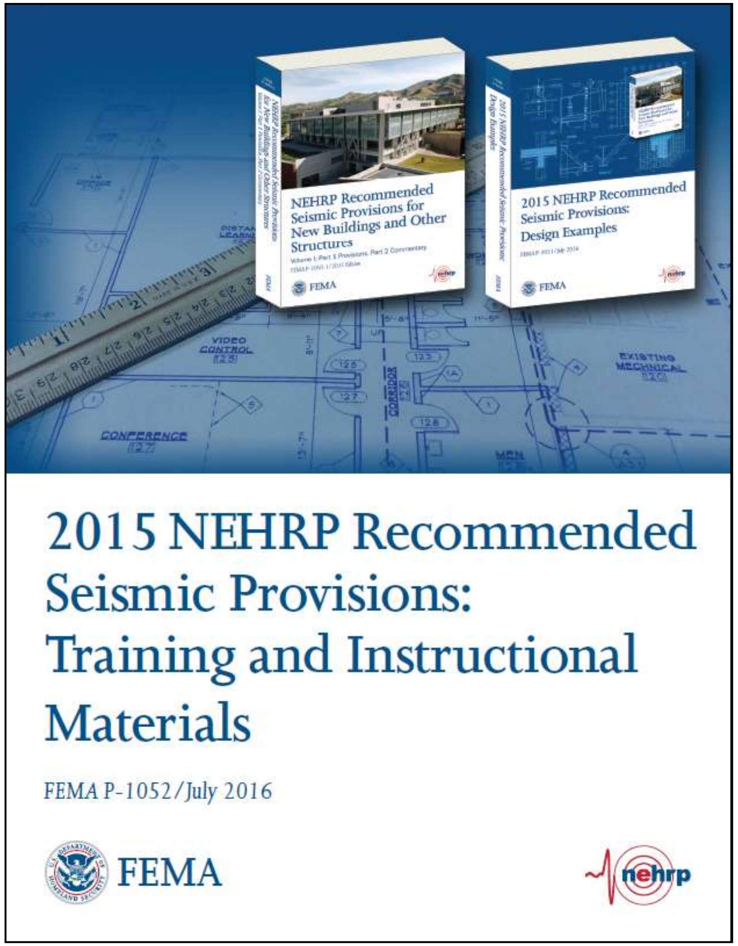


7

Foundation and Liquefaction Design

Ian McFarlane, S.E. and Stephen K. Harris, S.E.

Originally developed by Michael Valley, S.E.

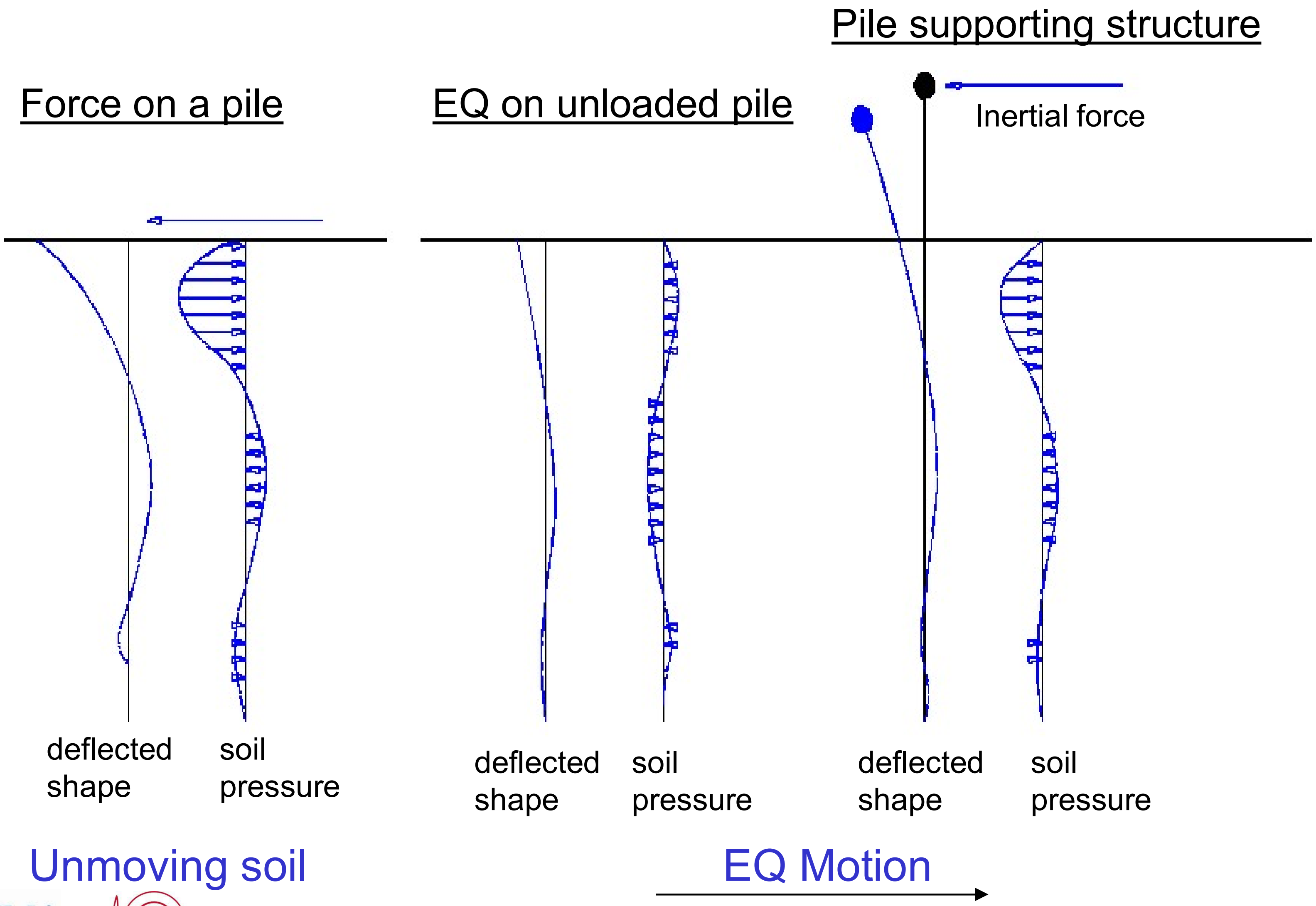
FOUNDATION AND LIQUEFACTION DESIGN

- TABLE OF CONTENTS:

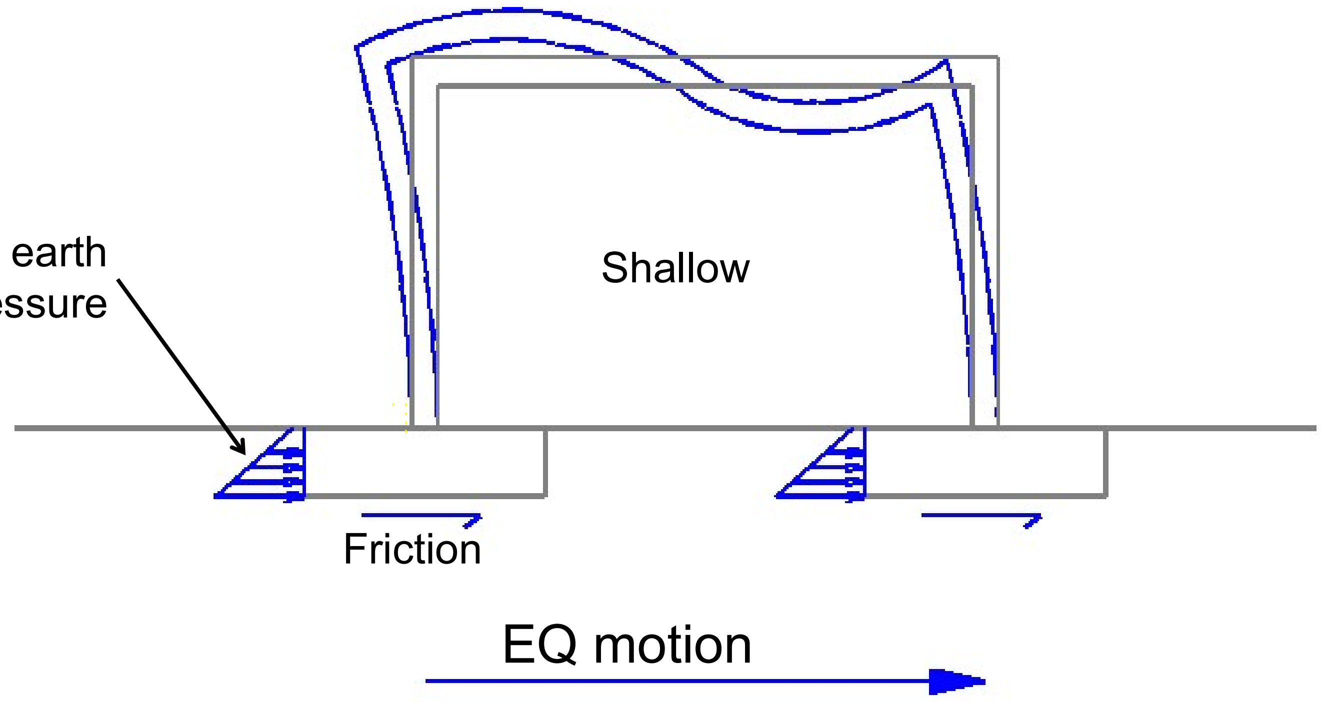
FOUNDATION DESIGN

- Proportioning Elements for:
- Transfer of Seismic Forces
- Strength and Stiffness
- Shallow and Deep Foundations
- Elastic and Plastic Analysis

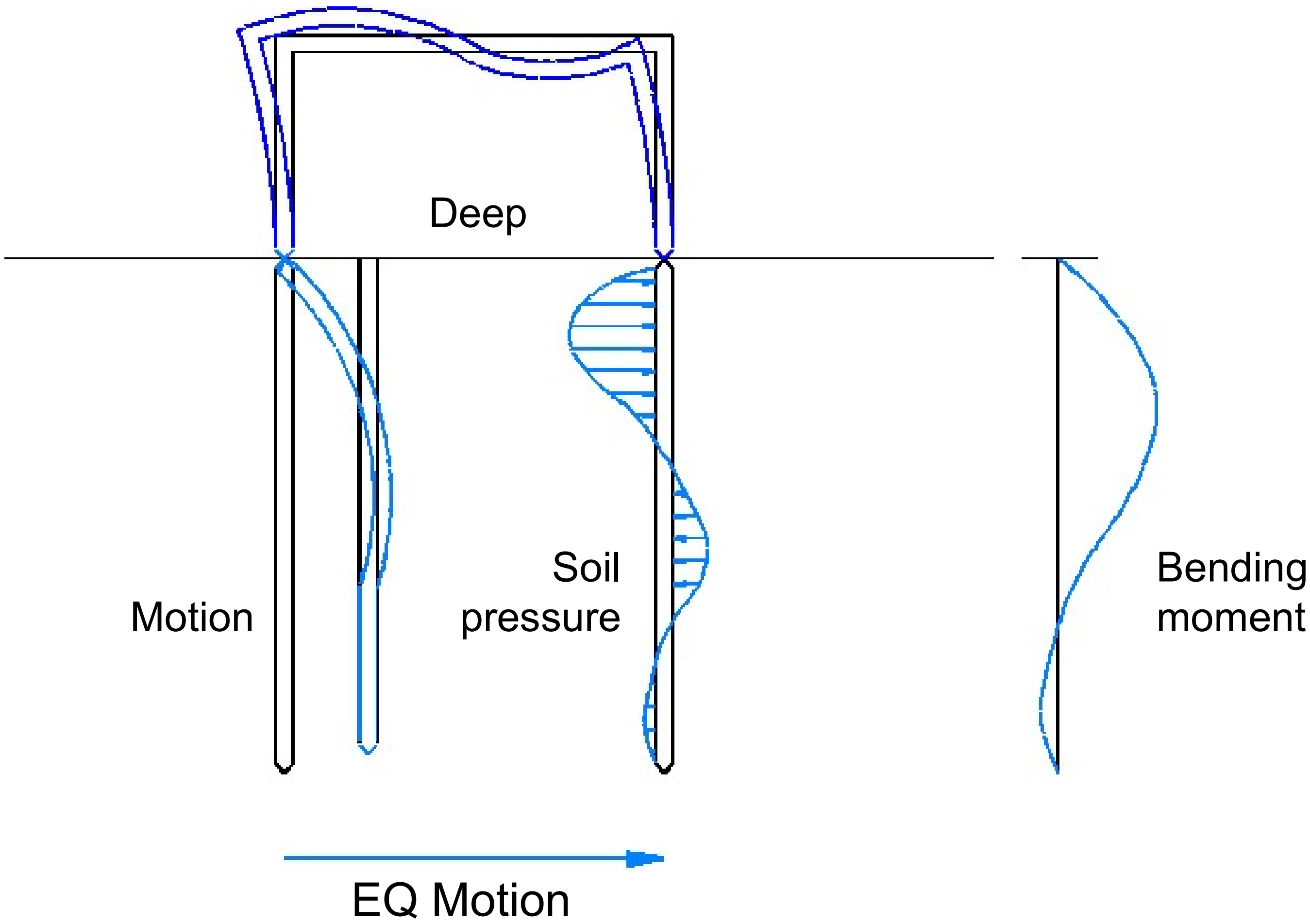
Load Path and Transfer of Seismic Forces



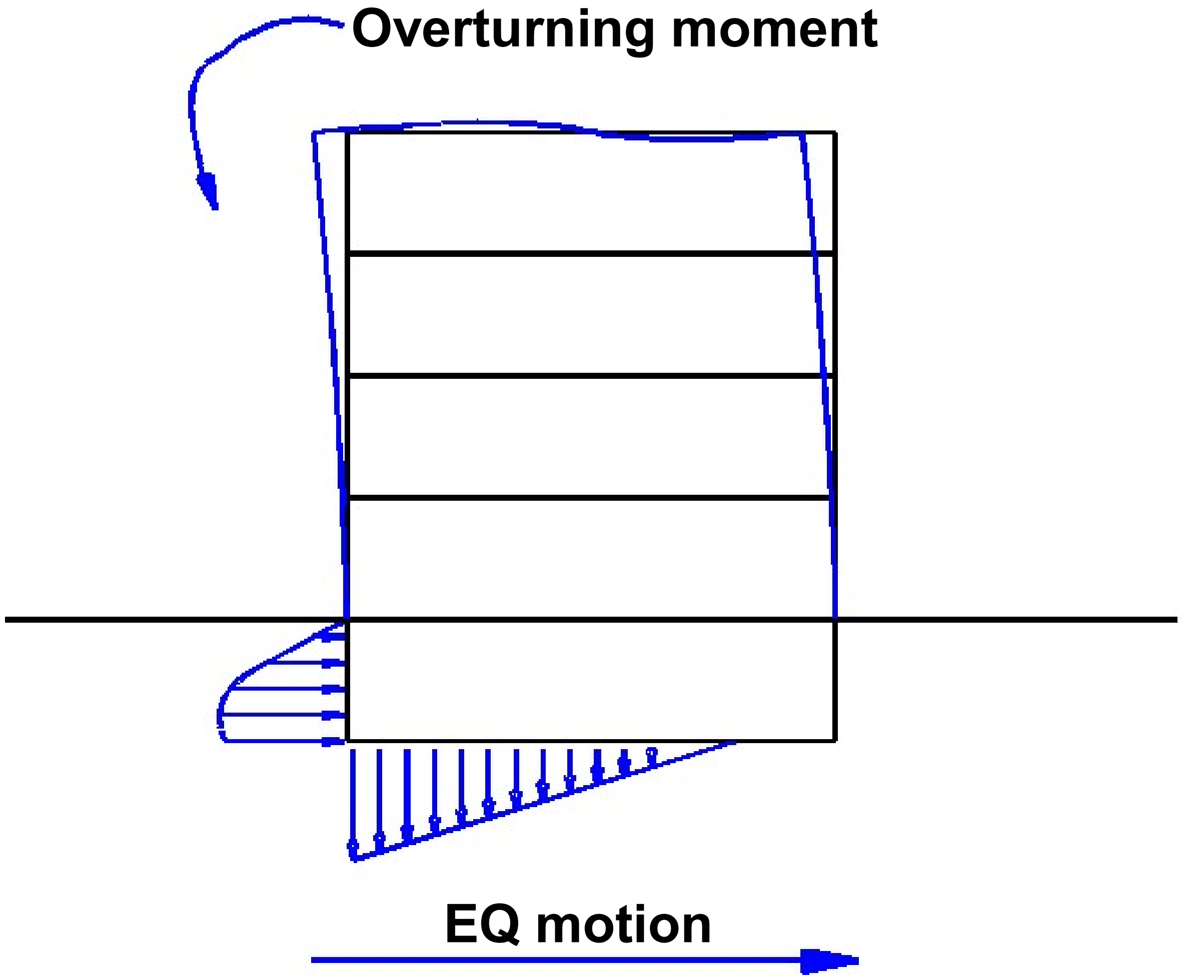
Load Path and Transfer of Seismic Forces
foundation force transfer



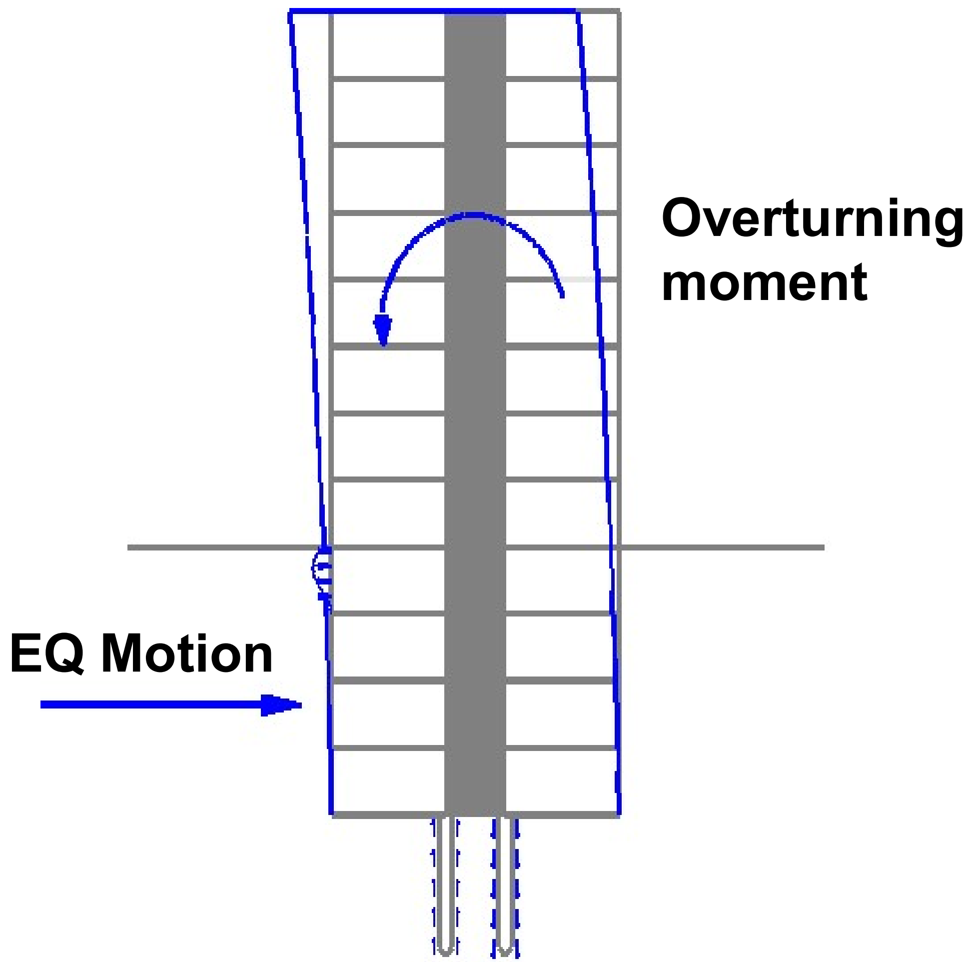
Load Path and Transfer of Seismic Forces
soil to foundation force transfer



