FOUNDATION DESIGN

Proportioning elements for:

- **Transfer of seismic forces**
- **Strength and stiffness**
- **Shallow and deep foundations**
- **Elastic and plastic analysis**

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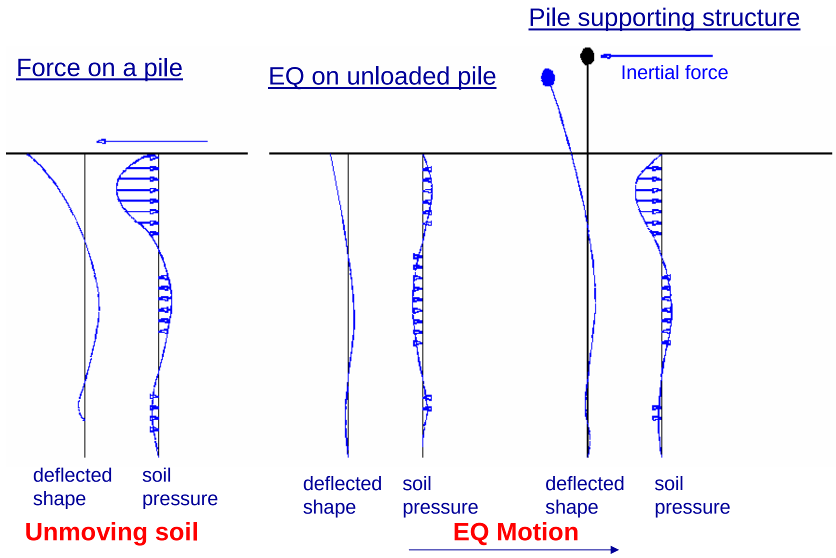
The subtitles are effectively a table of contents although the topics are not really treated in that specific order. This unit is primarily aimed at the structural engineering of foundations, not at the geotechnical engineering.

This presentation relates to example computations Chapter 4 in the FEMA 451, *NEHRP Recommended Provisions: Design Examples*.

Load Path and Transfer to Soil
Soil Pressure



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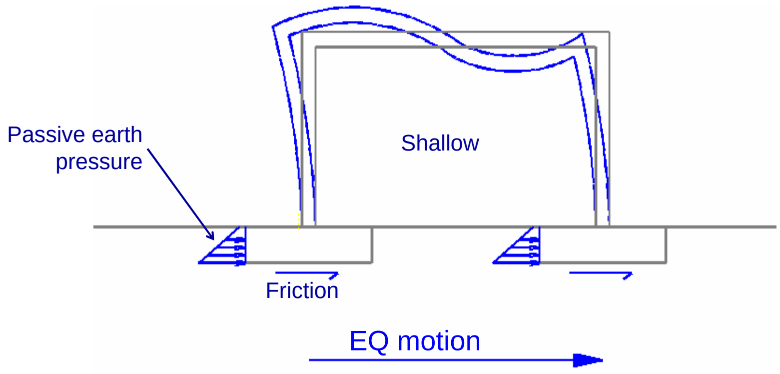
First model: soil pressures in unmoving soil caused by force at top of deep pile; most of stress resisted at top of pile; only small stresses below about twice the characteristic length of pile.

Second model: unloaded pile subject to earthquake ground motion; small stresses induced by upper levels of soil lagging behind deep motion. Note opposing directions of "push."

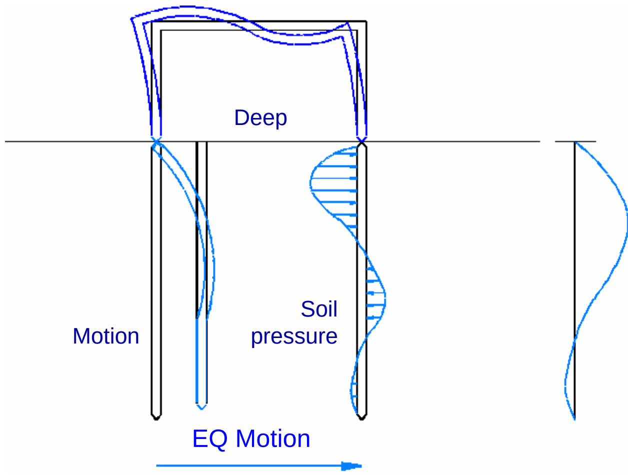
Third model: both types of force act on pile. The lag of structure induces inertial forces at top of pile similar to static force in first model; net force shape similar to static situation.

Load Path and Transfer to Soil
Soil-to-foundation Force Transfer



As building lags behind ground motion, induced inertial forces must be transferred between footing and soil. Design may consider that inertial forces are transferred as passive earth pressure on face of footing, friction on bottom of footing, or both.

