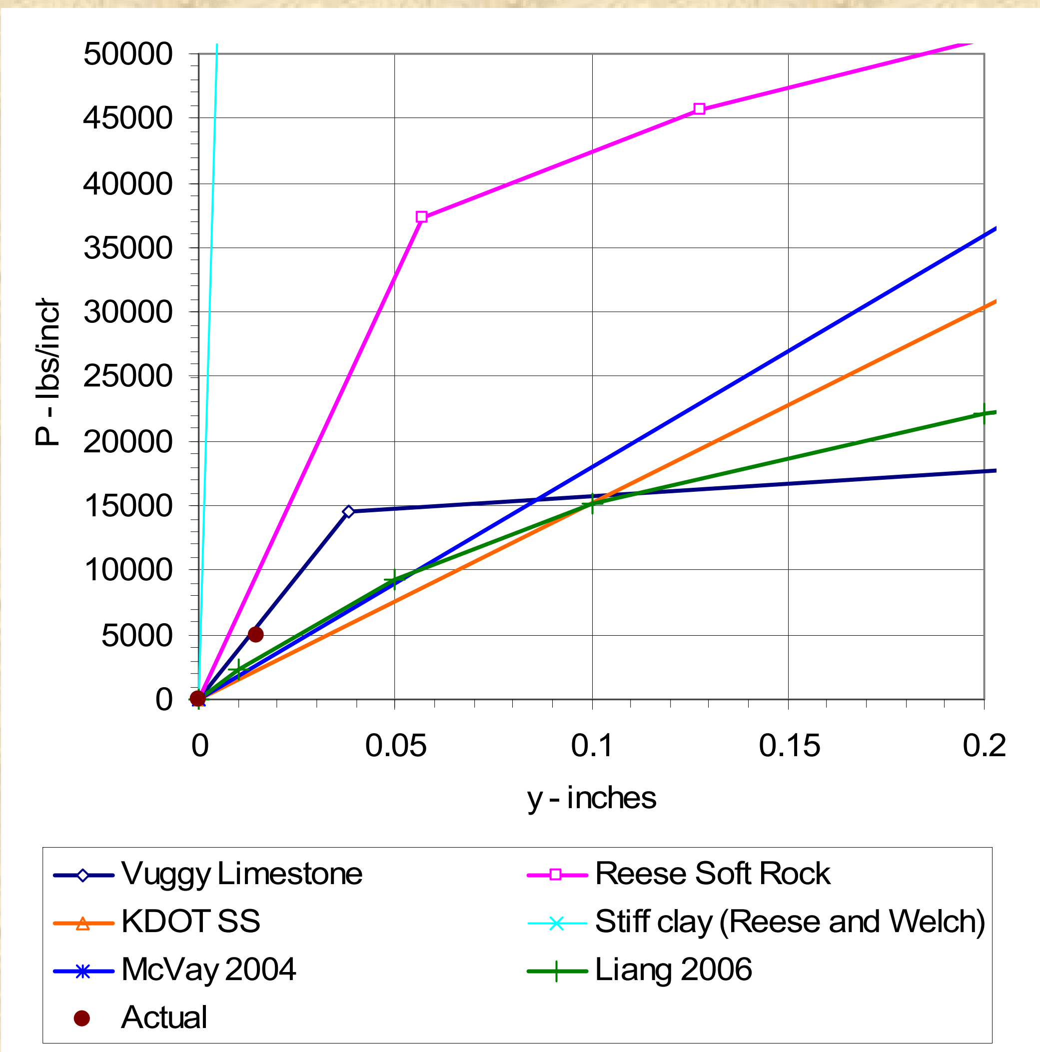
γ_r = 0.059 pci

RMR/GSI = 42

m_i = 6

Pomeroy Mason #2 at 10.5' depth



Input Rock Properties

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RQD = 44

Liang 2006

q_u = 3797 psi

E_i = 914888 psi

γ_r = 0.059 pci

RMR/GSI = 42

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Cont….

- Pile Policy
  - Wave Equation analysis
    - Preliminary analysis, assumed driving equipment
    - Verification analysis, contractor’s actual equipment