Load Path and Transfer of Seismic Forces
vertical pressures - shallow



Load Path and Transfer of Seismic Forces
vertical pressures - deep



Allowable Stress Design vs Strength Design

- Allowable Stress Design
  - Traditional/historic approach
  - Allowable geotechnical values have inherent factor of safety
  - Requires separate ASD load combination
- Strength Design
  - Permitted in 2015 Standard
  - Nominal strength with phi factors
  - Allows direct comparison of foundation capacity with superstructure capacity

Allowable Stress Design vs Strength Design

- Nominal Strength can be:
  - Nominal strength associated with anticipated failure mechanism
  - Nominal strength associated with limitation on maximum deformation at failure
- Settlement based on sustained loads rather than the loading associated with nominal strength should be considered

Allowable Stress Design vs Strength Design

- Nominal Strength Example
  - Strength reduction factor $\Phi=0.45$ for compression (bearing pressure)
- Allowable Stress Example
  - Factor of safety of ~3.0

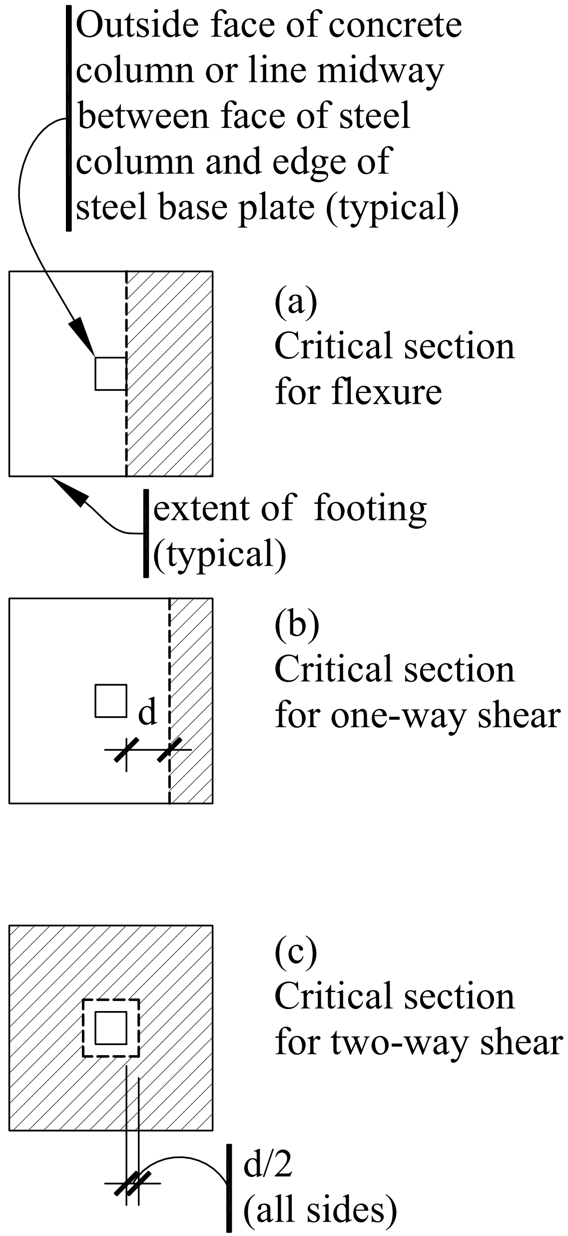
Foundation Design Concepts

- Most foundation failures are associated with excessive foundation movement, not loss of load-bearing capacity
- Maintaining a reasonably consistent service load-bearing pressure is encouraged to minimize differential settlement

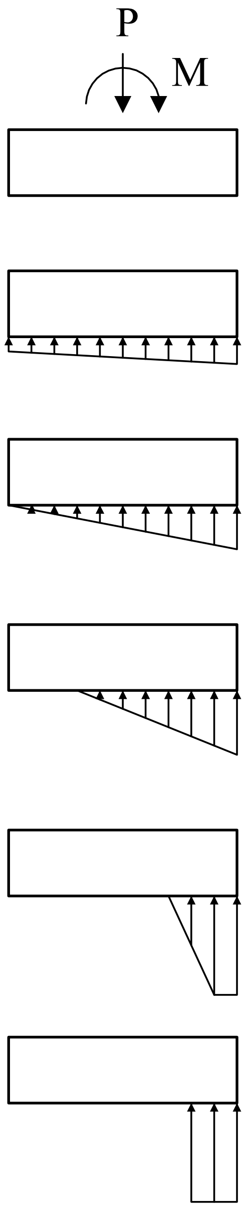
Design of Concrete Footings

- ACI 318 References
  - Chapters 7,8 – One way, Two-way slabs referenced for detailing requirements
  - Chapter 13 – General Foundations
  - Chapter 18 – Seismic Provisions
  - Chapter 22 – Sectional Strength

Reinforced Concrete Footings: Basic Design Criteria (concentrically loaded)



Footing Subject to Compression and Moment: Uplift Nonlinear



(a)
Loading

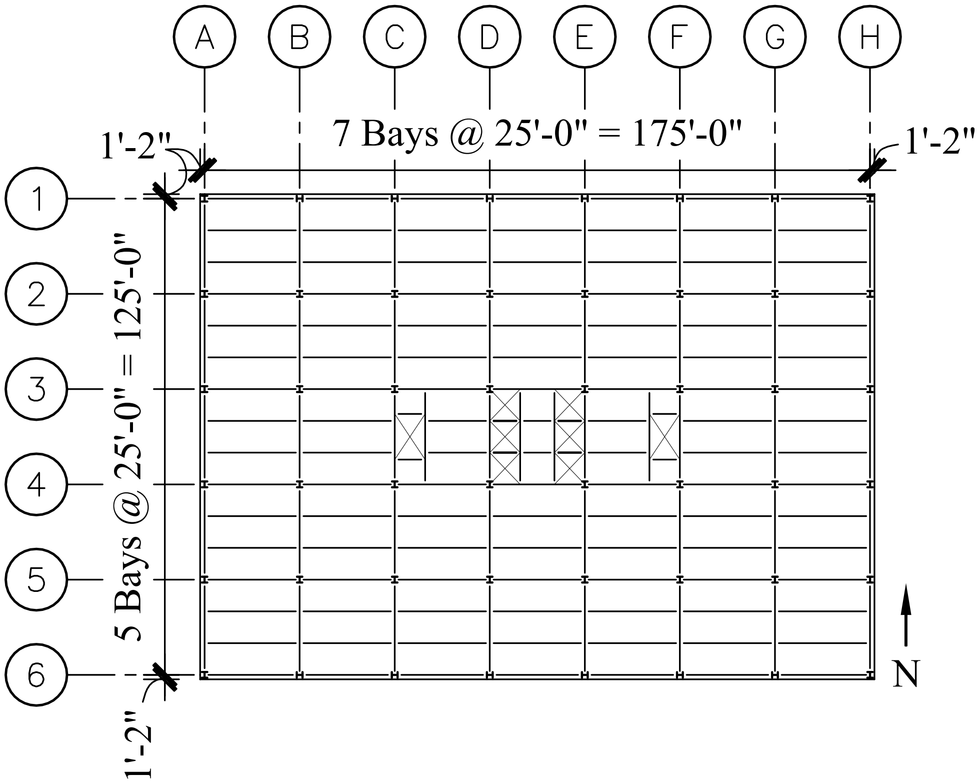
(b)
Elastic, no uplift

(c)
Elastic, at uplift

(d)
Elastic, after uplift

(e)
Some plastification

(f)
Plastic limit



Example
7-story building: shallow foundations designed for perimeter frame and core bracing

Shallow Footing Examples

Soil parameters:

- Medium dense sand
- (SPT) N = 20
- Density = 125 pcf
- Friction angle = 33^o

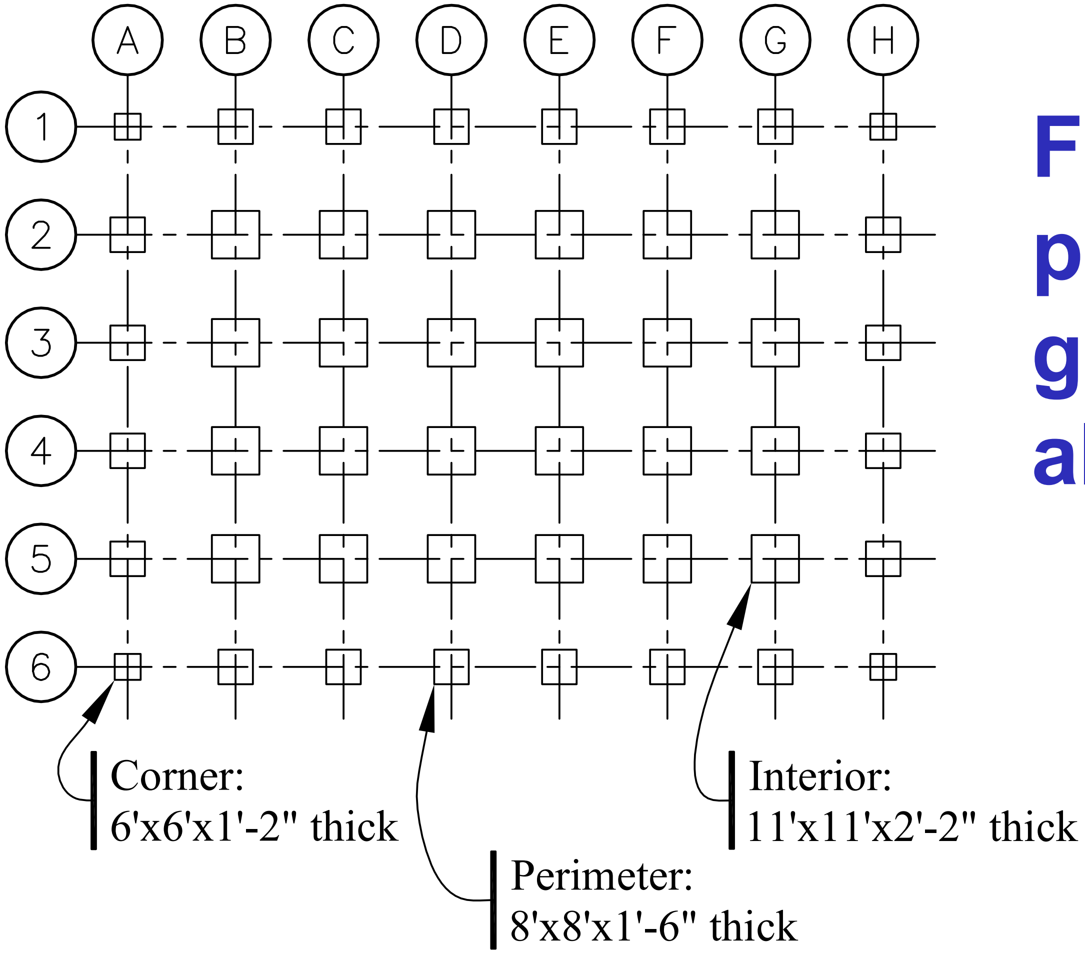
Bearing Capacity

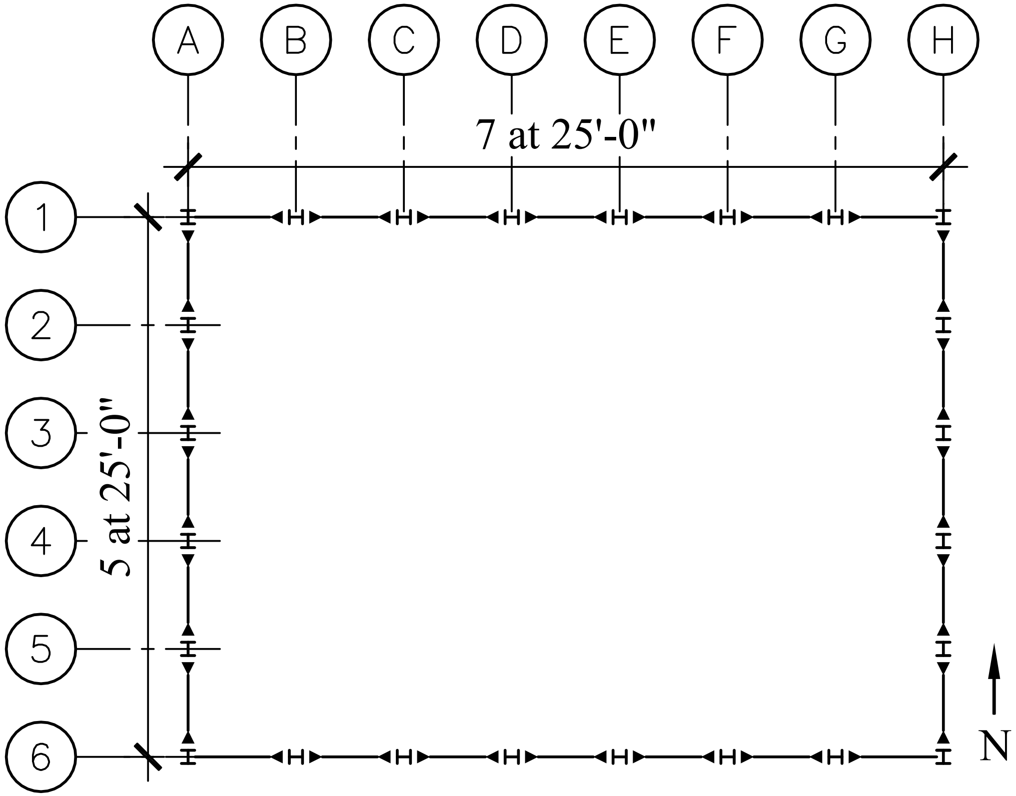
- 3,000B psf (Square)
- 4,000B' psf (Rectangular)
- B = footing width (ft)
- B' = average width of compressed area (ft)
- ϕ=0.45

Allowable Bearing (Settlement)

- 2,000 psf B<20 ft
- 1,000 psf B<40 ft

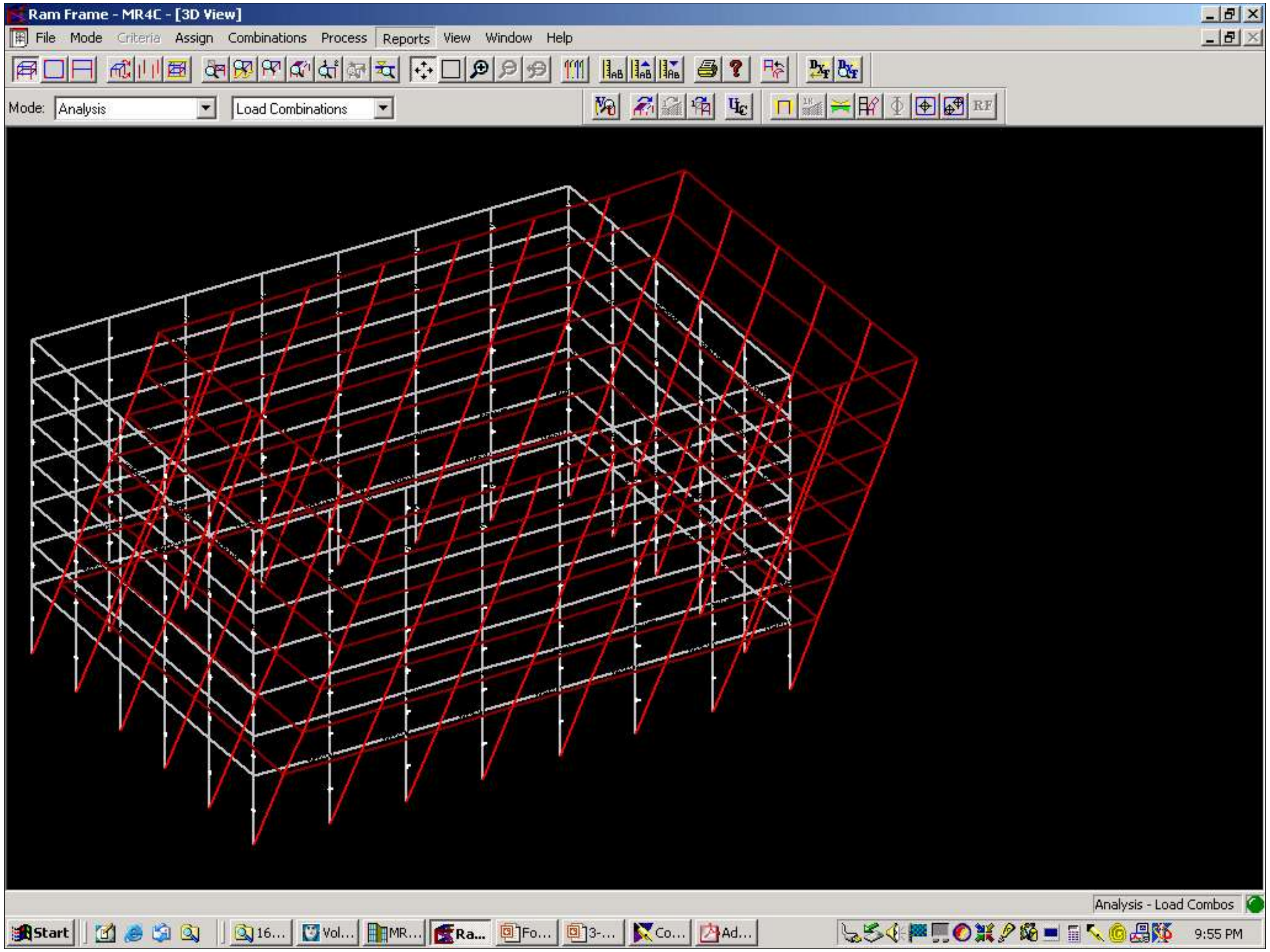
Footings proportioned for gravity loads alone





Design of footings for perimeter moment frame

7 Story Frame, Deformed



Combining Loads

- Maximum downward load:

  $1.2D + 0.5L + E$

- Minimum downward load:

  $0.9D + E$

- Definition of seismic load effect E:

  E = r1QE1 + 0.3 r2QE2 +/- 0.2 SDSD

  rx = 1.0   ry = 1.0 and   SDS = 1.0

Reactions

Grid		Dead	Live	E_x	E_y
A-5	P M_{xx} M_{yy}	203.8 k	43.8 k	-3.8 k 53.6 k-ft -243.1 k-ft	21.3 k -1011.5 k-ft 8.1 k-ft
A-6	P M_{xx} M_{yy}	103.5 k	22.3 k	-51.8 k 47.7 k-ft -246.9 k-ft	-281.0 k -891.0 k-ft 13.4 k-ft

Reduction of Overturning Moment

- NEHRP Provisions allow base overturning moment to be reduced by 25% at the soil-foundation interface
- For a moment frame, the column vertical loads are the resultants of base overturning moment, whereas column moments are resultants of story shear
- Thus, use 75% of seismic vertical reactions