Load Path and Transfer to Soil
Soil-to-foundation Force Transfer



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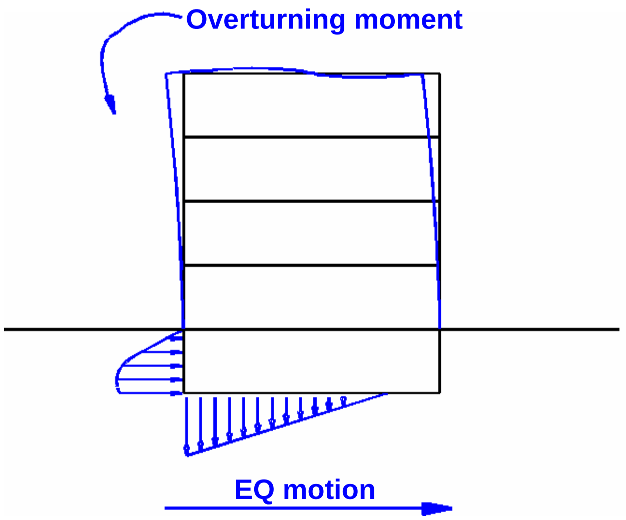
Same single story structure but now on deep pile foundation. One leg shows pile displacements; the other shows resulting earth pressures; the third diagram shows bending moment in pile. One reference that has long been used for laterally loaded piles is the Navy Design Manual 7.2, *Foundations and Earth Structures*. However, it and most other older methods are based upon assumptions of linear behavior in soil. Over the past two decades considerable progress has been made in developing design tools rooted in the strongly nonlinear behavior of soil. “LPLILE” is one widely used example that allows the user to specify soil parameters that model resistance of soil to lateral movement of piles.

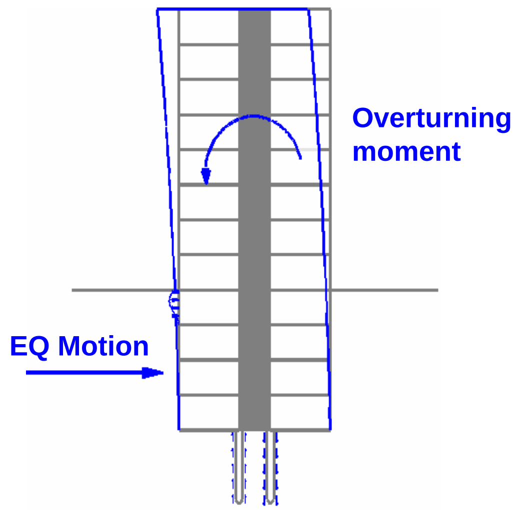
Load Path and Transfer to Soil

Vertical Pressures - Shallow



As aspect ratio of building height to width increases, overturning moment becomes significant; induced vertical forces must be transferred in addition to horizontal pressures. (Similar vertical forces in footing result from column moments not specifically related to overturning.) Slide shows overturning moment being resisted below basement of medium sized building; horizontal pressures are transferred at the basement walls.

Load Path and Transfer to Soil
Vertical Pressures - Deep



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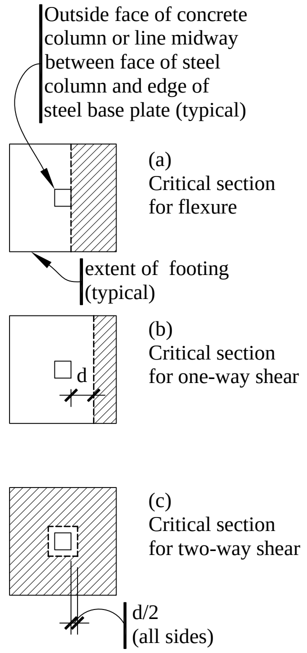
This example of tall building with shear wall continuing through deep basement shows that the horizontal and vertical forces can be resisted by different portions of foundation structure. Basement wall resists horizontal forces near ground surface; vertical forces resisted by piles at base of wall.

Reinforced Concrete Footings: Basic Design Criteria (concentrically loaded)



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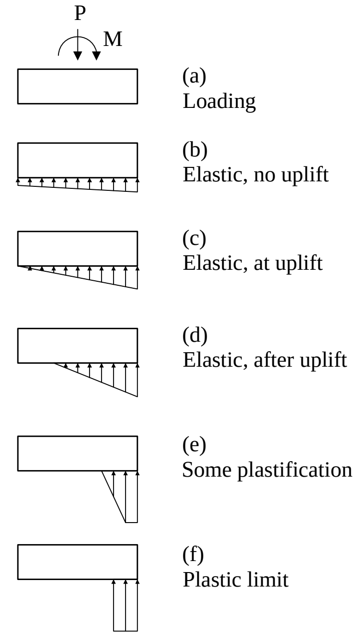
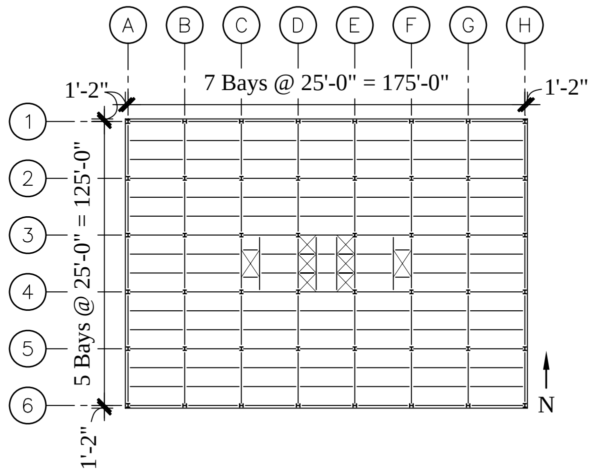
Reinforced concrete footings are proportioned according the the provisions of ACI 318, *Building Code Requirements for Structural Concrete*. It is often opined that foundations should not yield due to the high cost of foundation repair. However, nonlinear soil behavior is common in strong ground shaking, and it is traditional to design foundations for the reduced forces computed with the response modification factor, *R*, used for the superstructure. Neither the *NEHRP Recommended Provisions* nor earlier model building codes required the use of amplified forces for foundation design.

Footing Subject to Compression and Moment: Uplift Nonlinear



FEMA 356 has a good discussion of the plastic behavior of soil beneath eccentrically loaded footings. Just as for analysis of structural members, plastic analysis of a footing is simple “ by hand” but not so with a computer.

Both uplift and nonlinear behavior introduce complications in conventional analysis. Many commercially available software packages for structural analysis now handle the uplift case; a smaller set can also handle nonlinear behavior.



Example 7-story Building:

Shallow foundations designed for perimeter frame and core bracing.

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Foundation Design 14-9

Shallow Footing Examples

Soil parameters:

- Medium dense sand
- (SPT) N = 20
- Density = 120 pcf
- Friction angle = 33^{o}

Gravity load allowables

- 4000 psf, B < 20 ft
- 2000 psf, B > 40 ft

Bearing capacity (EQ)

- 2000*B* concentric sq.
- 3000*B* eccentric
- ϕ = 0.6

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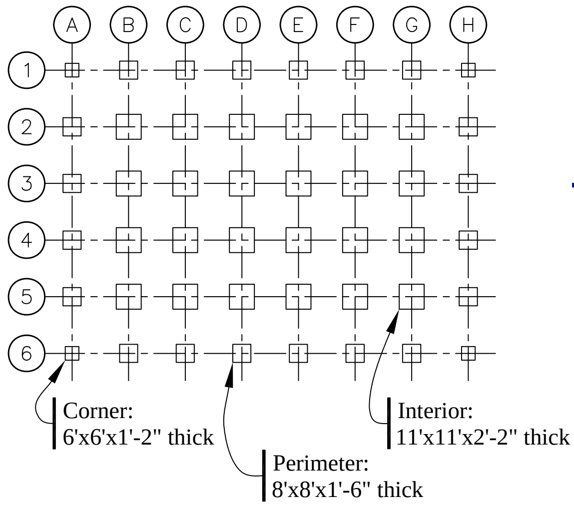
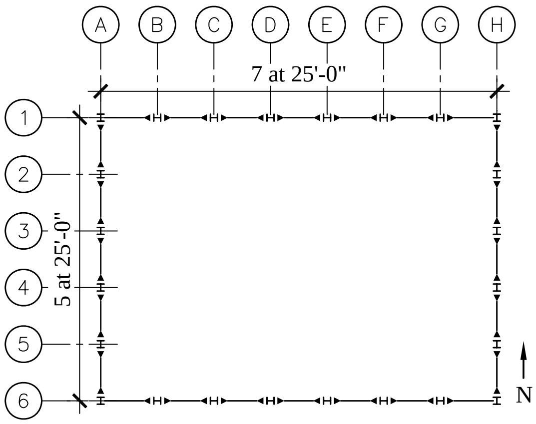
The gravity load allowables are set to control settlements. The values between 20 and 40 feet should be interpolated. The bearing capacity is the classic value from theoretical soil mechanics (normal gravity loads are checked. Refer to FEMA 356 for suggestions on resistance factors. The subject of strength design in soils is in its infancy, and many geotechnical professionals are not yet comfortable with strength design concepts.



Footings proportioned for gravity loads alone

The size of the square footing is controlled by the allowable bearing pressure at total loads, and the thickness is controlled by two-way shear at the critical section (“punching shear”).

The point of this information is primarily for later comparison with footings designed for seismic loads.