Additive Load w/ Largest eccentricity

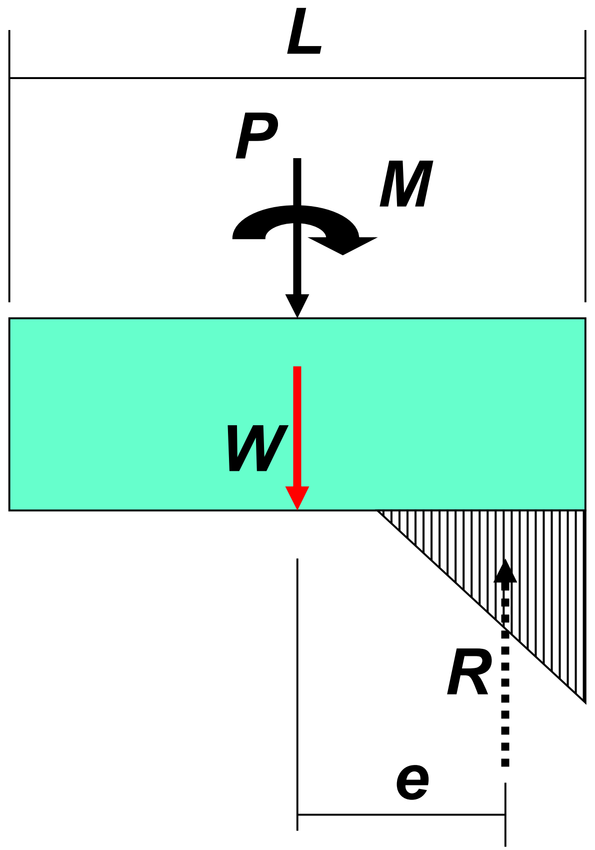
- Combining loads on footings A-5 and A-6, applying the 0.75 multiplier for overturning effects to the axial loads, and neglecting the weight of the foundation and overlying soil,
- P = 688 kips
- Mxx = -6,717 ft-kips
- Myy = -126 ft-kips (which is negligible)

Counteracting Load w/ Largest e

- Again combining loads on footings A-5 and A-6, including the overturning factor, and neglecting the weight of the footing and overlying soil,
- P = 332 kips
- Mxx = -5,712 ft-kips
- Myy = -126 ft-kips (negligible)

Elastic Response

- Objective is to set L and W to satisfy equilibrium and avoid overloading soil
- Successive trials usually necessary



Additive Combination

Given $\boldsymbol{P}$ = 688 k, $\boldsymbol{M}$ =6717 k-ft

e = **M** / **P** = 9.76 ft

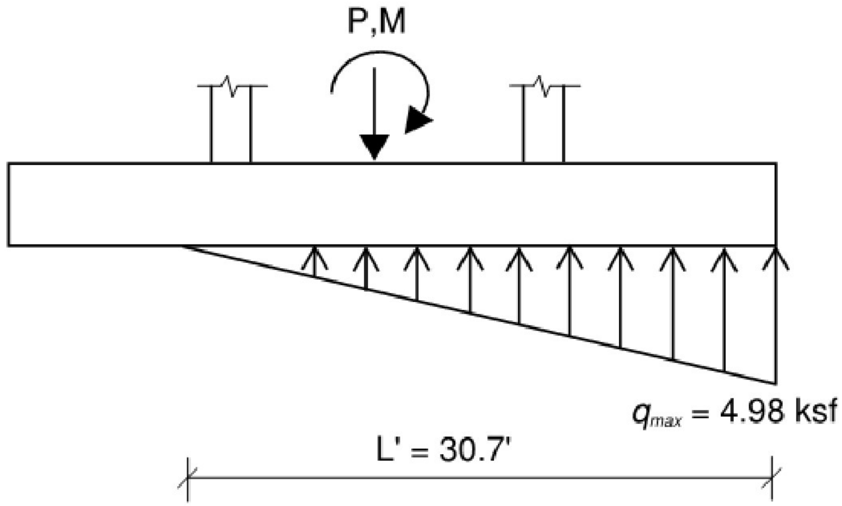
Try $\boldsymbol{L}$ = 40 ft, $\boldsymbol{B}$ = 9 ft

L/6 < **e** < **L/2** therefore elastic with some uplift

L' = 3 (**L/2-e**) = 30.7'

q_{max} = 2 **P** / [3 **B**(**L**/2-**e**)]

q_{max} = 4.98 ksf



Additive Combination

Bearing Capacity:

Q_{ns} = 4,000B’ = 4,000 min (B, L’/2)

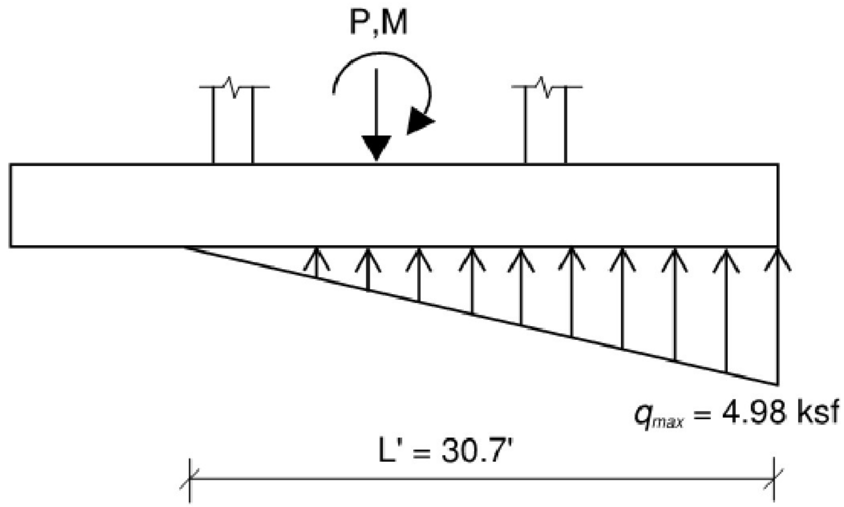
Q_{ns} = 4,000 min (9, 30.7/2) = 36,000 psf

Design bearing capacity:

ϕQ_{ns} = 0.45 (36,000 psf)

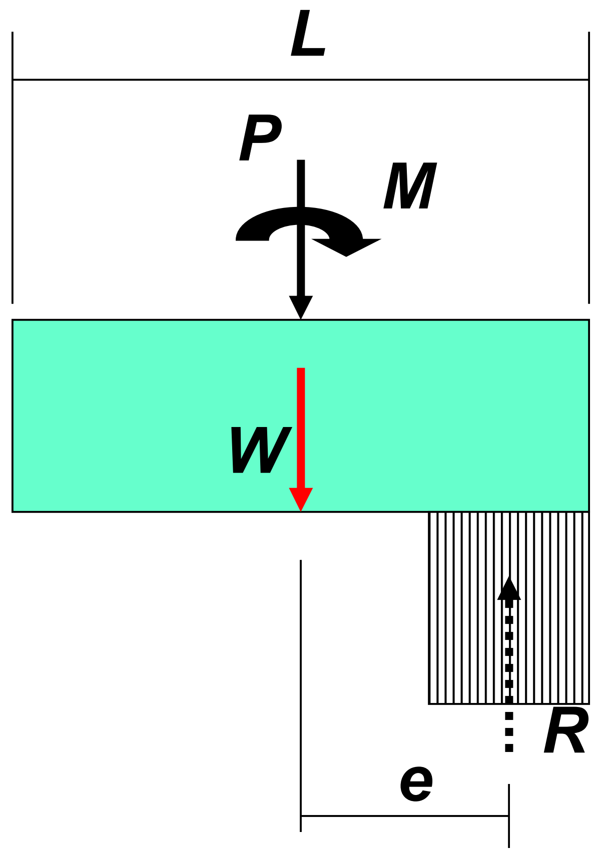
ϕQ_{ns} = 16.2 ksf > 4.98 ksf

OK by elastic analysis



Plastic Response

- Same objective as for elastic response
- Smaller footings can be shown OK thus



Counteracting Case

- Given ***P*** = 332 k; ***M*** = 5712 k-ft
- **e = M / P** = 17.2 ft
- Try ***L*** = 40 ft, B = 9 ft
- Try elastic solution:
  - L' = 3(40/2 – 17.2) = 8.4 ft
  - qmax = 2 **P** / [3 **B**(**L**/2-**e**)] = 8.82 ksf
- Bearing Capacity:
  - Qns = 4,000 min (9,8.4/2) = 16,800 psf
  - ϕQns = 0.45 (16.8 ksf) = 7.56 ksf < qmax **NG**

Counteracting Case

Try plastic solution:

L' = 4.54 ft

Q_{ns} = 4,000 min (9,4.54)

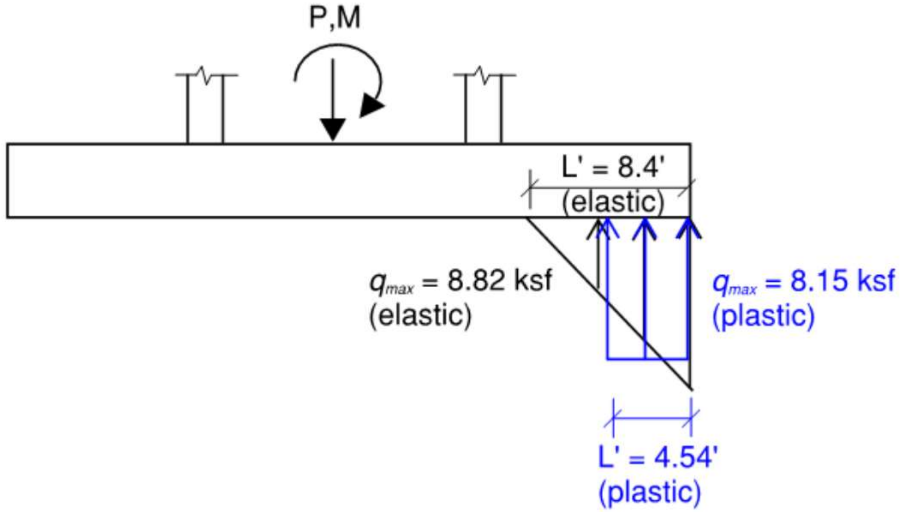
Q_{ns} = 18,120 psf

ϕQ_{ns} = 0.45(18.12) = 8.15 ksf

Check equilibrium:

(8.15 ksf)(4.54')(9') = 332 kips = P, satisfied

OK by plastic analysis



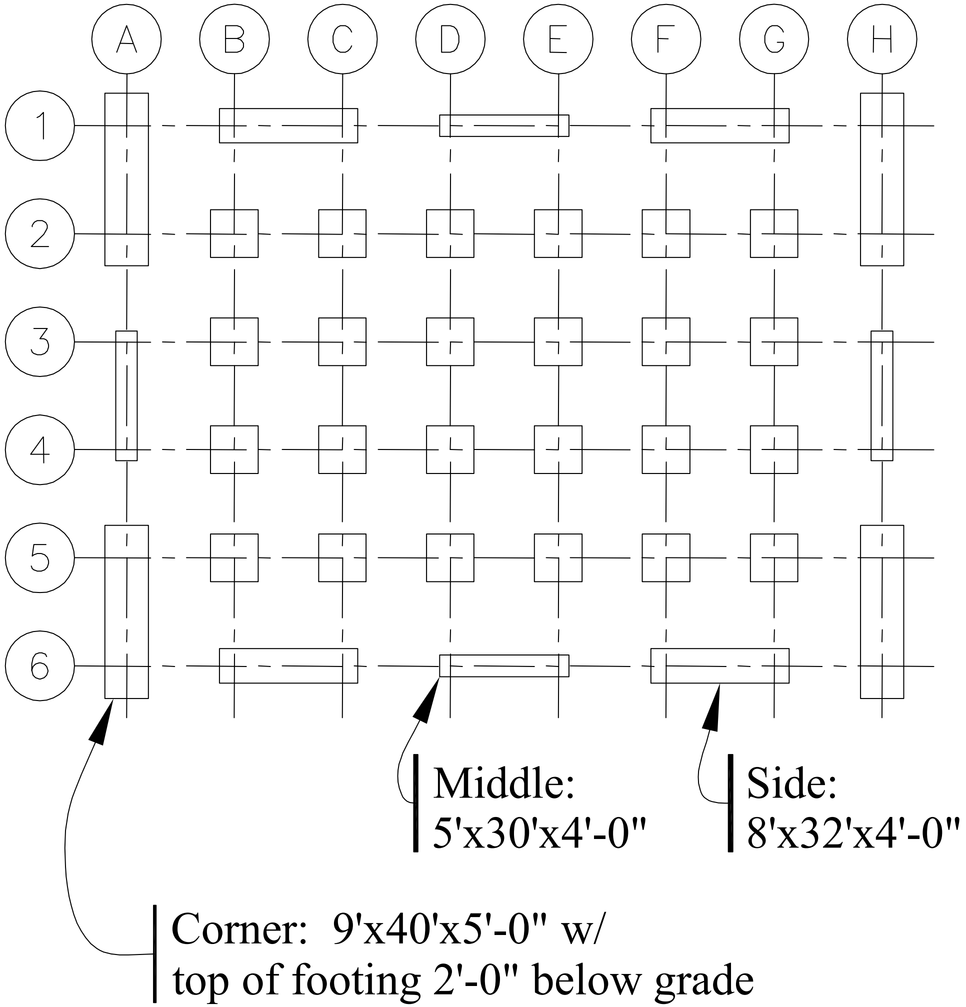
Settlement Analysis

- Long term settlement verification
- Load combination: D+0.5L per Appendix C Commentary (ASCE 7)
- P = 340 kips
- $Q_{sustained}$ = 340,000 / (9’ x 40’) = 945 psf
- $Q_{allowable}$ = 2,000 psf
- **OK for settlement**

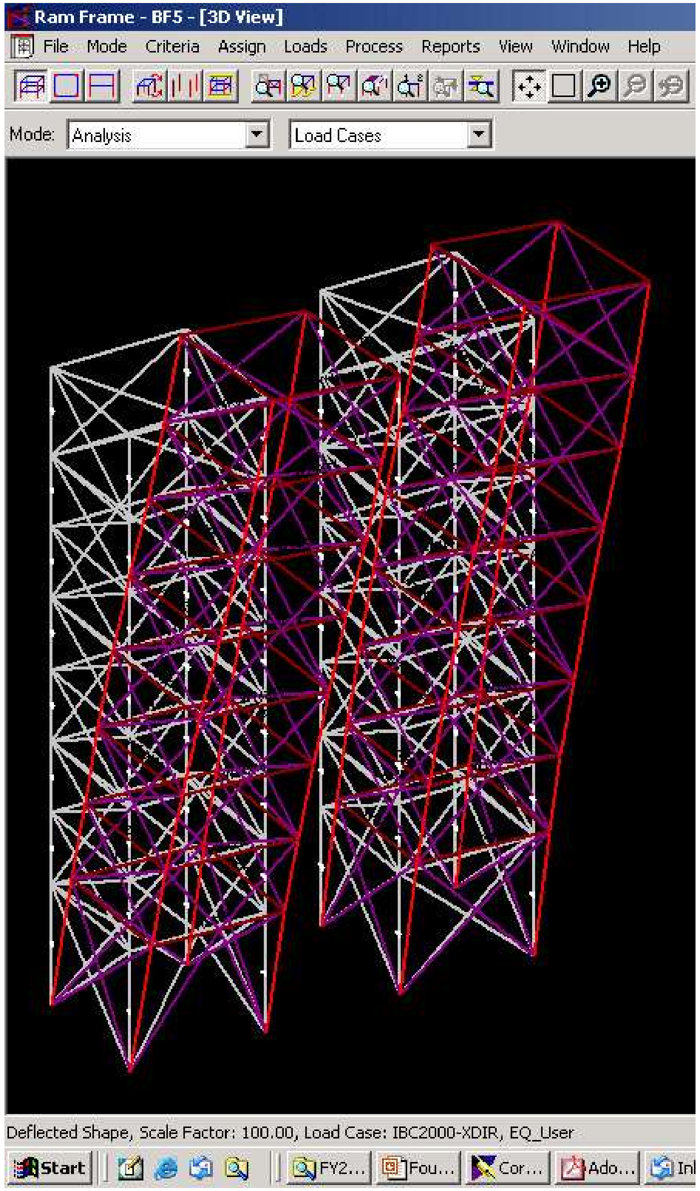
Additional Checks

- Moments and shears for reinforcement should be checked for the overturning case
- Plastic soil stress gives upper bound on moments and shears in concrete
- Horizontal equilibrium: $\boldsymbol{H_{max}} < \phi\mu(\boldsymbol{P}+\boldsymbol{W})$

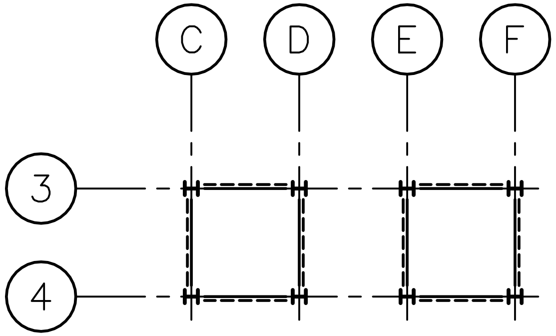
  in this case friction exceeds demand; passive could also be used