Design of Footings for Perimeter Moment Frame

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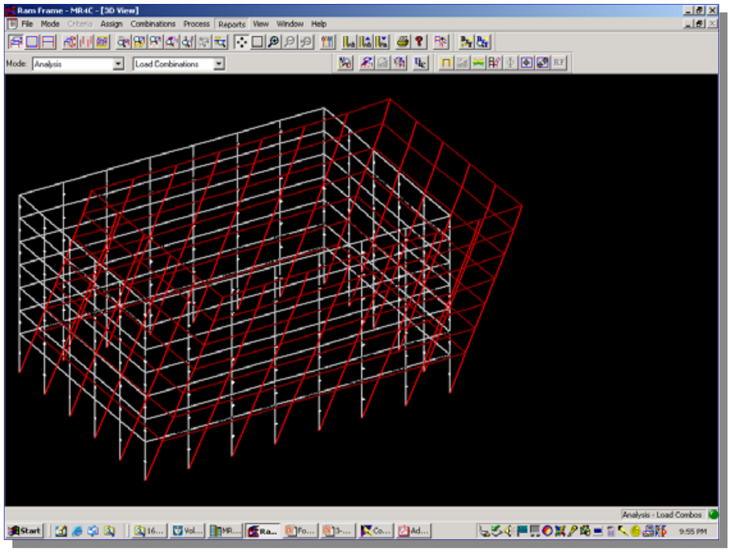
The only portion of the steel frame that resists lateral forces is at the perimeter, thus, the only footings that will be affected by the seismic load are at the perimeter.

7-Story Frame, Deformed



The image is taken from the RAM Frame analysis used to design the steel moment resisting frame for seismic loads.

Combining Loads

- Maximum downward load:
  $1.2D + 0.5L + E$
- Minimum downward load:
  $0.9D + E$
- Definition of seismic load effect E:
  $E = \rho_1 Q_{E1} + 0.3\ \rho_2 Q_{E2} +/- 0.2\ S_{DS} D$
  $\rho_x = 1.08 \quad \rho_y = 1.11$ and $S_{DS} = 1.0$

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Load combinations for strength-based design, which is the fundamental method for earthquake resistant design.

Greek rho is the redundancy factor. Q is the effect of horizontal seismic motions. The $0.2S_{DS}D$ is an approximation for the effect of vertical earthquake motions.

For the footings, the horizontal motions produce vertical and horizontal forces, as well as bending moments, at the base of each column. Dead and Live loads are taken to produce only vertical forces in this example.

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

Grid A-6 is at the lower left corner of the plan, and A-5 is adjacent. (Go back three slides to show the location on the plan.) Recall that the seismic reactions can be positive or negative; what is given here is for motion in the positive x and y directions. Carefully note that subscripts x and y on the load effect *E* refer to the global north-south and east-west, respectively, but the subscripts x and y on the moments at the column bases refer to the local strong and weak axes, respectively, which is just the opposite as the global directions, unfortunately.

The most significant point of this slide is that seismic uplift at A-6 exceeds the dead load by a considerable margin. It is possible to place a footing with sufficient size to resist the uplift and the overturning moment, but it is much more economical to combine one footing for the two locations. These reactions include the effects of horizontal torsion on the system. Also recall that the footing must resist horizontal forces.

Reduction of Overturning Moment

- *NEHRP Recommended 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.

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Additive Load w/ Largest Eccentricity

- At A5: P = 1.4(203.8) + 0.5(43.8) +
  0.75(0.32(-3.8) + 1.11(21.3)) = 324 k
  M_{xx} = 0.32(53.6) + 1.11(-1011.5) = -1106 k-ft
- At A6: P = 1.4(103.5) + 0.5(22.3) +
  0.75(0.32(-51.8) + 1.11(-281)) = -90.3 k
  M_{xx} = 0.32(47.7) + 1.11(-891) = -974 k-ft
- Sum M_{xx} = 12.5(-90.3-324) -1106 -974 = -7258

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None of these loads include the weight of the footing.

P is positive in compression. *M* is positive by the local right hand rule.

1.4 on dead load includes the vertical seismic acceleration, which is 0.2*D* here where S_{DS} is 1.0.

The 0.75 factor is the reduction on base overturning moment. Some authors might interpret that it also applies to the column base moments.

This is not the maximum downward load; it is the maximum ratio of moment to axial load for the additive combos.