Results for all Seismic Resistant System Footings

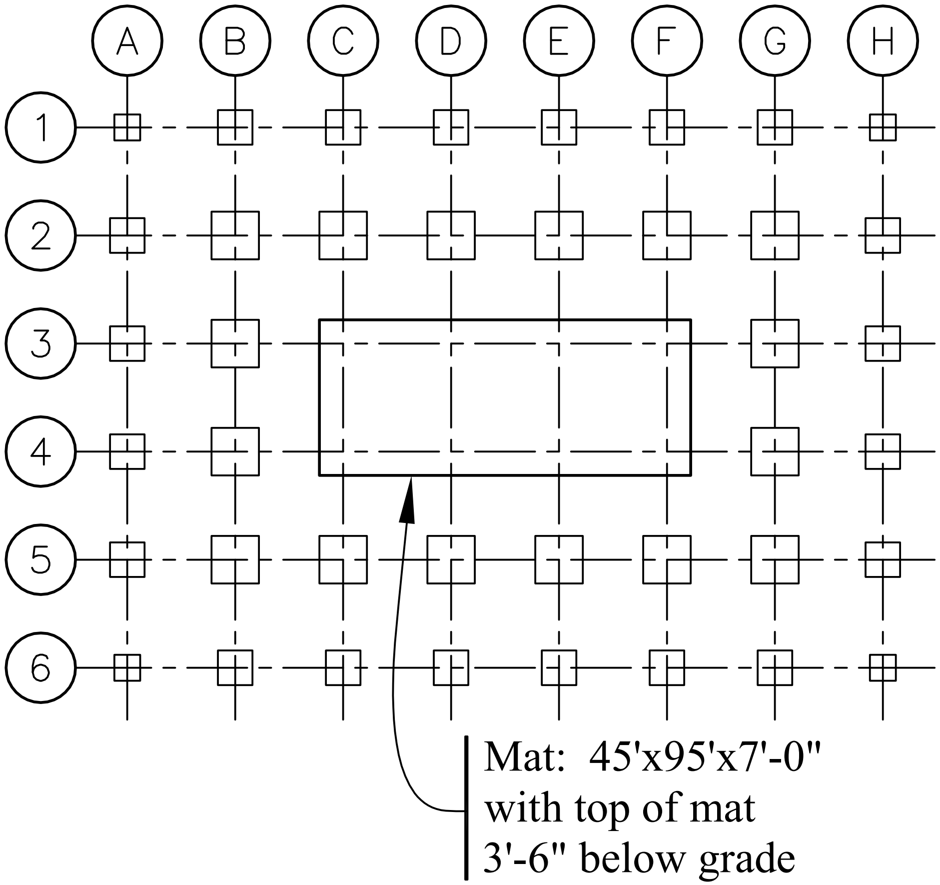


Design of footings for core-braced 7 story building



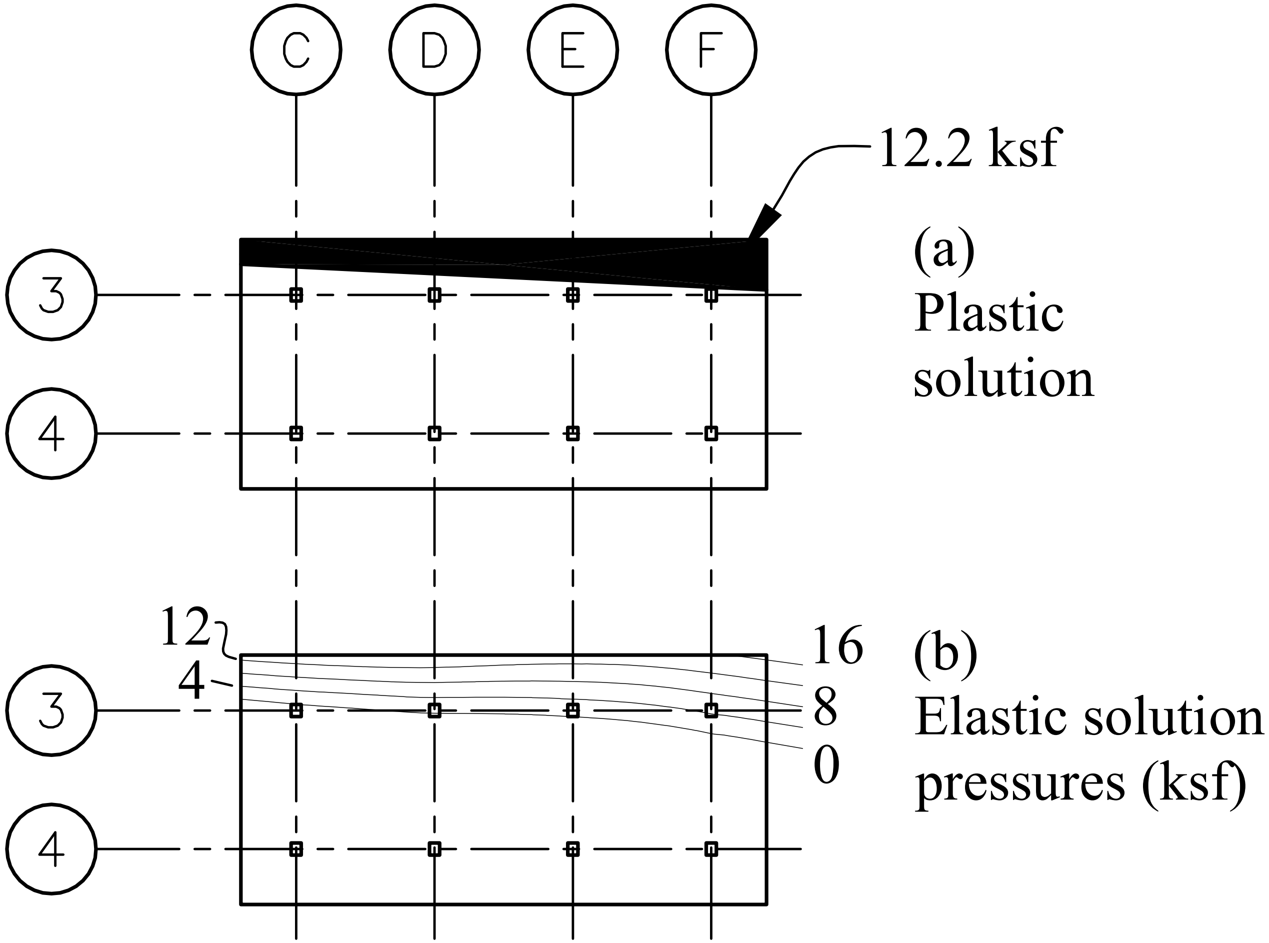
25 foot square bays at center of building

Solution for Central Mat



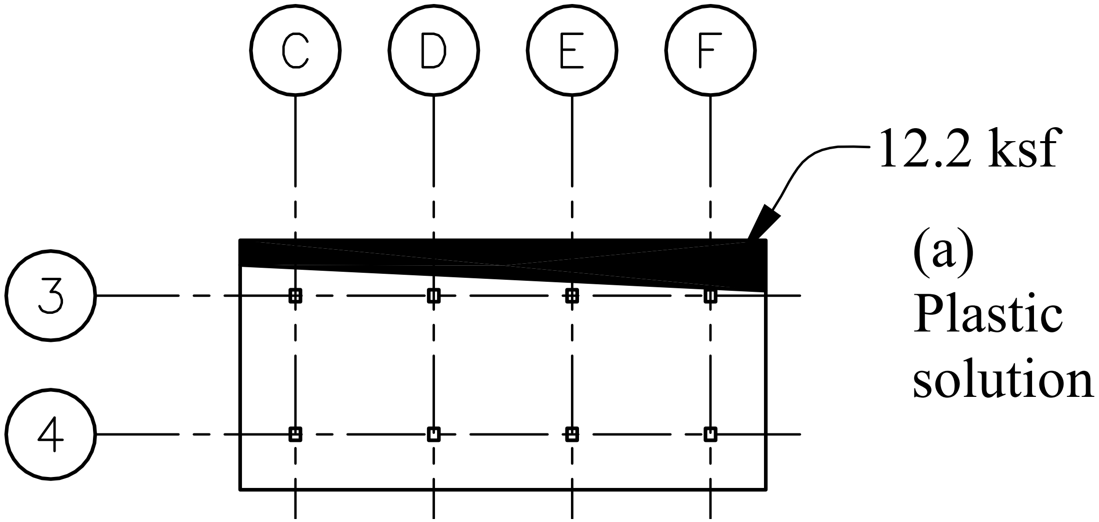
Very high uplifts at individual columns; mat is only practical shallow foundation

Bearing Pressure Solution



Plastic solution is satisfactory; elastic is not

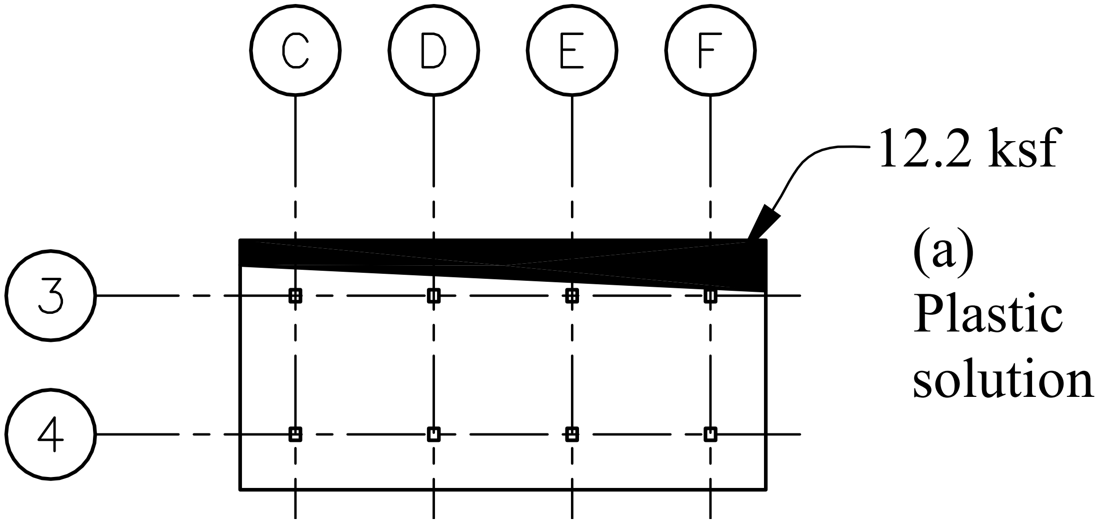
Central Mat Bearing



Design bearing capacity:

ϕQ_{ns} = 0.45 (27.12 ksf) = 12.21 ksf **OK**

Central Mat Bearing

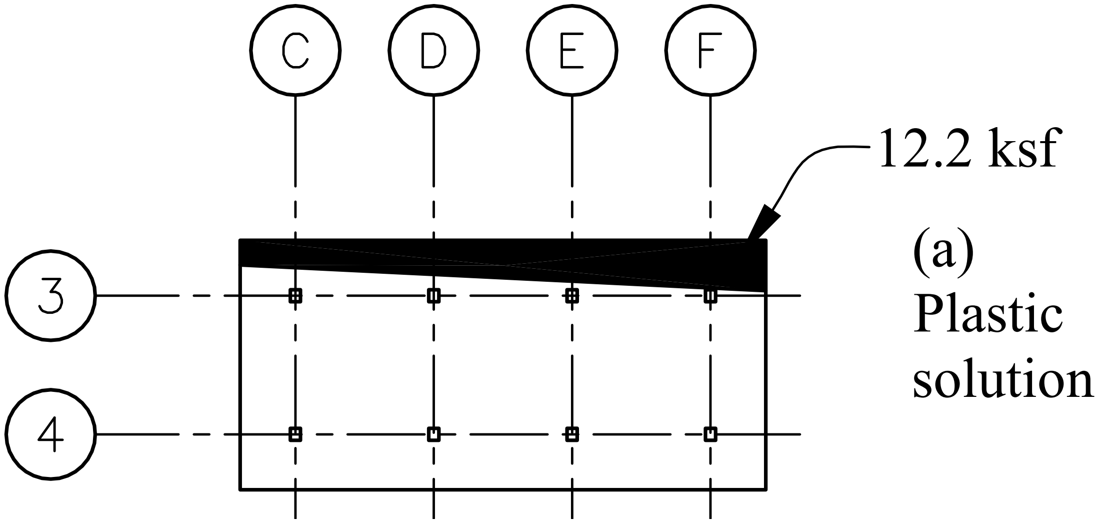


Verify equilibrium:

(12.21)(6.78)(95) = 7864 kips ≈ Axial Load (1749 kips)

(7849)(5.42) = 42,542 ft-kips ≈ Off-axis moment (42,544 ft-kips)

Central Mat Bearing



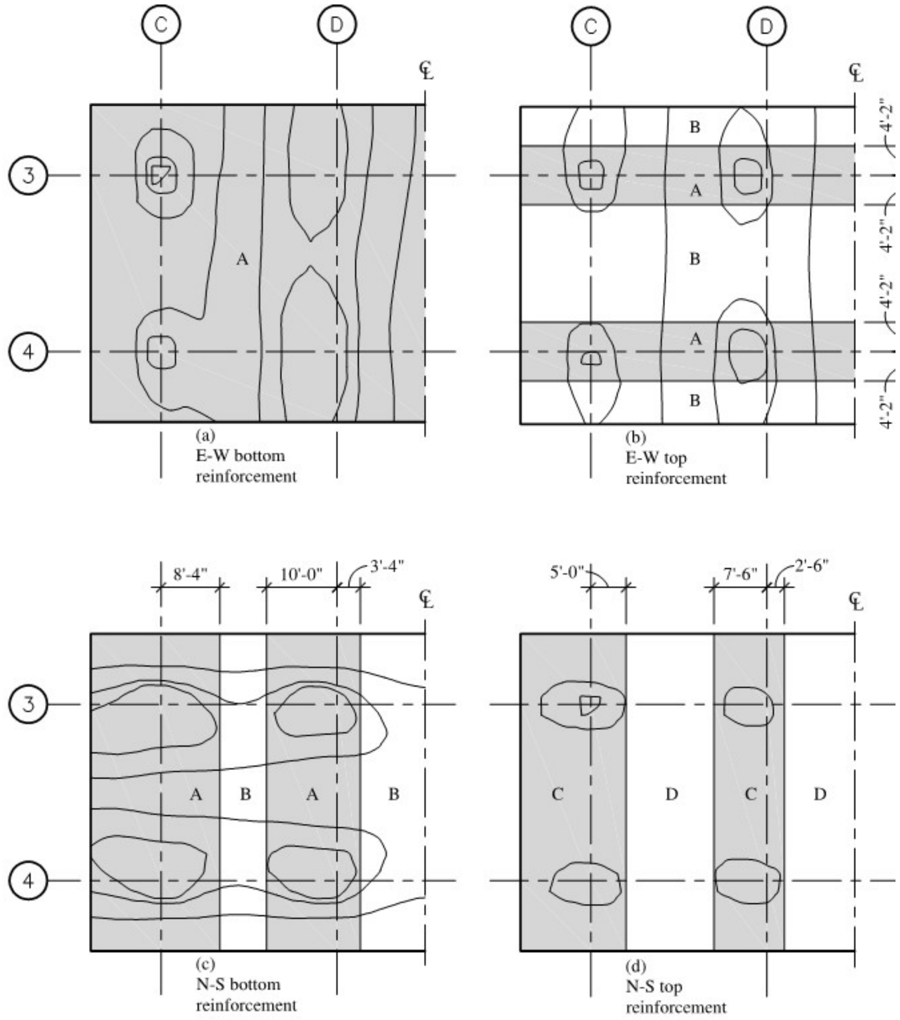
Verify equilibrium:

(12.21 ksf)(6.78 ft)(95 ft) = 7864 kips ≈ Axial Load (1749 kips)

(7849 kips)(5.42 ft) = 42,542 ft-kips ≈ Off-axis moment (42,544 ft-kips)

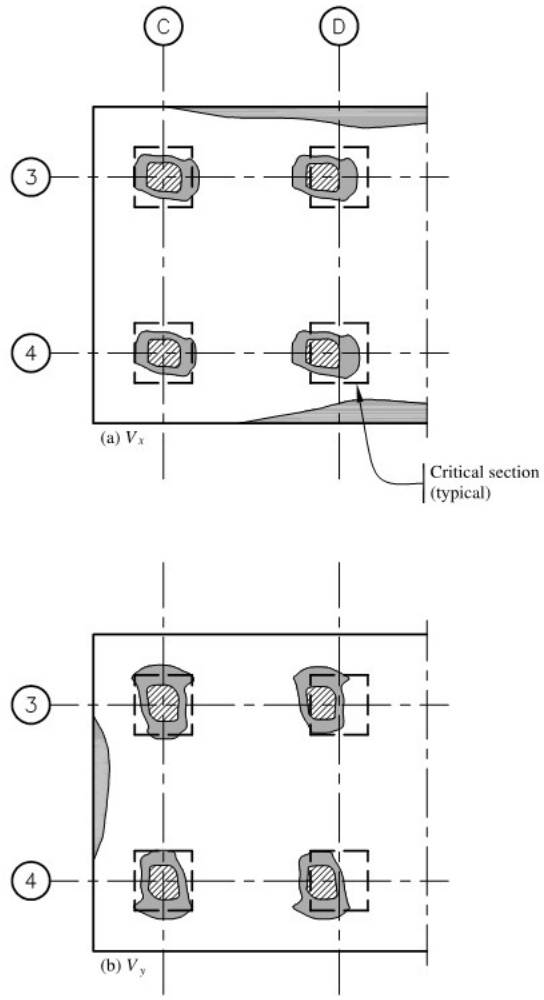
Central Mat Flexural Reinforcement

- Use moment contours to define areas of flexural reinforcement density

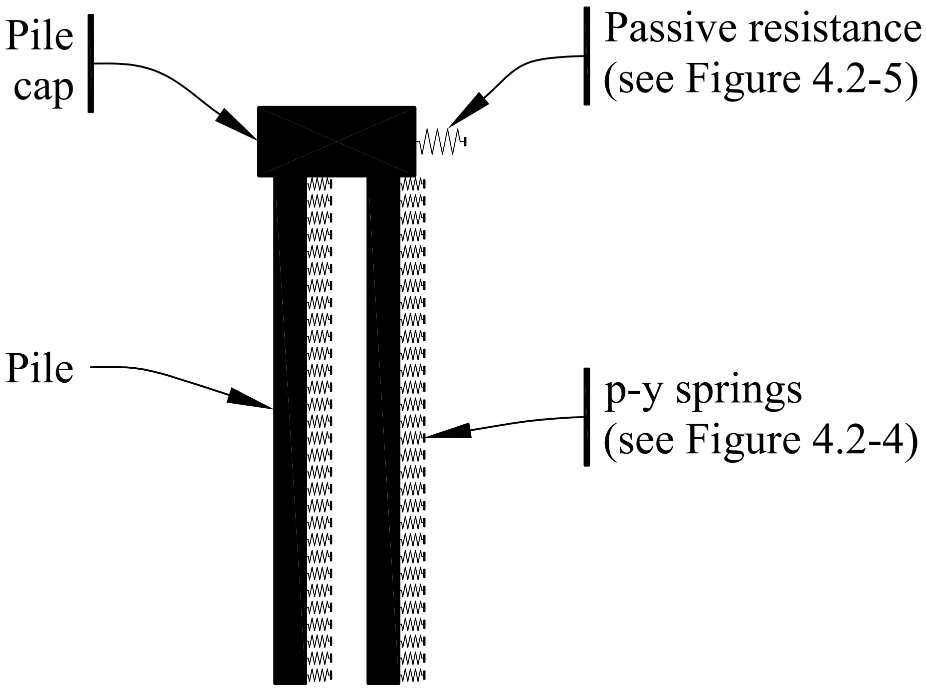


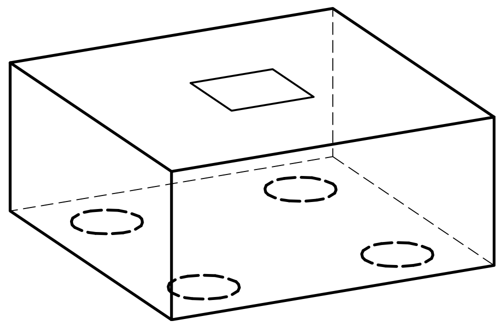
Central Mat Shear Design

- Critical section at d/2 (two-way) and d (one-way)
- Be aware of size-effect in thick foundation elements



Pile/Pier Foundations





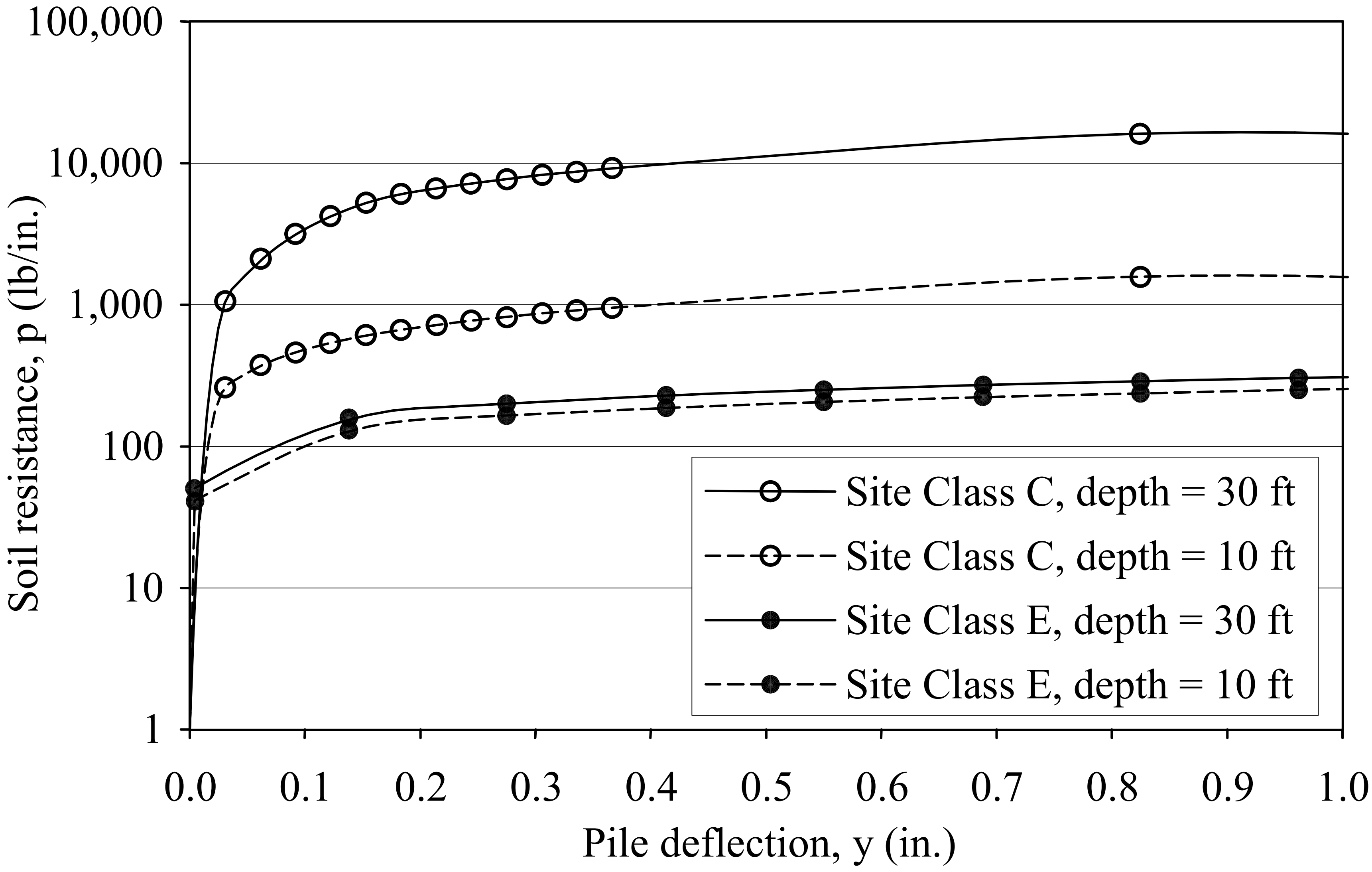
View of cap with column above and piles below

Pile/Pier Foundations

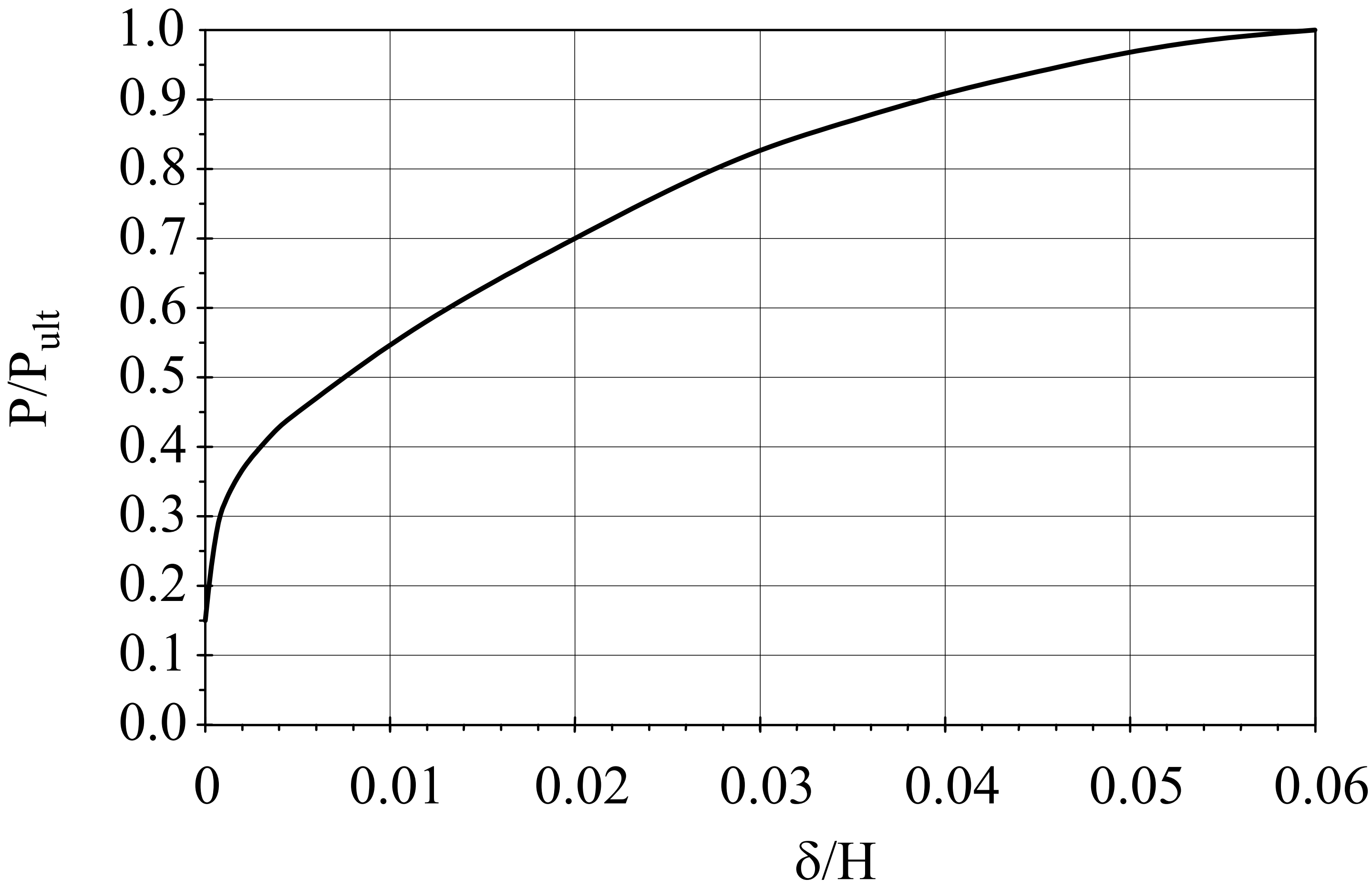
- Pile Stiffness:
  - Short (Rigid)
  - Intermediate
  - Long
- Cap Influence
- Group Action

- Soil Stiffness
  - Linear springs – nomographs e.g. NAVFAC DM7.2
  - Nonlinear springs – LPILE or similar analysis