Counteracting Load with Largest *e*

- At A-5: P = 0.7(203.8) + 0.75(0.32(-3.8) + 1.11(21.3)) = 159.5 k

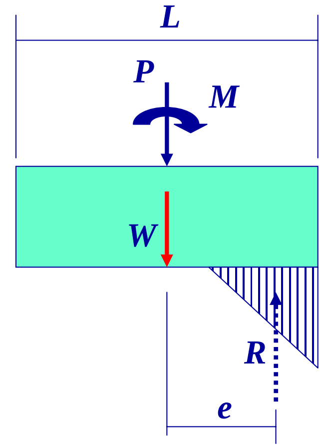
  M_{xx} = 0.32(53.6) + 1.11 (-1011.5) = -1106 k-ft
- At A-6: P = 0.7(103.5) + 0.75(0.32(-51.8) + 1.11(-281)) = -173.9 k

  M_{xx} = 0.32(47.7) + 1.11(-891) = -974 k-ft
- Sum M_{xx} = 6240 k-ft

Note that the net vertical load is upward without the weight of the footing. It so happens that this combo also gives the maximum eccentricity, when combined with the weight of footing and soil.

Elastic Response

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



Slide is drawn for the case with substantial moment, such that uplift will occur at the heel. Note that eccentricity *e* changes as *W* changes.

For our footing, *L* will exceed 25 feet by some margin, given that the two columns are 25 feet apart.

Additive Combination

Given P = 234 k, M =7258 k-ft

Try 5 foot around, thus L = 35 ft, B = 10 ft

- Minimum $W = M/(L/2) - P$ = 181 k = 517 psf

Try 2 foot soil cover & 3 foot thick footing

- W = 245 k; for additive combo use 1.2W
- $Q_{max} = (P + 1.2W)/(3(L/2 - e)B/2)$ = 9.4 ksf
- $\phi Q_n = 0.6(3)B_{min}$ = 10.1 ksf, OK by Elastic

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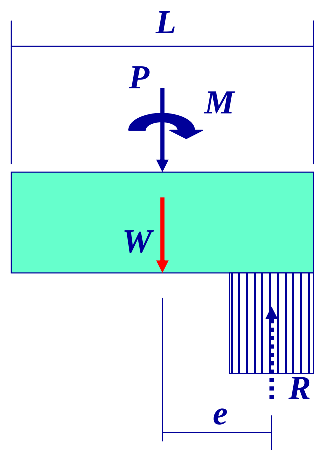
Initial approximation of W is simply to keep the resultant of earth pressure within the footing. It must be somewhat larger in order to control the bearing pressures.

Note that the load factor on W does not include the amplifier for vertical seismic acceleration; this is the author's interpretation of the *NEHRP Recommended Provisions*.

The minimum B used to find the nominal bearing capacity is found by comparing the width of the footing and the half length of the loaded area. The half length is used because the soil pressure is not uniform.

Plastic Response

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



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Counteracting Case

Given P = -14.4 k; M = 6240

Check prior trial; W = 245 k (use 0.9W)

- e = 6240/(220.5 – 14.4) = 30.3 > 35/2 NG

New trial: L = 40 ft, 5 ft thick

- W = 400 k; e = 18.0 ft; plastic Q_{max}= 8.6 ksf
- ϕQ_n = 0.6(3)4 = 7.2 ksf, close
- Solution is to add 5 k, then e = 17.8 ft and $Q_{max} = \phi Q_n$ = 7.9 ksf

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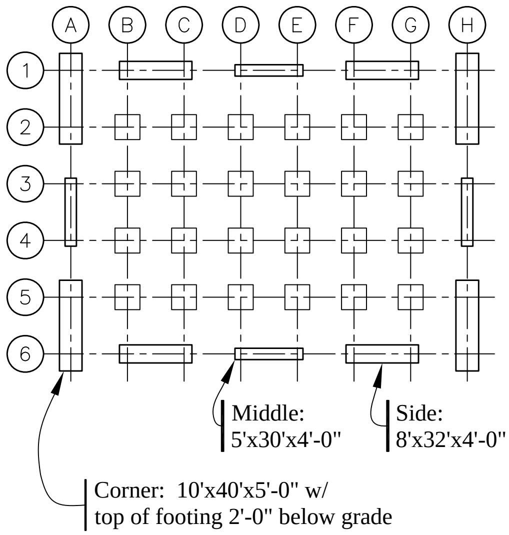
Note how much larger the footing must be for the counteracting case. Also, it would have been even larger if the elastic solution were used in lieu of the plastic solution.

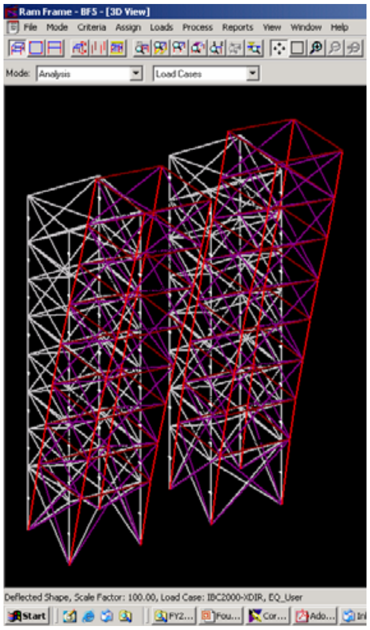
Additional Checks

- 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: $H_{max} < \phi\mu(P+W)$
  in this case friction exceeds demand; passive could also be used.