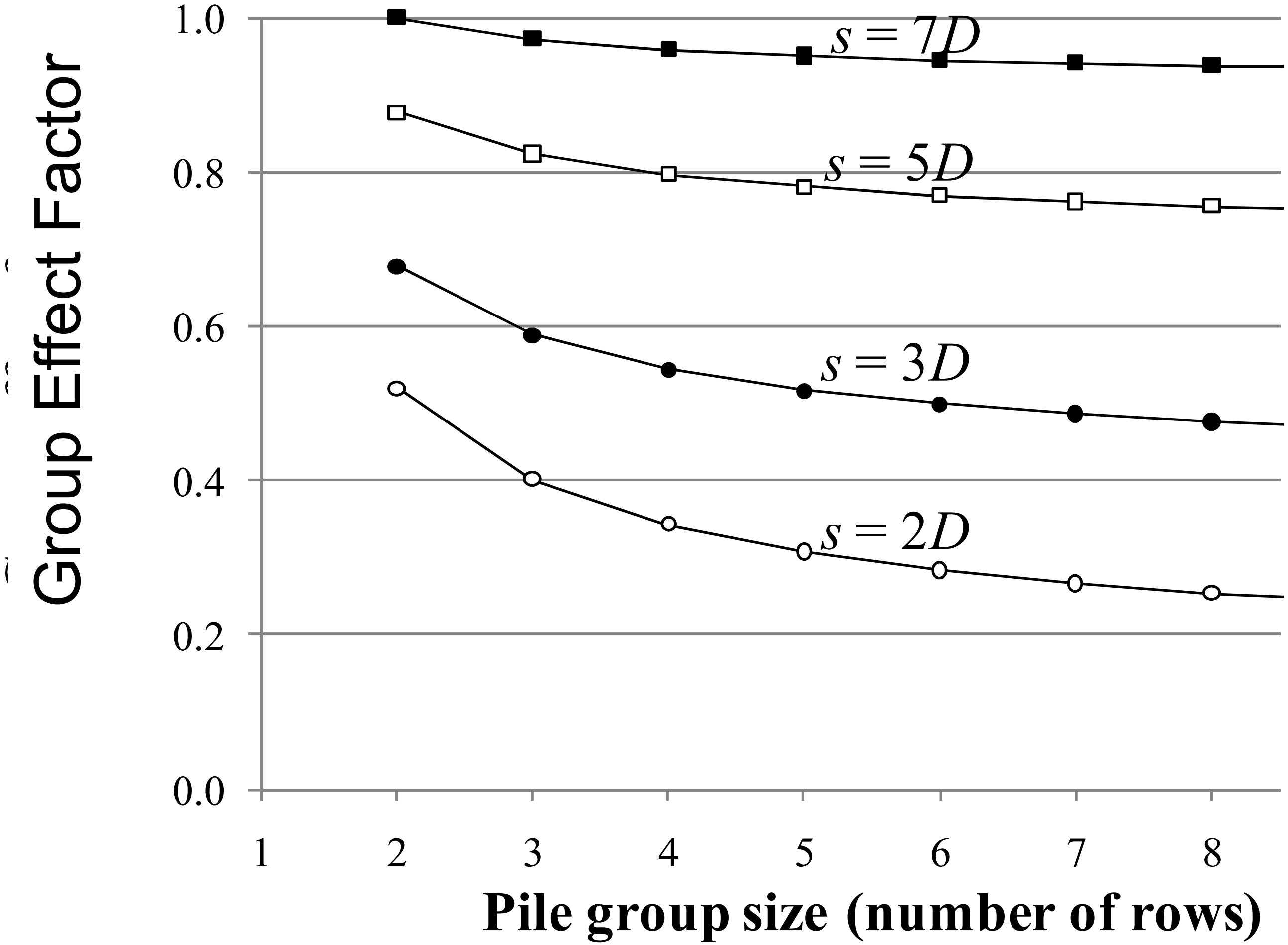
Sample *p-y* Curves



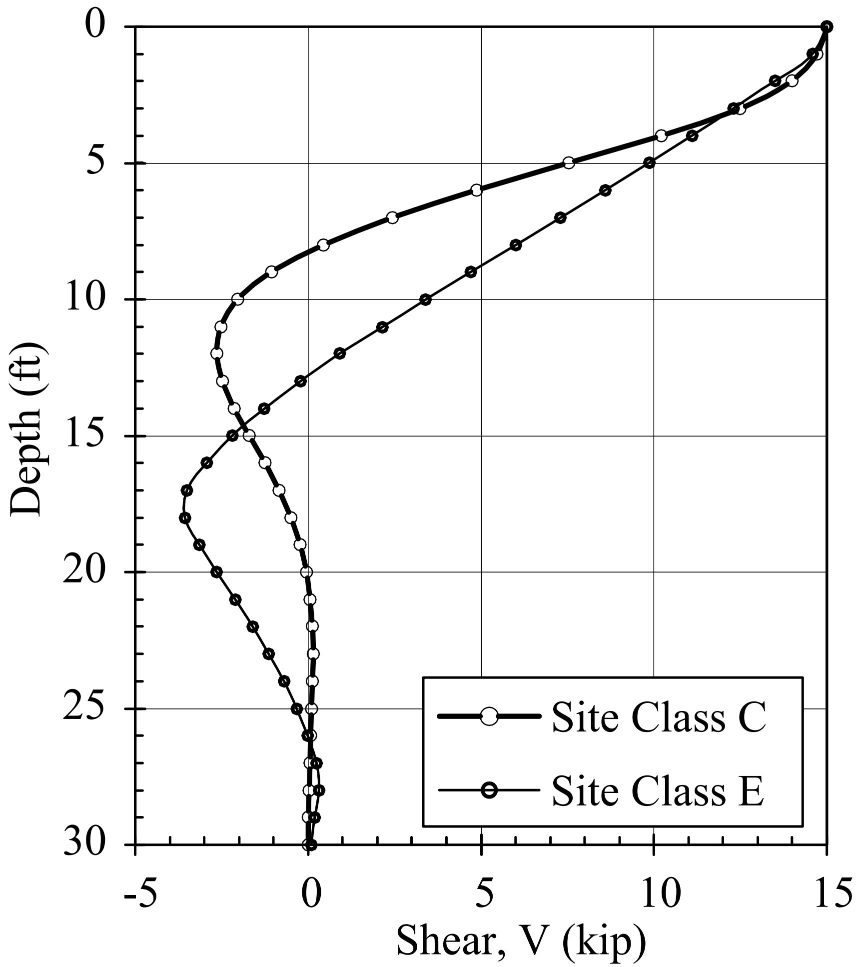
Passive Pressure



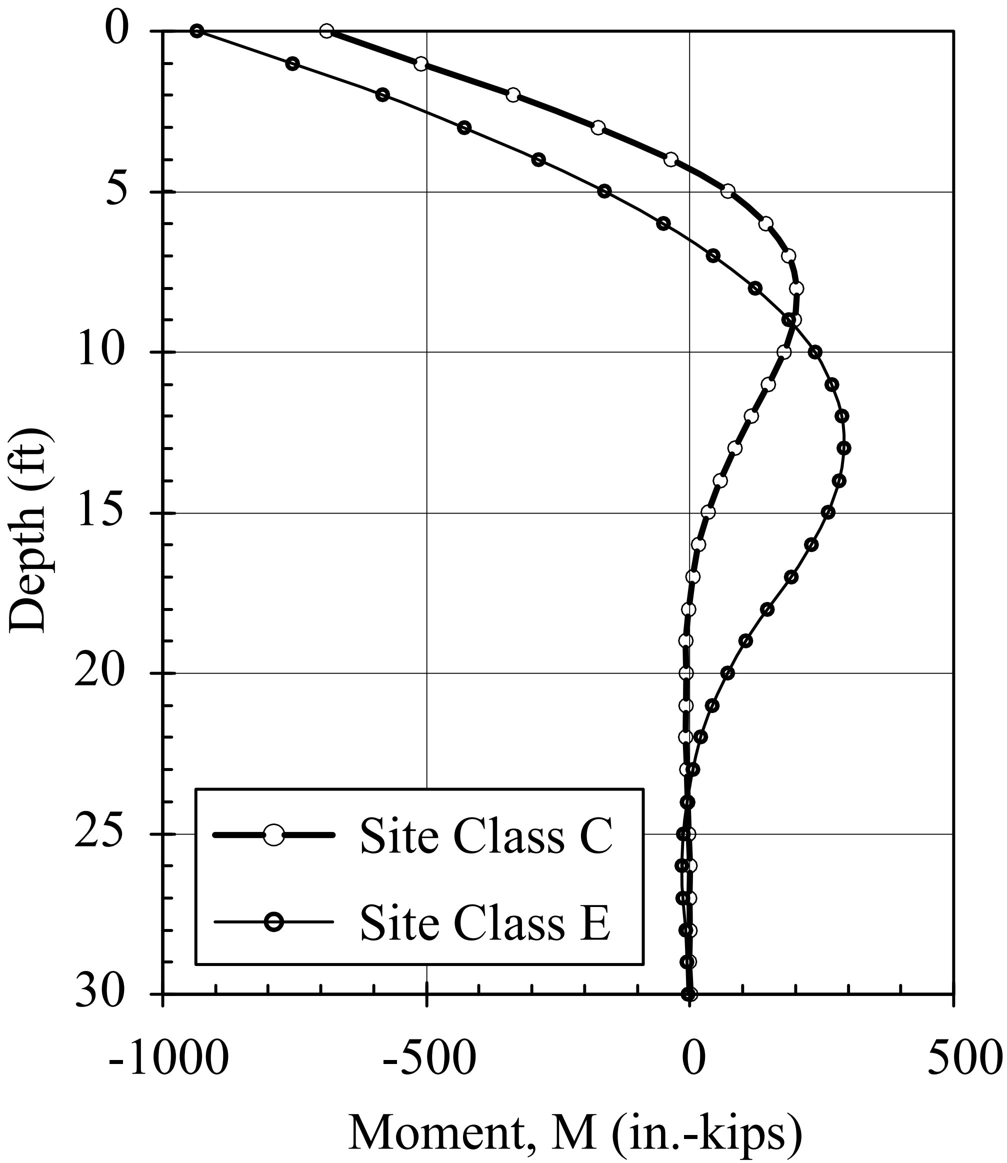
Group Effect



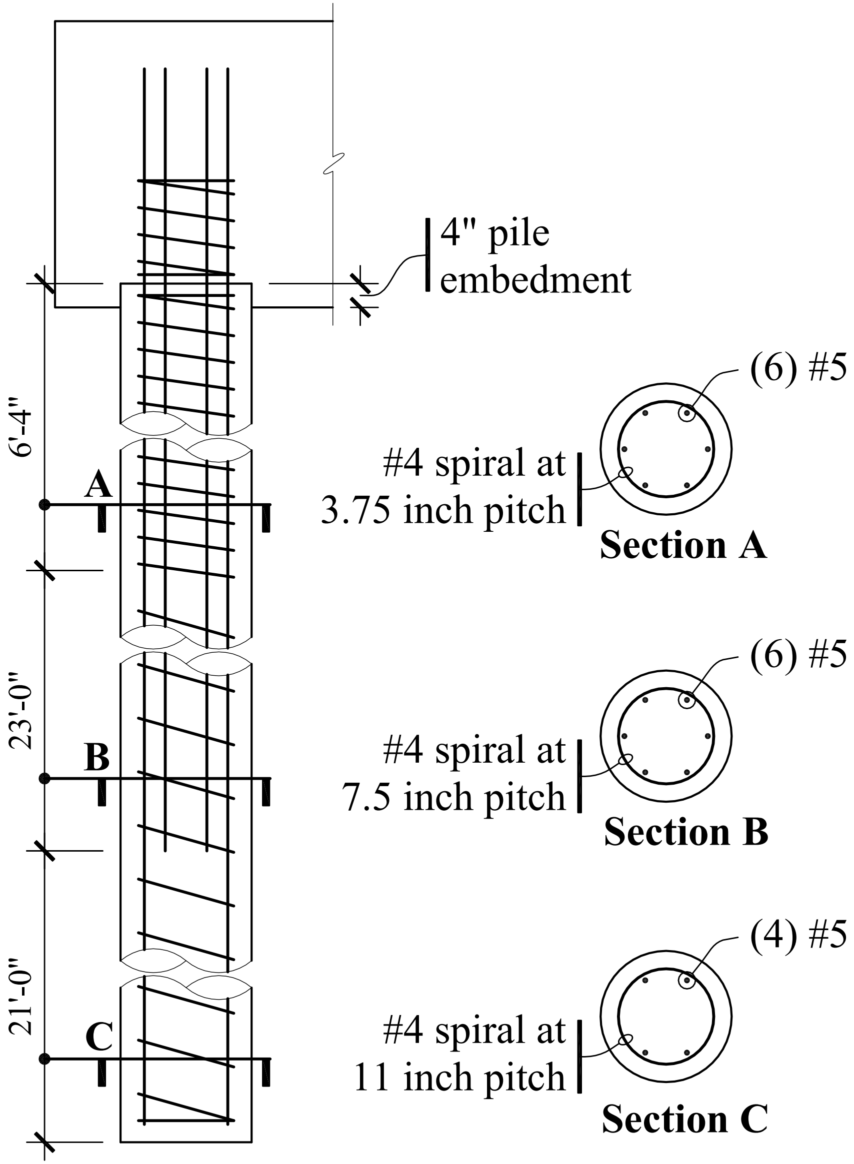
Pile Shear: Two Soil *Stiffnesses*



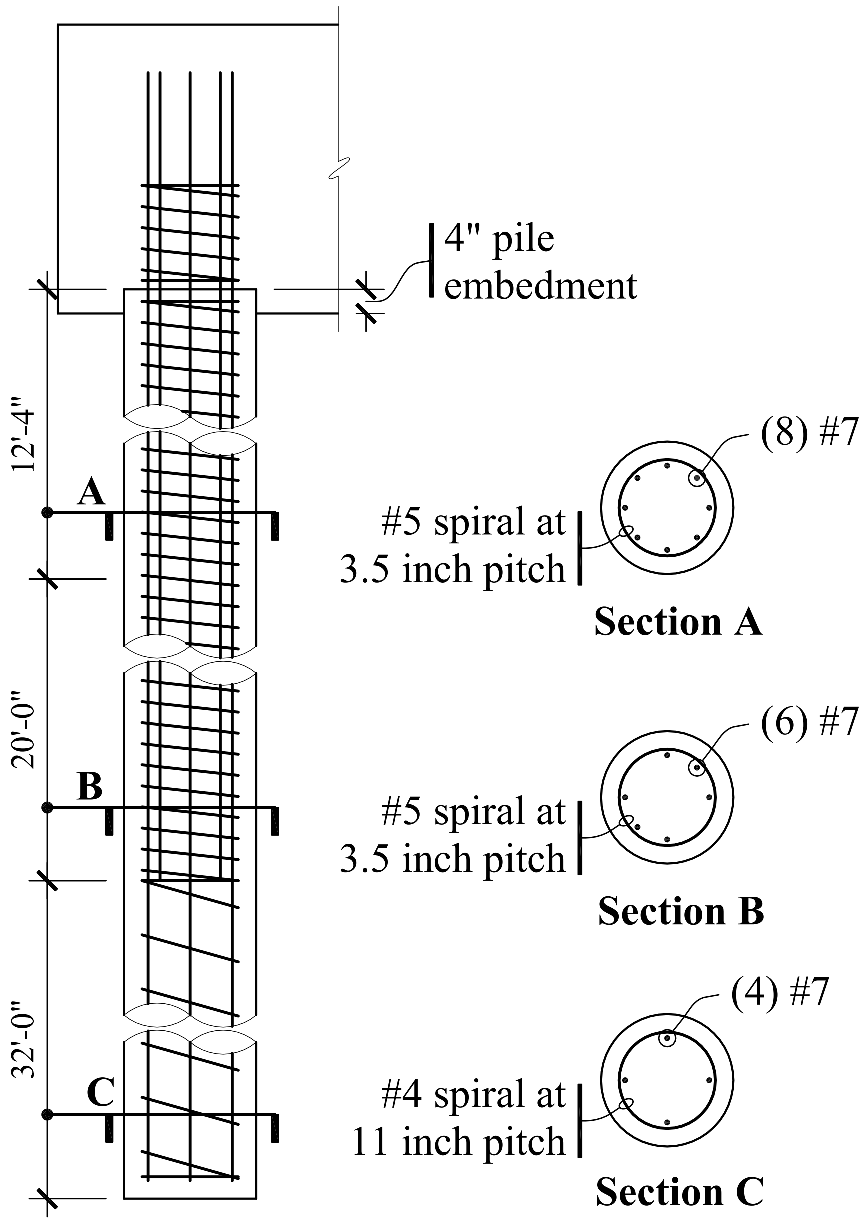
Pile Moment vs Depth



Pile Reinforcement



- Site Class C
- Larger amounts where moments and shears are high
- Minimum amounts must extend beyond theoretical cutoff points
- “Half” spiral for 3D



Pile Design

- Site Class E
- Substantially more reinforcement
- "Full" spiral for 7D
- Confinement at boundary of soft and firm soils (7D up and 3D down)

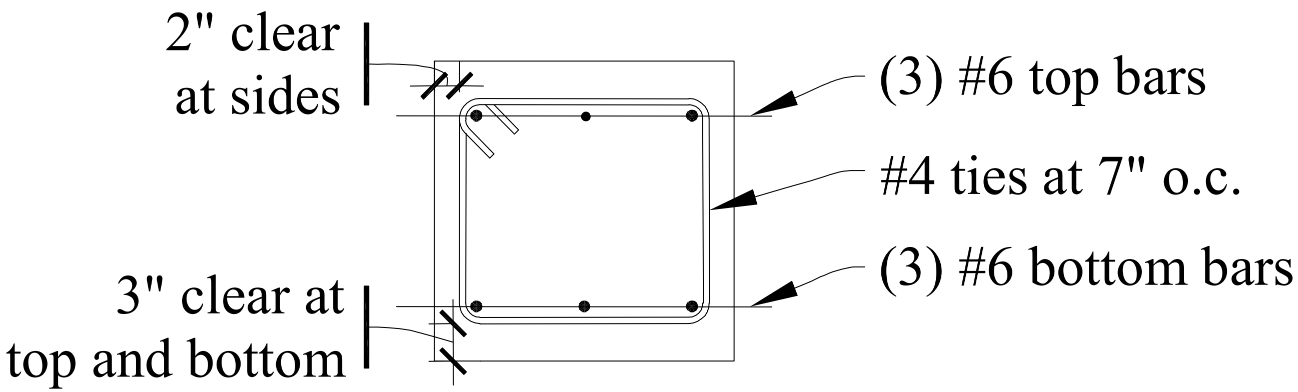
Other Topics for Pile Foundations

- Foundation Ties: $\boldsymbol{F} = \boldsymbol{P_G}(\boldsymbol{S_{DS}}/10)$
- Pile Caps: high shears, rules of thumb; look for 3D strut and tie methods in future
- Liquefaction: another topic ISM2
- Kinematic interaction of soil layers

ISM2 Remove pending Steve Harris material

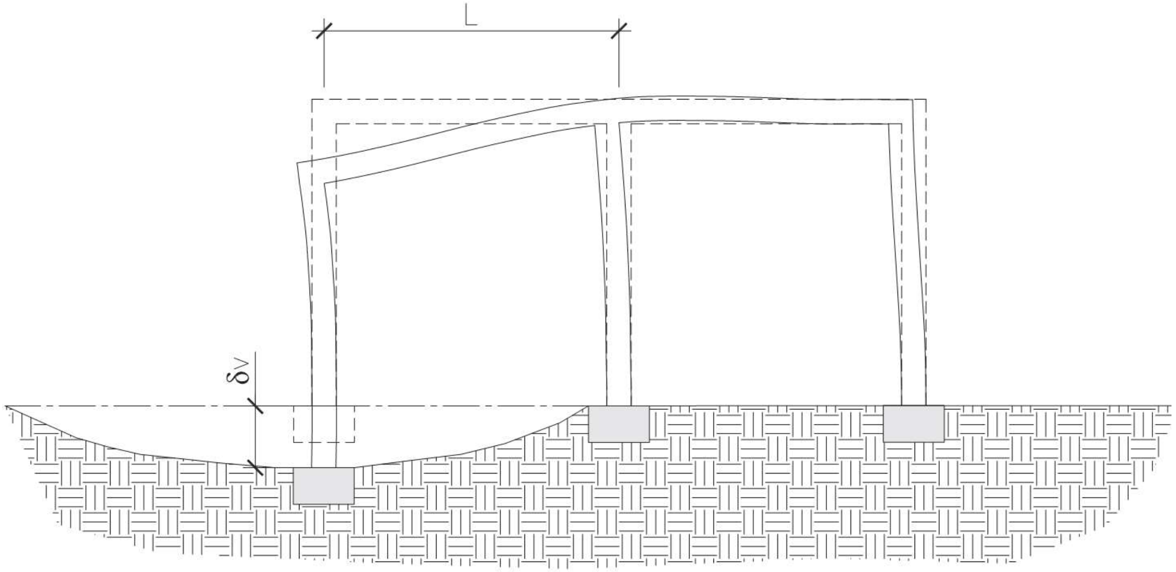
Ian S. McFarlane; 2.05.2016

Tie between pile caps



- Designed for axial force (+/-)
- Pile cap axial load times $S_{DS}/10$
- Oftentimes use grade beams or thickened slabs on grade

Example: Differential Settlement



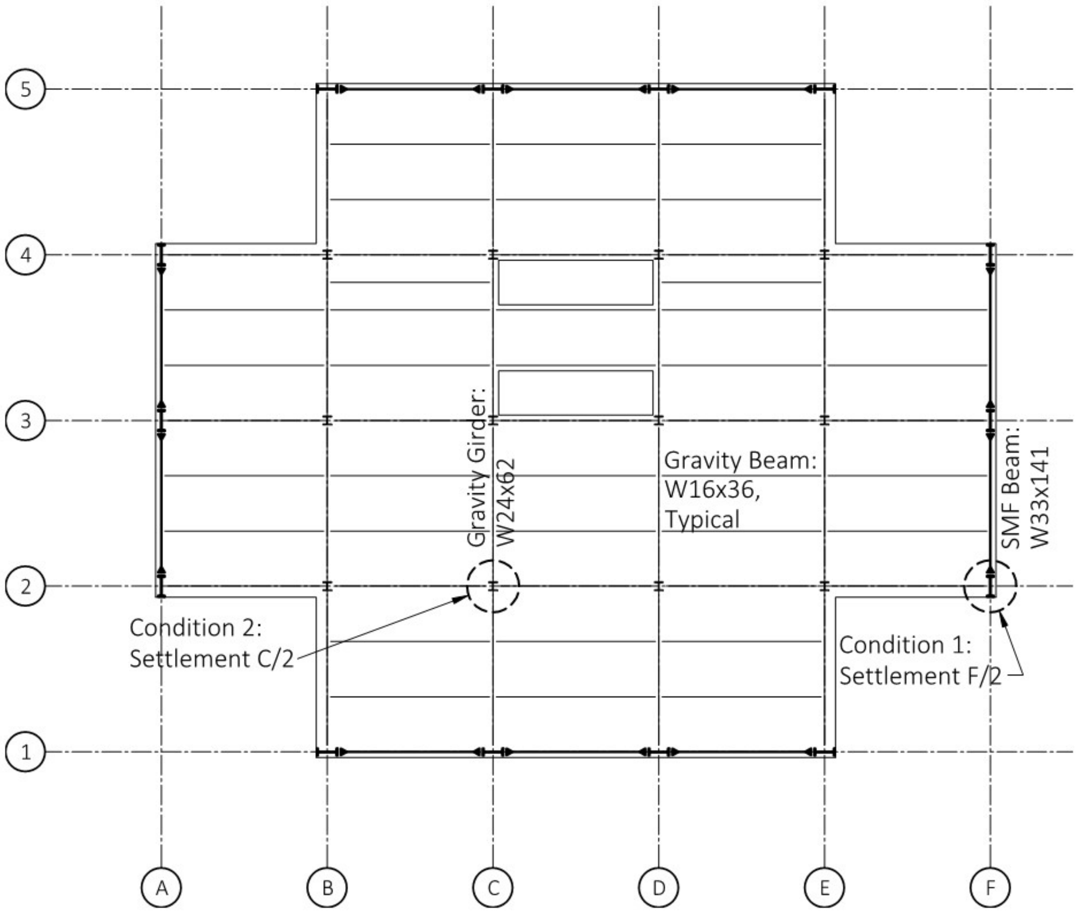
- Predicted differential settlement exceeds limit in Table 12.13-3
- Structure must be shown to perform acceptably when subject to imposed settlement

Structural Requirements

- Analysis is required to use shallow foundations
- No loss of gravity support permitted
- Residual member strength at least 2/3 of undamaged nominal strength
  - If demands exceed nominal strength consideration of nonlinear behavior is required

Floor Framing Plan

- Steel SMF frame building
- Two conditions studied
  - Interior
  - Perimeter
- Expected Settlement, δ_v=8"

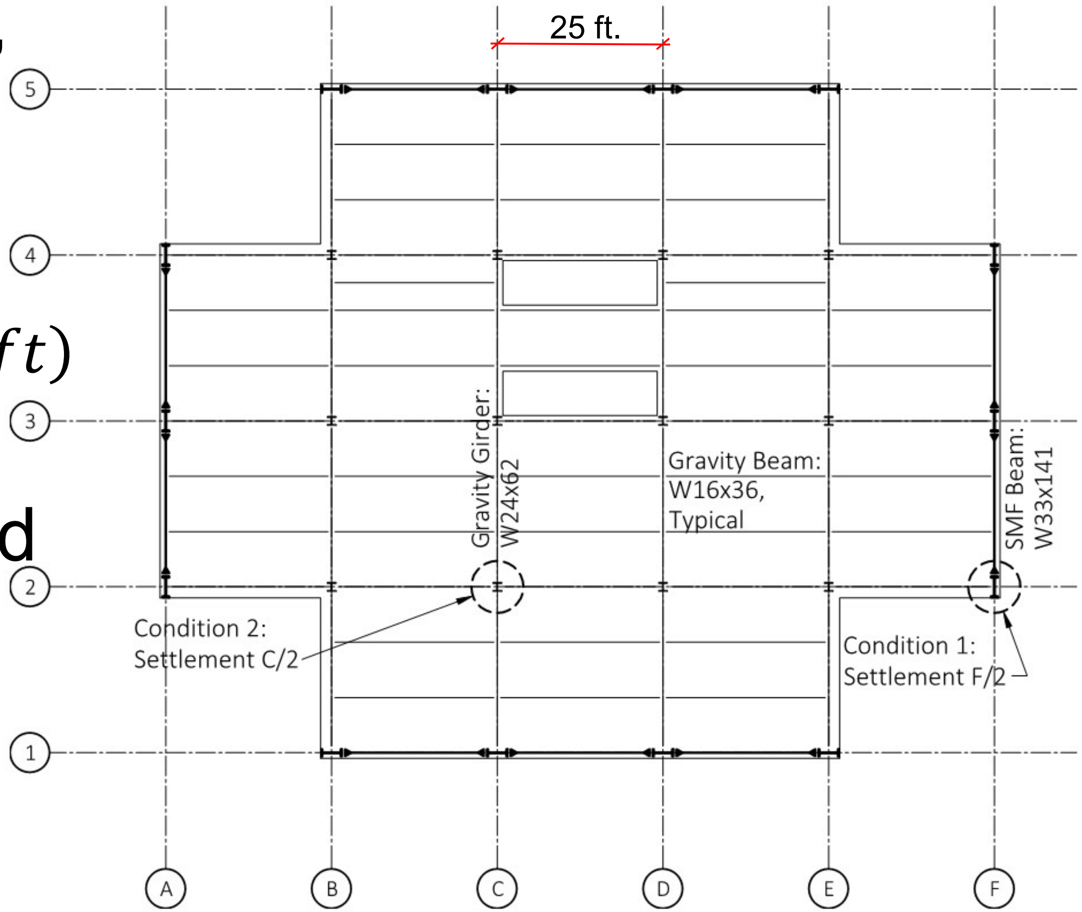


Rotation Demand

- Settlement δ_v=8”
- Bay width 25 ft.

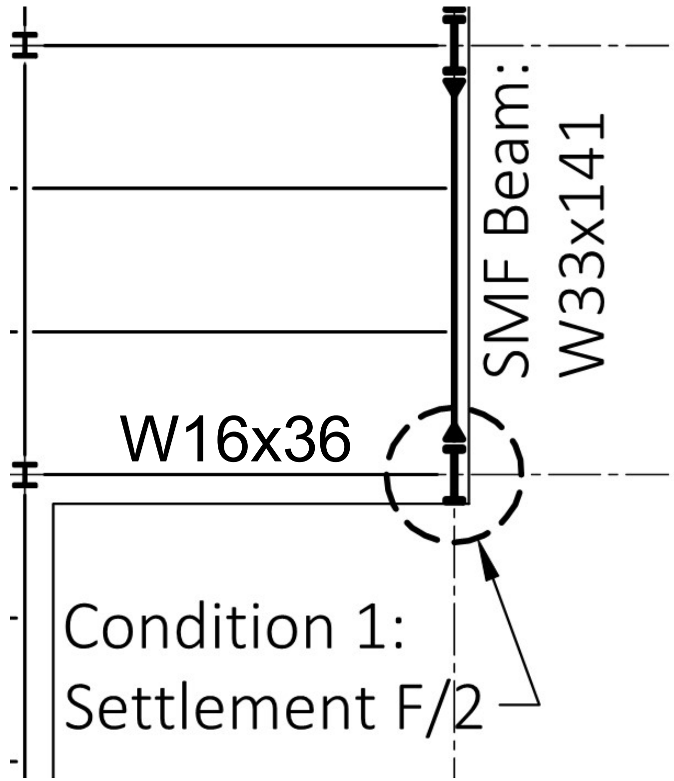
$$\delta_v/L = 8\,in/(25\,ft \times 12\,in/ft) = 0.027\,rad$$

- Exceeds 0.01 rad limit in Table 12.13-3



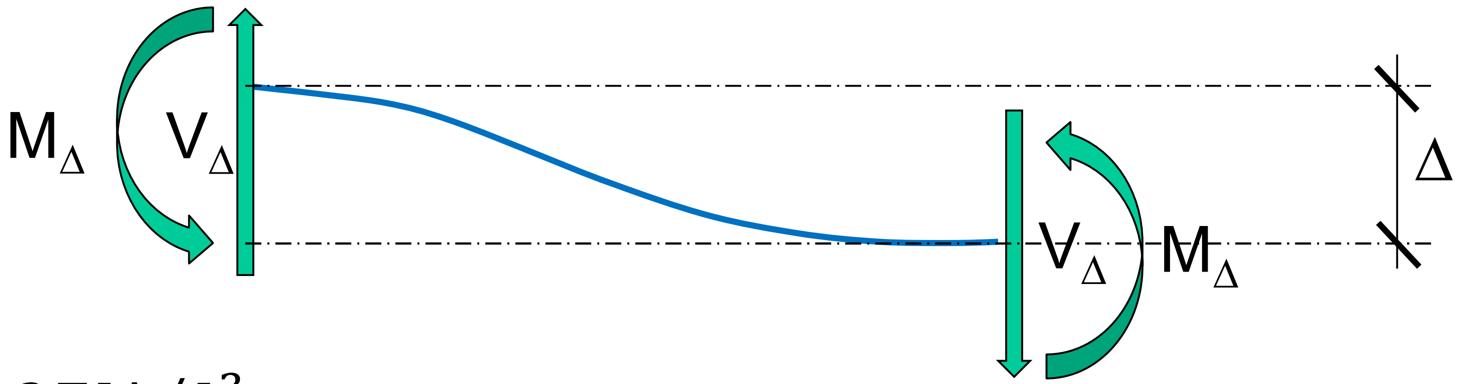
Condition 1

- SMF members and connection can sustain 0.04 rad.
- Must assess:
  - Simple shear tab connection to web
  - Gravity beam
  - Column weak axis