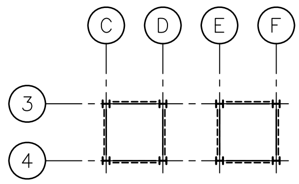


Results for all SRS Footings

Given the combined footing strategy, footing sizes are more strongly influenced by the uplift on columns at the ends of frames than by the moments transmitted by the columns. Note that a complete perimeter grade beam would be a very feasible solution for this project, especially in cold climates where a continuous perimeter wall for frost control is necessary. A 4 ft by 4 ft continuous grade beam would be sufficient.



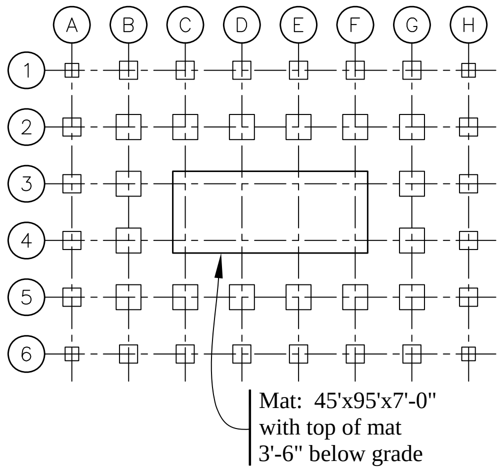
Design of Footings for Core-braced 7-story Building



25 foot square bays at center of building

The screen capture is from the RAM Frame analysis of the structure, and the small plan is based on the same grids used for the 7 story moment frame. The braced frames appear to be 8 stories high, because there is a small penthouse over the core.

Solution for Central Mat



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

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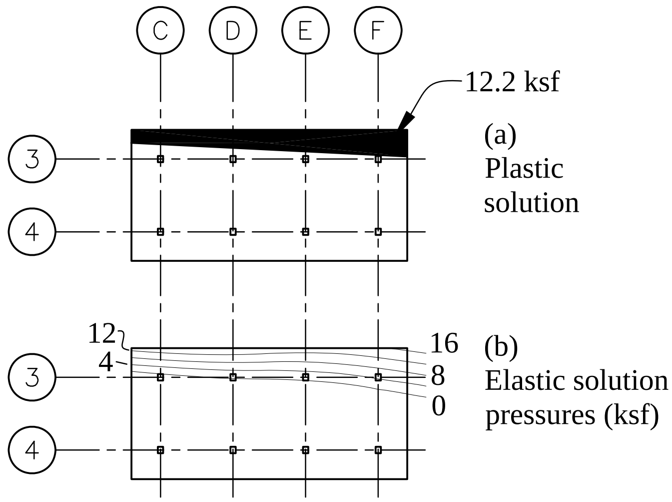
The fundamental method is the same as used in the previous example: Determine the total applied vertical and horizontal loads and the moments. The complicating factor here is that the bending is significant about two axes simultaneously. Elastic solutions can be found from software that has the capacity for compression-only springs; RISA 3D was used in this case. Plastic solutions typically need to be done "by hand" but spreadsheets are a great asset for the successive trial nature of the solution.

Bearing Pressure Solution



Plastic solution is satisfactory; elastic is not; see linked file for more detail.

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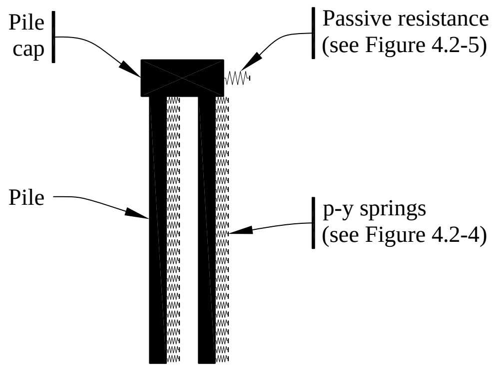
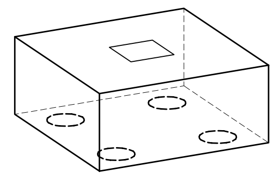
Slide shows the results from "hand" analysis for plastic distribution and for RISA 3D elastic solution. See Chapter 4 of FEMA 451 for more detail on the solution as well as the design of the footing cross section for moment and shear.

Pile/Pier Foundations





View of cap with column above and piles below.

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Pile/Pier Foundations

Pile Stiffness:

- Short (rigid)
- Intermediate
- Long

Cap influence

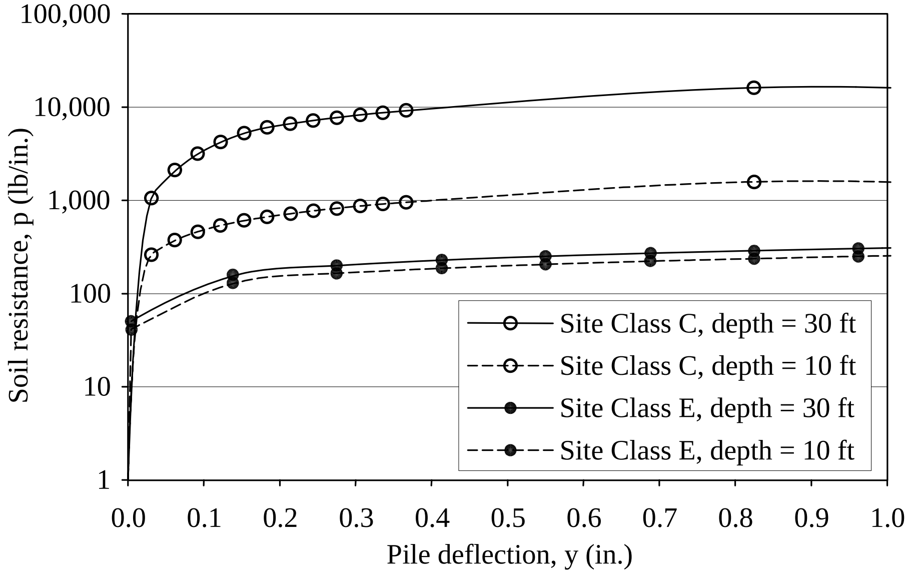
Group action

Soil Stiffness

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

Most pile analysis for lateral loads is performed assuming linear response in the pile itself, although it is now common to consider nonlinear soil response. Some “by-hand” plastic techniques do make use of the classic pile stiffness idealizations.

Sample *p-y* Curves



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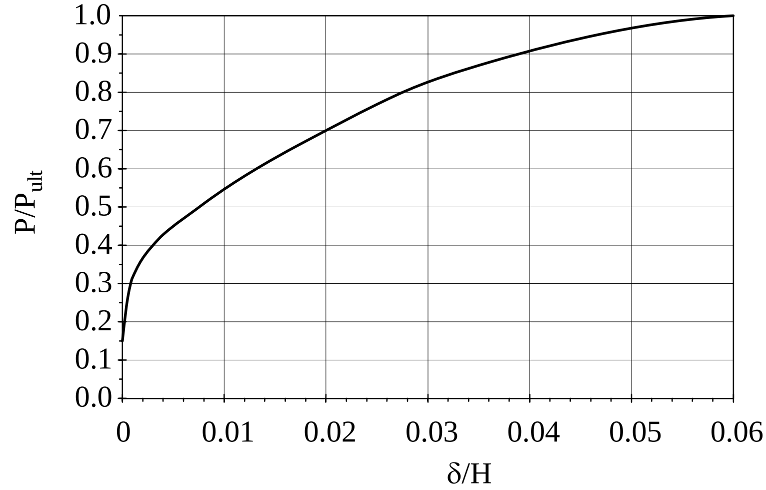
Note the logarithmic scale on the vertical axis.

Passive Pressure



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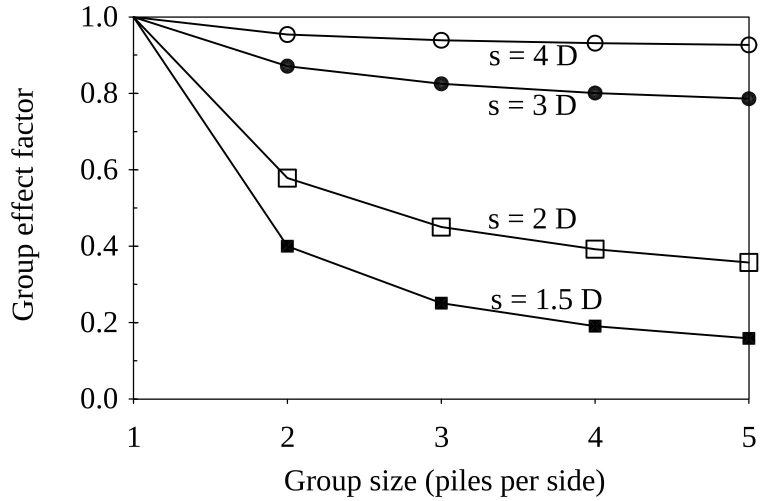
Foundation Design 14-31

This passive pressure mobilization is useful for inclusion of the pile cap. It is from FEMA 356.