Check of W16x36 Beam

- Consider the beam fixed-ended with an imposed displacement



$$V_\Delta = 12EI\Delta/L^3$$

$$V_\Delta = (12)(29000ksi)(448in^4)(8in)/(25\,ft \times 12\,in/ft)^3 = 46\,kips$$

$$M_\Delta = 6EI\Delta/L^2$$

$$M_\Delta = (6)(29000ksi)(448in^4)(8in)/(25\,ft \times 12\,in/ft)^2$$

$$M_\Delta = 6{,}929\,kip - in = 577\,kip - ft$$

Add gravity loading

- Compute the gravity shear and moment
  - 85 psf dead load, 50 psf live load
  - 4.17 feet tributary width

$$w_{ug} = 1.2(4.17 \times 85 + 280) + 0.5(4.17 \times 50) = 0.866\ kip/ft$$

$$V_{ug} = w_{ug}l/2 = (.866)(25)/2 = 11\ kips$$

$$M_{ug} = w_{ug}l^2/12 = (0.866)(25)^2/12 = 45\ kip - ft$$

$$V_u = 46 + 11 = 57\ kips$$

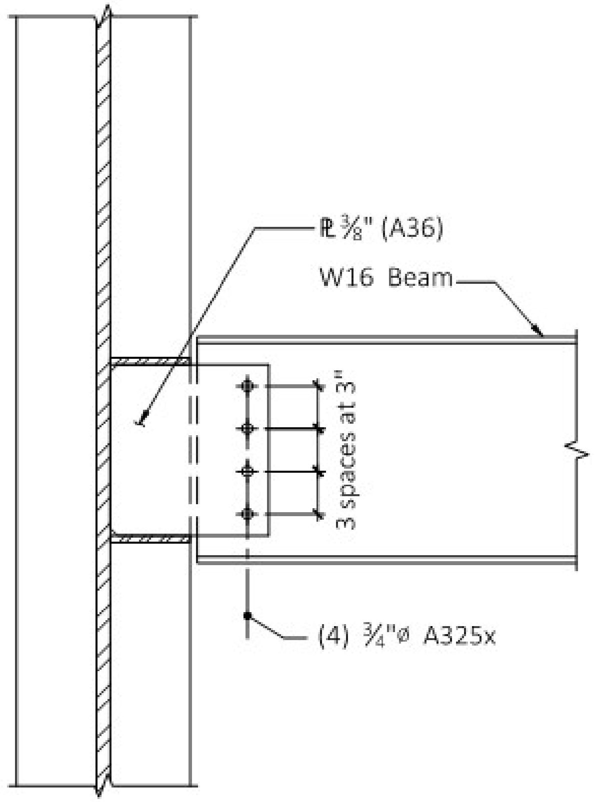
$$M_u = 577 + 45 = 622\ kip - ft$$

Compare to design values

- Shear capacity ϕVn = 141 kip
  - Beam is adequate for shear
- Flexural capacity ϕMn = 174 kip-ft
  - Demand exceeds flexural capacity
  - Either the beam will yield in flexure or the connection will yield

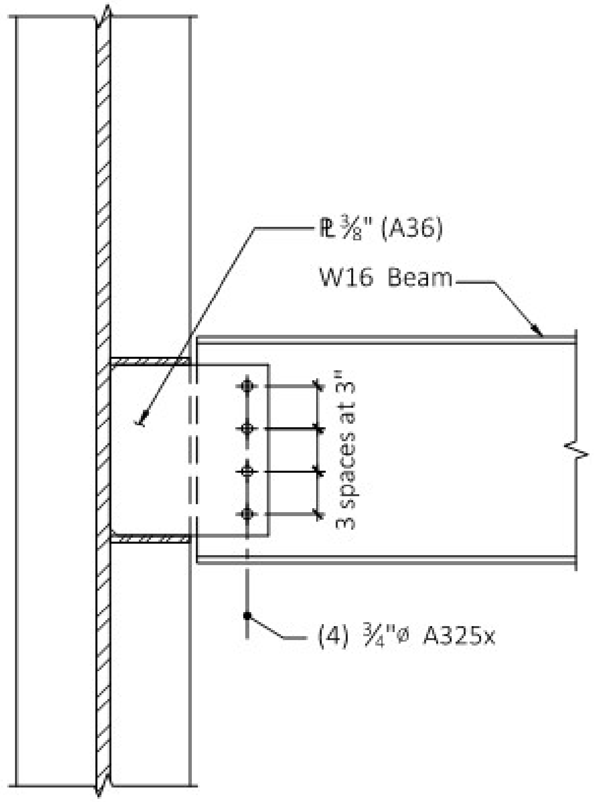
Check Connection

- Shear capacity
  ϕVn = 78.3 kip
- Flexural capacity
  ϕMn = 21 kip-ft
- Connection will yield in flexure



Check Connection

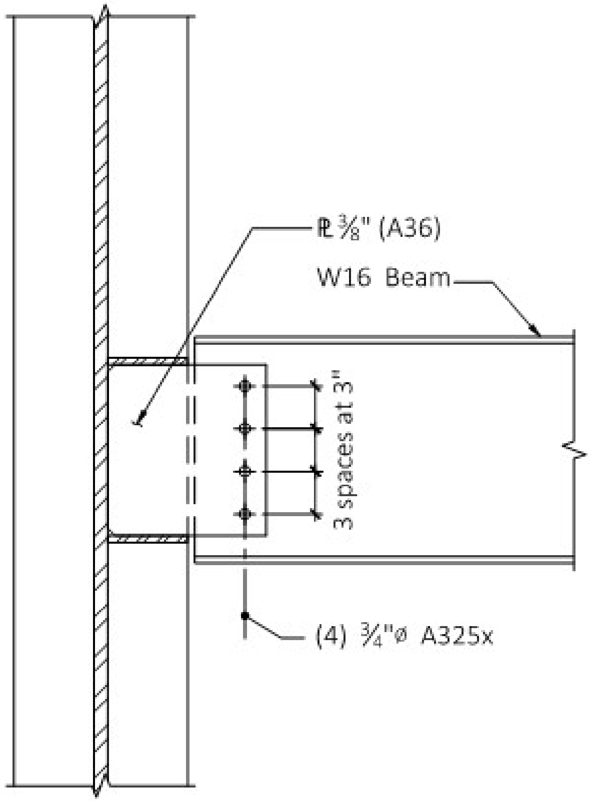
- Check plate thickness to ensure ductile behavior (plate yielding)
- Check permissible rotation per ASCE 41



Check Connection

- Maximum plate thickness is acceptable
  - But *not* if it were 50 ksi steel

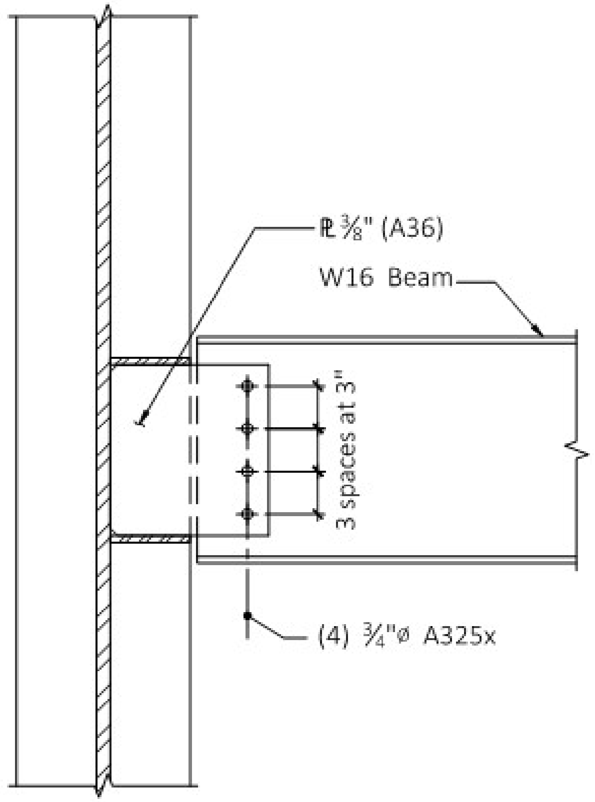
$$t_{max} = \frac{6F_v(A_b C')}{0.90F_y d^2} = 0.44\ in.$$



Check Connection

- Check plastic rotation to provide collapse resistance per ASCE 41
  - Acceptable

$$\theta_{CP} = 0.15 - 0.0036 d_{bg}$$
$$= 0.117\ rad$$

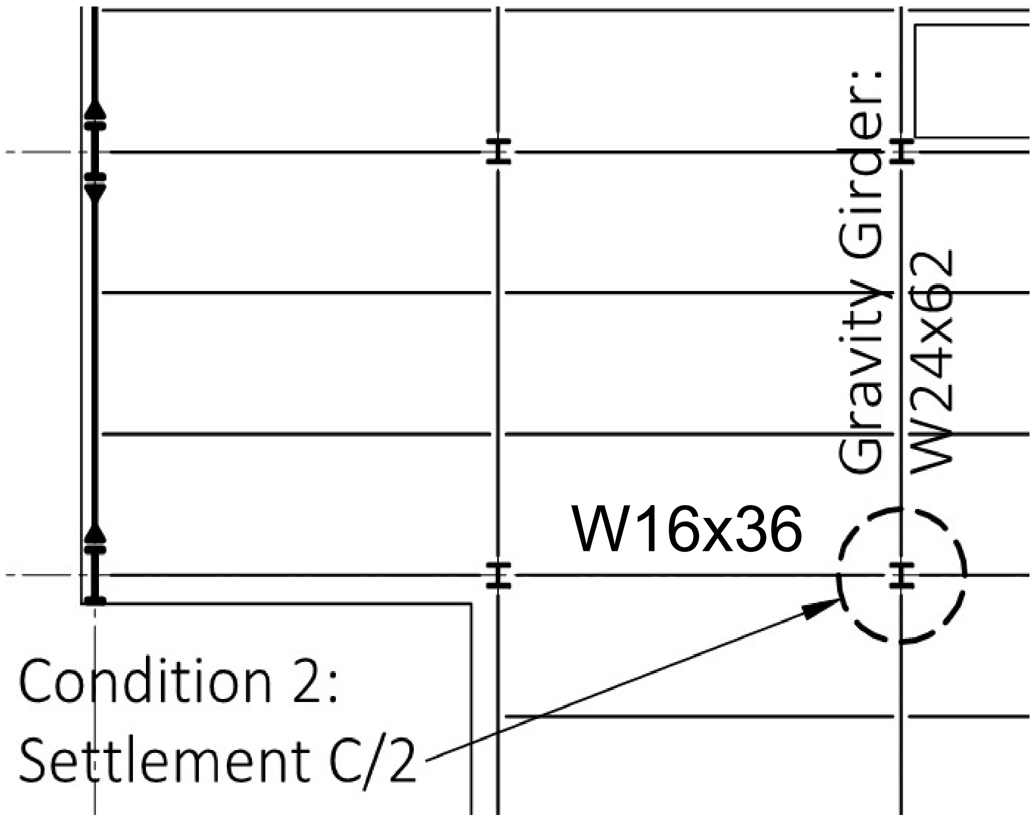


Bi-Axial bending of column F/2

- Considered acceptable, because:
  - SMF column is known to be able to resist any moment the beam can impart
  - Yield moment of weak axis connection is so small as to be considered negligible

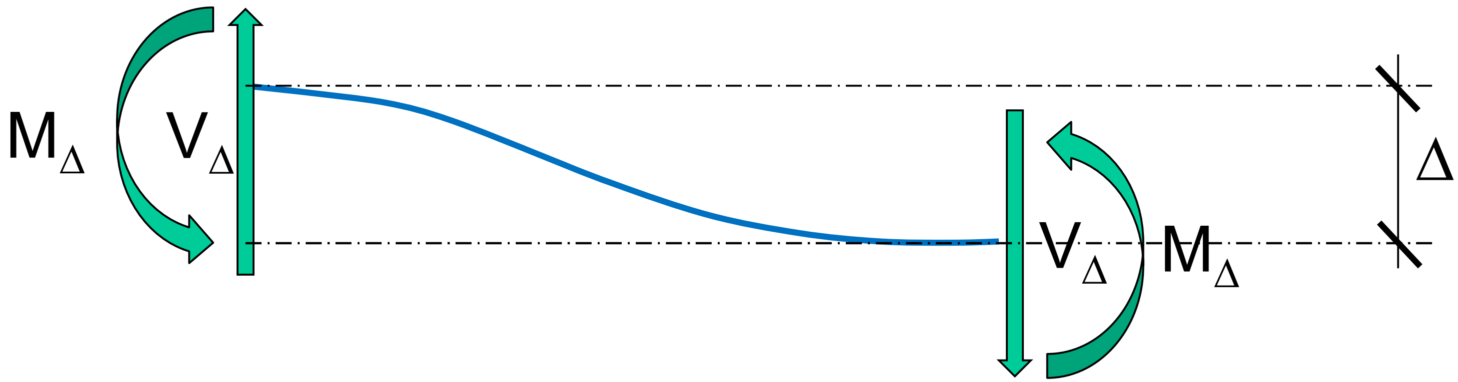
Condition 2

- Must assess:
  - Gravity girder



Check of W24x62 Girder

- Consider the beam fixed-ended with an imposed displacement



$$V_\Delta = (12)(29000ksi)(1{,}5550in^4)(8in)/(25\ ft \times 12\ in/ft)^3 = 160\ kips$$

$$M_\Delta = (6)(29000ksi)(1{,}550in^4)(8in)/(25\ ft \times 12\ in/ft)^2$$

$$M_\Delta = 23{,}973\ kip - in = 1{,}998\ kip - ft$$

Add gravity loading

- Compute the gravity shear and moment
  - 85 psf dead load, 50 psf live load
  - Two equal point loads of 208 sq. ft. each

$$P_{ug} = 1.2(208 \times 85) + 0.5(208 \times 50) = 26\ kips$$

$$V_{ug} = P_{ug} = 26\ kips$$

$$M_{ug} = P_{ug}l/3 = (26)(25\ ft)/3 = 217\ kip - ft$$

$$V_u = 160 + 26 = 186\ kips$$

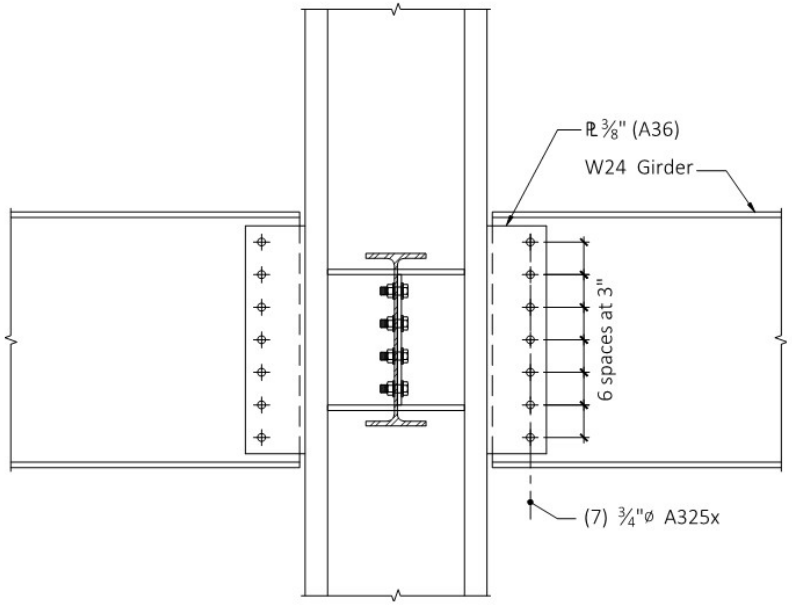
$$M_u = 1{,}998 + 217 = 2{,}215\ kip - ft$$

Compare to design values

- Shear capacity ϕVn = 306 kip
  - Girder is adequate for shear
- Flexural capacity ϕMn = 482 kip-ft
  - Demand exceeds flexural capacity
  - Either the girder will yield in flexure or the connection will yield

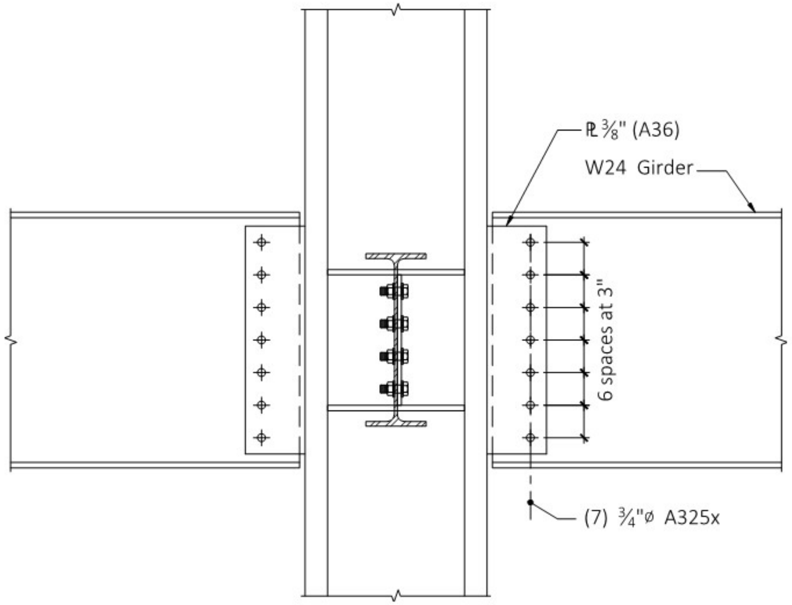
Check Connection

- Shear capacity
  ϕVn = 133 kip
  - Short slotted holes req'd
- Flexural capacity
  ϕMn = 63 kip-ft
- Connection will yield in flexure



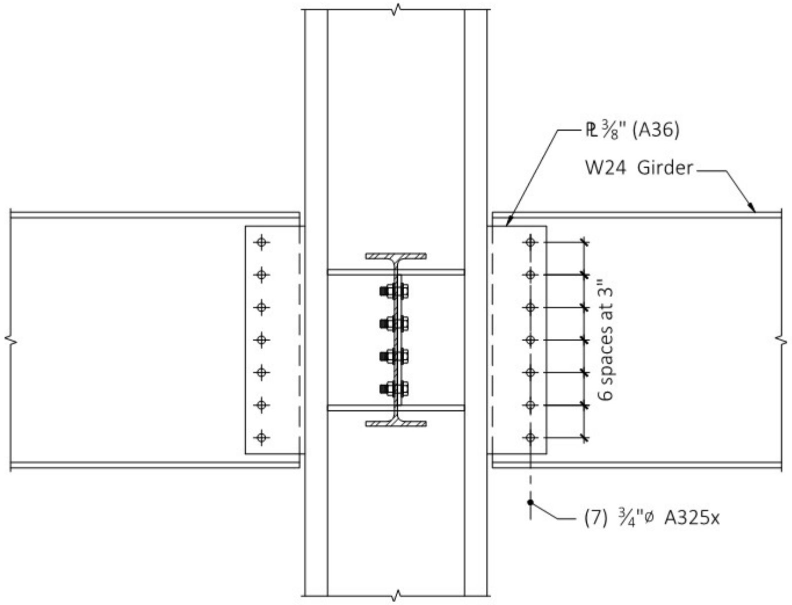
Check Connection

- Check plate thickness to ensure ductile behavior (plate yielding)
- Check permissible rotation per ASCE 41



Check Connection

- Maximum plate thickness is acceptable
  - But *not* if it were 50 ksi steel

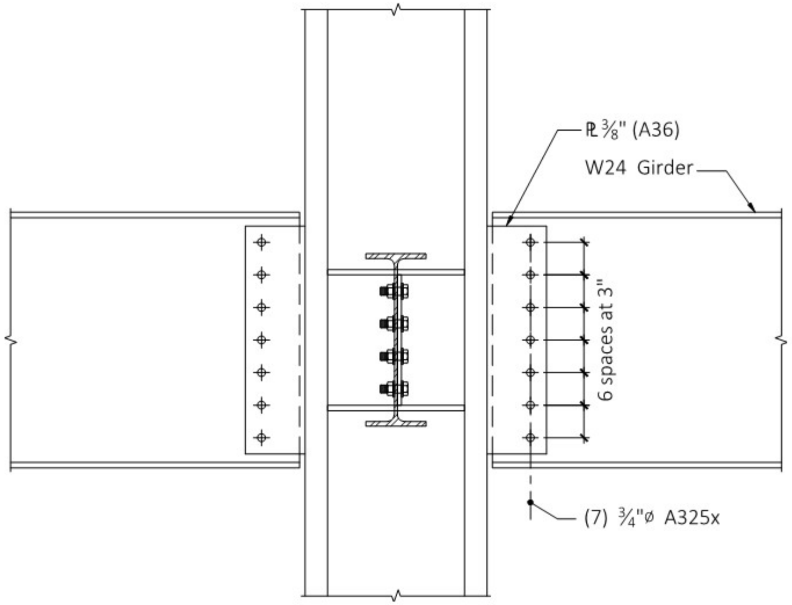


$$t_{max} = \frac{6F_v(A_bC')}{0.90F_yd^2} = 0.43\ in.$$

Check Connection

- Check plastic rotation to provide collapse resistance per ASCE 41
  - Acceptable

$$\theta_{CP} = 0.15 - 0.0036 d_{bg}$$
$$= 0.085\, rad$$



Conclusions

- The building can sustain the differential settlements imposed by the liquefaction
- Acceptable to use shallow foundations
  - This assessment was primarily concerned with connection ductility
  - Other systems may have different concerns
    - Concrete systems must be checked for shear and remain essentially elastic
    - Non SMF steel moment connections may not have sufficient ductility
    - Must consider load reversals causing the need for added bracing

Foundations on Liquefiable Soil

- Effects of Liquefaction on Structures
  - Differential settlement leading to damage in the superstructure
  - Lateral spreading leading to damage to foundations and superstructure

Performance Goals of Provisions

- Structures are intended to resist collapse due the liquefaction effects
  - MCE_G ground motions.
- Distinct from remainder of Provisions
  - 2/3 of MCE_R ground motions

Nonlinear Behavior

- Settlement or lateral spreading due to liquefaction may cause nonlinear behavior
- This behavior must be addressed where applicable
- Complete nonlinear analysis not required, provided that nonlinear behavior is assessed appropriately.