Group Effect



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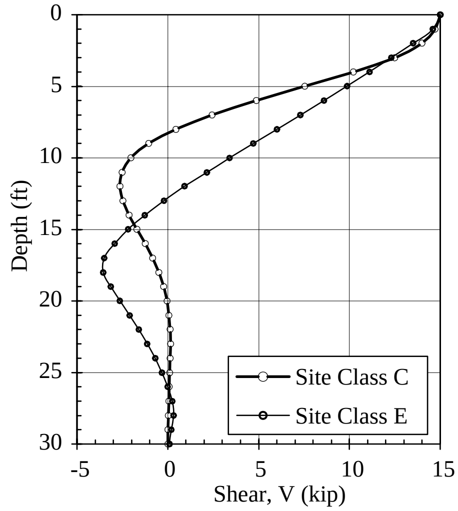
Group effect factors computed from PoLam, et al, *Modeling of Pile Footings and Drilled Shafts for Seismic Design*, MCEER-98-0018.

Pile Shear: Two Soil

Stiffnesses



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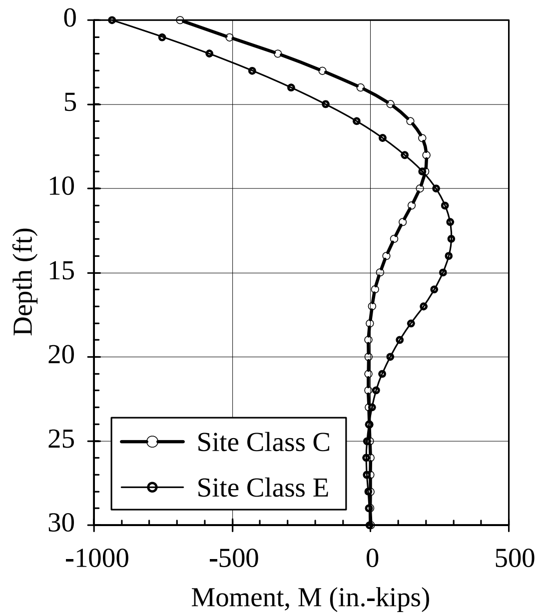
Note that the shear forces in the pile (as well as deformations and bending moments) carries to greater depths in soft soils than in firm soils. Pile (or pier) foundations are often used in stiff soils to control settlement of heavy structures or heave of expansive soils.



Pile Moment vs Depth

FEMA

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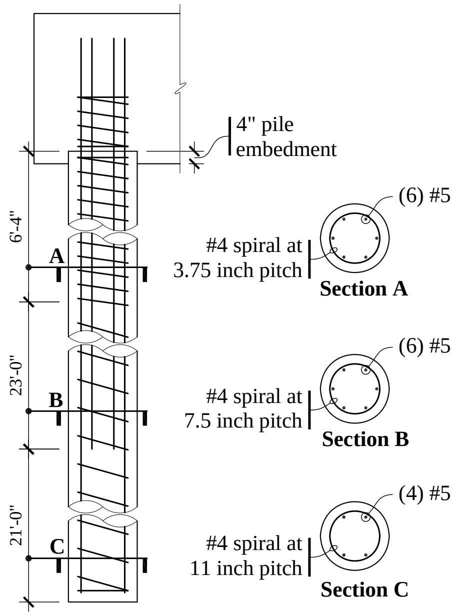
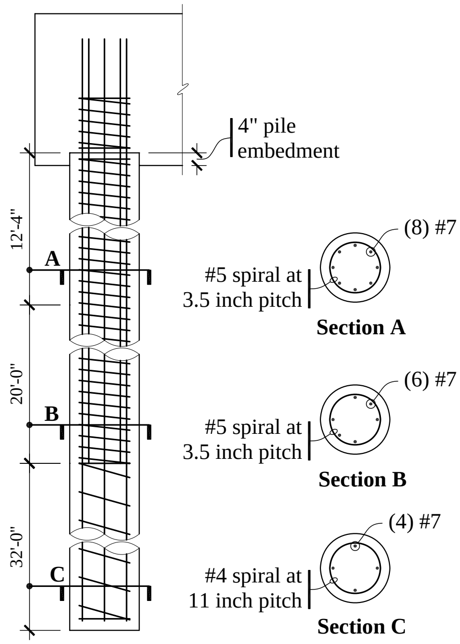
Foundation Design 14-34

See Chapter 4 of FEMA 451.



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)

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The drawing shows one of the piles with detail of reinforcement. See Chapter 4 of FEMA 451.

Other Topics for Pile Foundations

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

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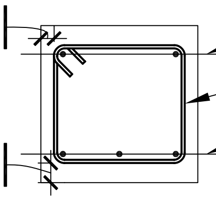
Tie Between Pile Caps



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

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Required in the higher Seismic Design Categories for softer soils. It is designed for "pure" axial force. Fundamental objective is to prevent relative lateral displacement between column bases. It "fixes" the column bases for translation, but it is not intended to restrain rotation at the column bases.