Liquefaction

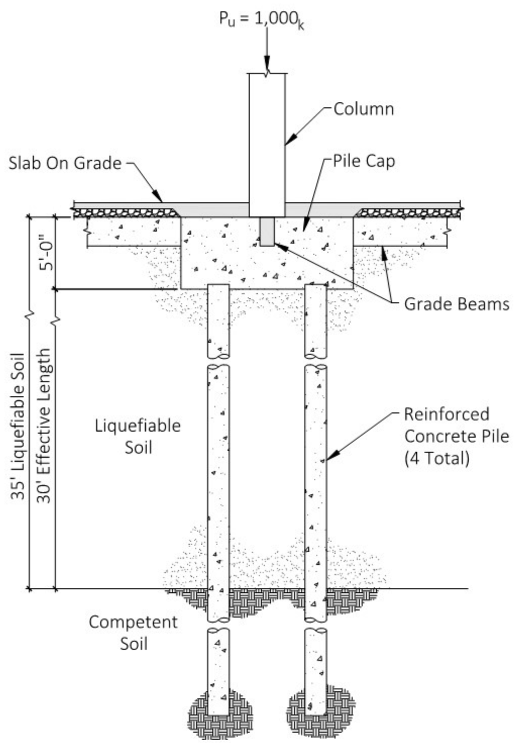
- Occurs in loose, saturated sands and silts with poor drainage
  – Differential settlement

Lateral Spreading

- Can occur when liquefaction occurs adjacent to a scarp, channel or riverbank.

Picture by NISEE-PEER, Univ. of California, Berkeley. Used by permission.

Example: Lateral Spreading



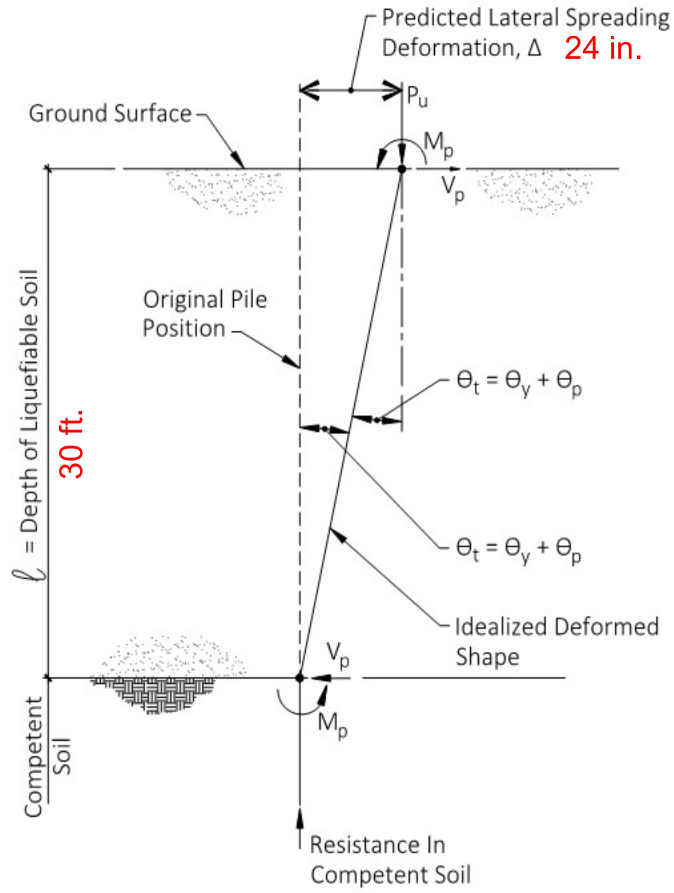
- Structure is on deep foundations
- Lateral displacements exceed Table 12.13-2 limits

Structural Requirements

- Analysis is required to demonstrate acceptable pile behavior
- Nonlinear behavior is permitted
  - No loss of gravity support permitted
  - Residual member strength at least 2/3 of undamaged nominal strength
  - Prescriptive detailing for ductility
  - Nominal shear capacity must not be exceeded

Pile Foundation

- Lateral displacement 24 in
  - – Exceeds 18 in limit
- Occurs over depth of 30 ft.
- Factored loads per pile 250k



Pile Diameter

- Improved flexural ductility at low axial loads
- Choose diameter, such that $\frac{250k}{A_g f_c'} < 0.1$;

$$Pile\ Diameter \geq 2\sqrt{\frac{250k}{\pi(5ksi)(0.1)}} = 25.2\ in$$

- Choose 30 in. diameter

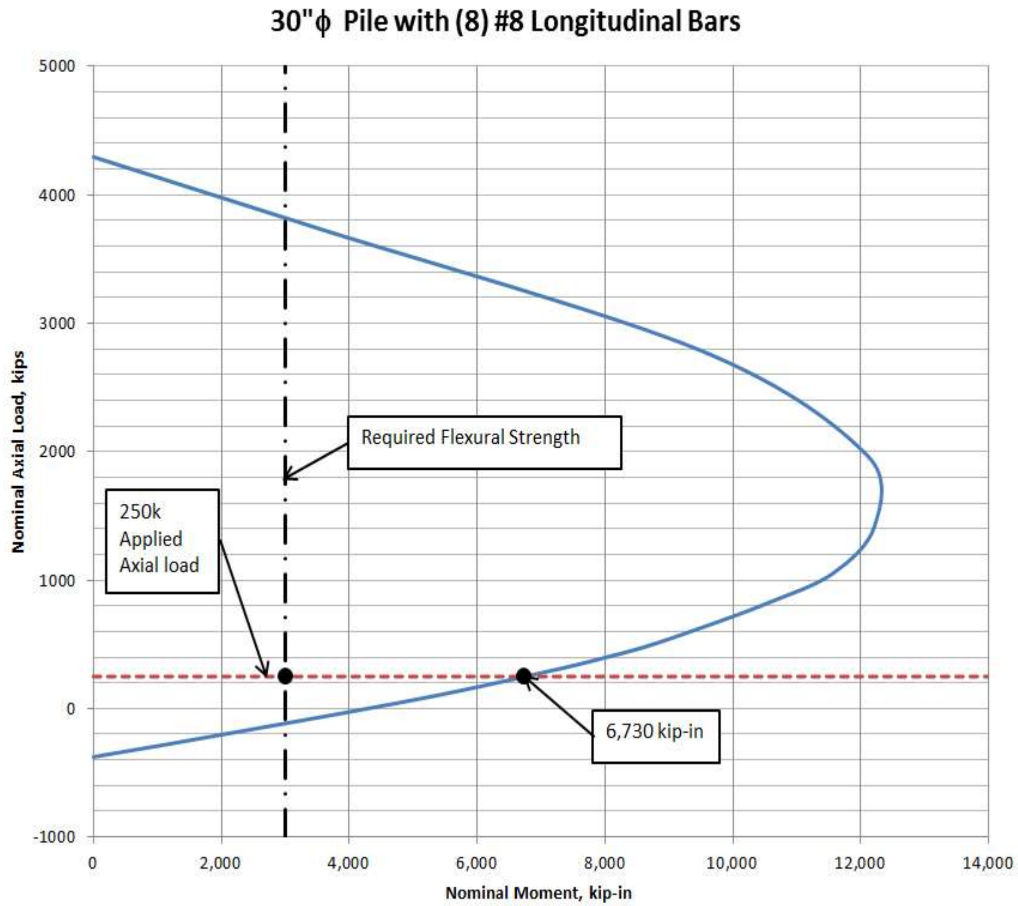
Flexural Demand

- Consider fixed-fixed behavior between the bottom of the cap and the top of the competent soil.
  - Half of P-Delta moment occurs at two locations

$$M_u = \frac{P_u \Delta}{2} = \frac{250k(24in)}{2}$$
$$= 3{,}000\ kip - in$$

Axial-Moment Interaction

- Consider fairly light reinforcing and concrete strength of 5ksi



Rotation and Curvature

- Rotation is computed from displacement and effective length

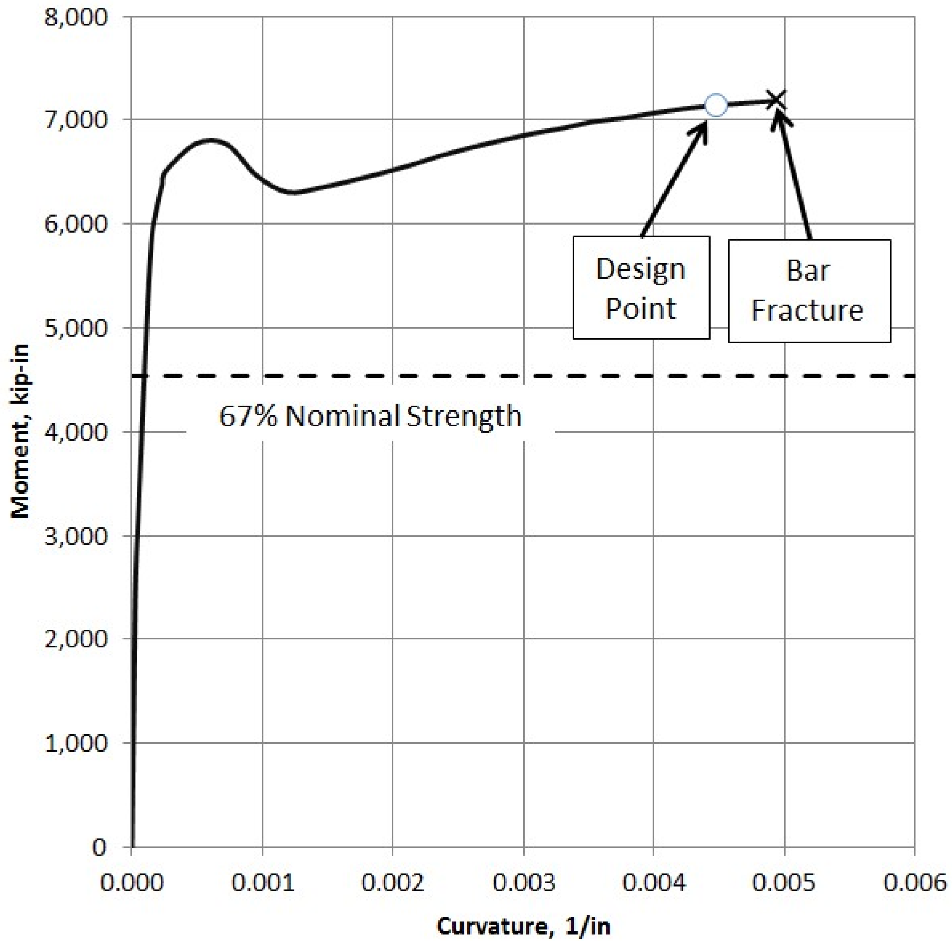
$$\theta_t = \frac{24in}{360in} = 0.0667\ rad.$$

- Curvature depends on presumed hinge length: ½ pile diameter

$$\varphi = \frac{\theta_t}{l_p} = \frac{0.0667rad}{15in} = 0.0044\ in^{-1}$$

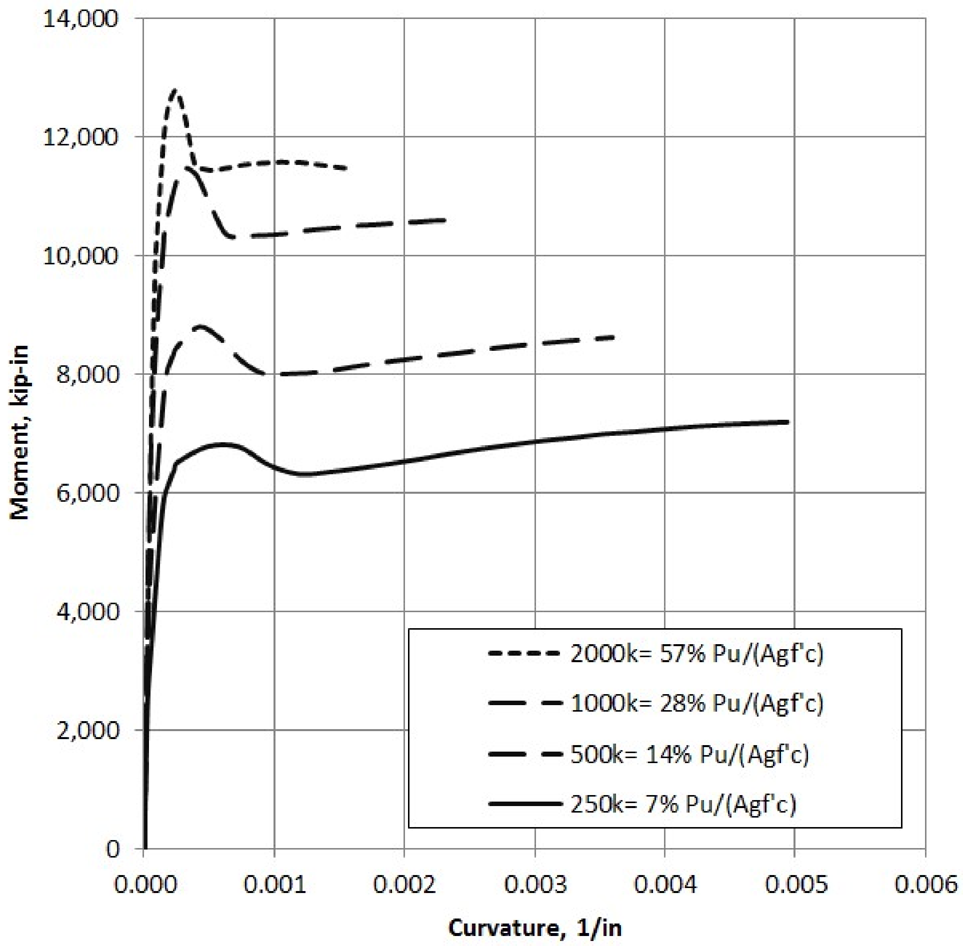
Moment-Curvature

- Strength at imposed curvature remains above 67% limit
- Imposed curvature is less than limit due to bar fracture



Moment-Curvature

- Consider whether smaller pile diameters would be acceptable
- Smaller diameters result in earlier bar fracture
  - Not acceptable
  - Use 30” initial design



Detailing for Ductility

- ACI 318-14, Sections 18.7.5.2 through .4
- Spiral spacing
  - ¼ member dimension (7.5 in.)
  - Six time bar diameter (6 in.)
  - 6 in.
- Volumetric ratio (larger of the following):

$$\rho_s = 0.12\, f_c' / f_{yt} = 0.01$$

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f_c'}{f_{yt}} = 0.021$$

Detailing for Ductility

- Choose #5 spirals and compute required spacing

$$\rho_s = V_s/V_{ch} = A_s \pi d_{ch}/s\pi\left(\frac{d_c}{2}\right)^2 = \frac{4A_s}{d_{ch}s}$$

$$s = \frac{4A_s}{d_{ch}\rho_s} = \frac{4(0.31)}{(24)0.021} = 2.5\ in$$

- Use pitch of 2½ inches
- Required from the top of the pile to 7 pile diameters (17'-6") below the interface of the liquefiable soil and the competent soil below.

Shear Strength

- Compute shear demand at probable moment
  - Taken conservatively as 1.25 times nominal moment

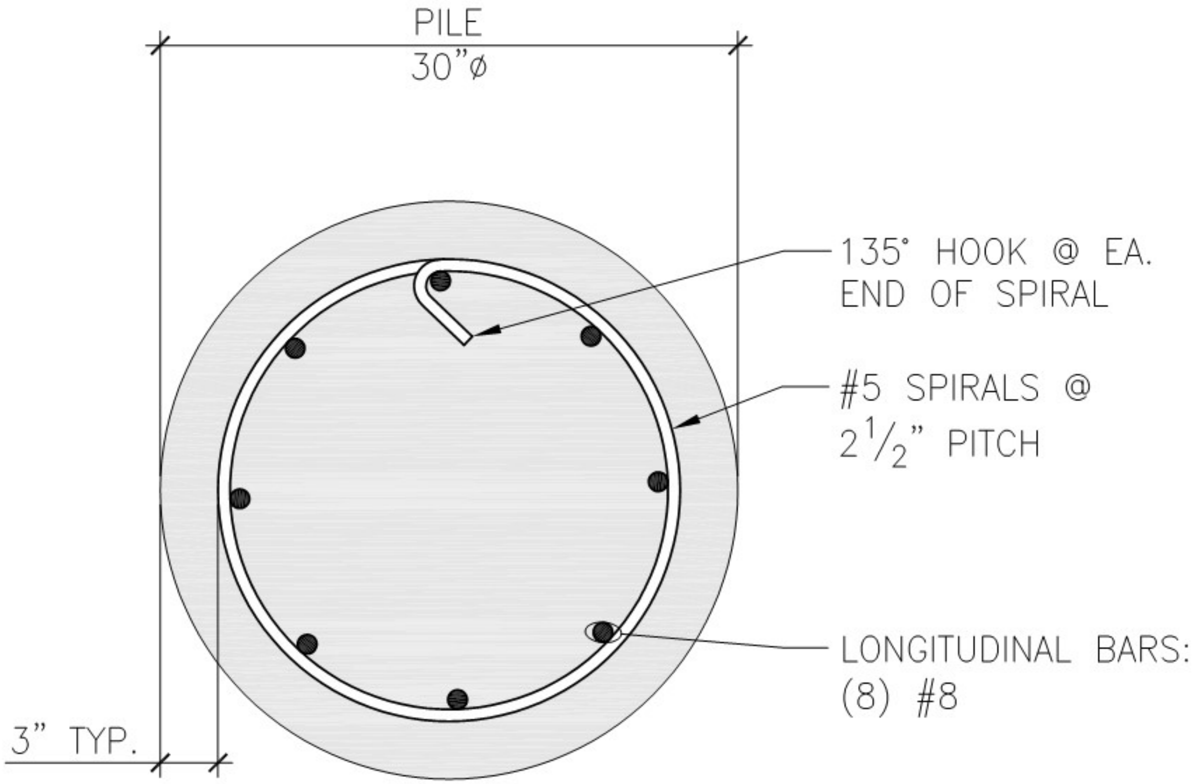
$$V_u = 2M_{pr}/l = 2(1.25 \times 6{,}730\ kip - in)/360\ in = 47\ k.$$

- Compute shear strength ignoring concrete within plastic hinge

$$\varphi V_n = (0.75)(2)(0.31)(60ksi)\ (0.8 \times 30)/2.5 = 232k$$

- Shear capacity is sufficient.

Pile Section Detail



Questions

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