NCSEA Structural Engineering Exam Review Course Lateral Forces Review

Geotechnical, Seismic Earth Pressure, and Foundations

Sections with Lateral Geotechnical Topics

Using NCEES Outline of Material

I. Analysis of structures
- A. Lateral forces
  - 4. Dynamic (seismic) earth pressure
- B. Lateral force distribution
  - 2 and 3. Seismic design categories
  - IBC seismic site classes

II. Design and detailing of
- H. Foundations and retaining structures
  - 1. Spread footings, 2. Piles, 3. Drilled shafts/piers

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Seismic lateral earth pressure determination is a soil-structure interaction problem
- Depends on the soil and the wall
- Flexible walls have to move enough to develop active and passive earth pressures
- Rigid walls are assumed to limit lateral deflections such that the *at-rest* earth pressure is maintained

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Yielding/flexible walls include sheet pile walls, cantilevered retaining walls, gravity retaining walls, mechanically stabilized earth (MSE) walls
- Rigid walls include building walls restrained by floors and foundations, walls of buried reinforced concrete box structures, U-walls (two L-shaped retaining walls with a common footing)
- U-walls are rigid if they are designed and constructed to be rigid. They could also be flexible.

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Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Flexible walls—use Mononobe-Okabe method to determine the lateral seismic earth pressure
- Rigid walls—use the Wood's method
- Complex cases of soil-structure interaction can use numerical methods: equivalent-linear codes or fully nonlinear codes.
- See NCHRP Report 611—seismic wall design guidance. Two volumes on internet and Volume 2 has example problems solved.

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Review of NCHRP 611, AASHTO Seismic Design documents, SEA of California white papers indicate retaining wall seismic design is a complex topic with many approaches
- What seismic acceleration should be used for retaining wall design?
  - Depending on the structure's location, importance, and design life, a site peak ground acceleration, PGA, is selected. PGA is not a good measure of ground motion felt by retaining structures. Values ranging from 1/3 to 2/3 PGA, typically 1/2 PGA used for ordinary walls. See 2012 AASHTO page 11-26, $k_h = A_s/2$, where δ=1 in. to 2 in.

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Mononobe-Okake Method
- Backfill soil assumed to be dry to moist, unsaturated cohesionless (sandy) soil
- Forces acting on retaining wall assumed to be static (from Coulomb equation) plus horizontal and vertical pseudostatic components
- Pseudostatic accelerations: $a_h = k_h g$ and $a_v = k_v g$, where g is gravitational acceleration, 32.2 ft/sec^2

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Mononobe-Okabe total active force from both static and dynamic components equals

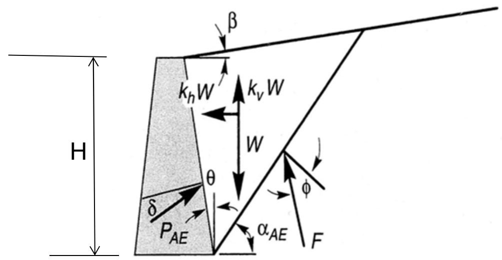
  $P_{AE} = ½ K_{AE} \gamma H^2 (1 – k_v)$, AASHTO $k_v = 0$

  where the dynamic earth pressure coefficient, $K_{AE} = [\cos^2 (\Phi – \theta – \psi)] / [\cos\psi \cos^2\theta \cos(\delta+\theta+\psi) (\mu)]$

  where $\mu = \{ 1 + [(\sin(\delta + \Phi) \sin(\Phi – \beta – \psi)) / (\cos(\delta + \theta + \psi) \cos(\beta – \theta)]^{0.5}\}^2$

  – Terms above are defined in the figure below from Steve Kramer's book, *Earthquake Engineering,* also see 2012 AASHTO manual.

Section I – A.4 – Dynamic (Seismic) Earth Pressure



δ is about 2/3 Φ

Earthquake Engineering, Kramer

Φ = backfill soil's angle of internal friction in degrees, determined by drained shear strength testing

$\Psi = \tan^{-1}[k_h/(1-k_v)]$

$\alpha_{AE} = 45^o + \Phi/2$,

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- $P_{AE} = P_A$ (static) + ΔP_{AE} (seismic)
- $P_A = ½ \gamma H^2 K_A$ where K_A is the Coulomb Active Earth Pressure Coefficient (static pressure)
- To find the seismic component of lateral earth pressure, $\Delta P_{AE} = P_{AE} - P_A$
- Static component acts at ⅓ H above the base, dynamic component acts at 0.6 H above the base, total lateral force P_{AE} acts at h above the base h = $[P_A(H/3) + \Delta P_{AE}(0.6H)] / P_{AE}$
- 2012 AASHTO pg 11–29, P_{AE} at h= H/3 to H/2

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Mononobe-Okabe total passive force from both static and dynamic components equals

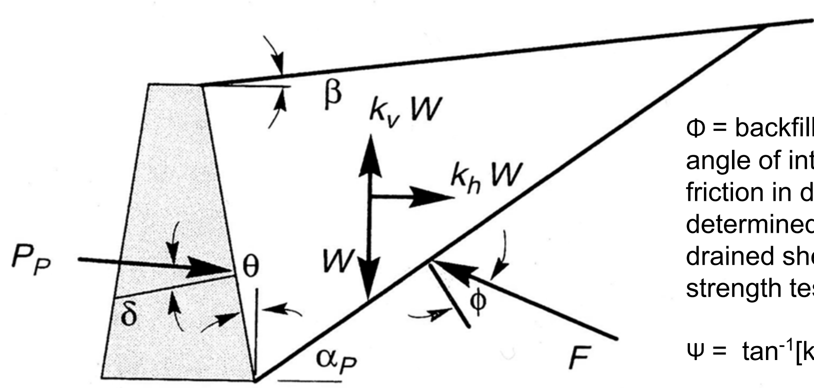
  $P_{PE} = ½\ K_{PE}\ \gamma\ H^2\ (1 – k_v)$

  where the dynamic earth pressure coefficient, $K_{PE} = [\cos^2 (\Phi + \theta – \psi)] / [\cos\psi \cos^2\theta \cos(\delta-\theta+\psi)\ (\mu)]$

  where $\mu = \{ 1 – [(\sin(\delta + \Phi) \sin(\Phi + \beta – \psi)) / (\cos(\delta – \theta + \psi) \cos(\beta – \theta)]^{0.5}\}^2$

  – Terms above are defined in the figure below from Steve Kramer's book, *Earthquake Engineering,* 2012 AAHSTO page 11–31 says do not use M-O but use static K_p for embedment depths less than 5 feet.

Section I – A.4 – Dynamic (Seismic) Earth Pressure



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$\alpha_{AE} = 45^o – \Phi/2$,

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Note that in the active case, the dynamic component adds to the driving earth pressure forces, while for the passive case the dynamic component subtracts from the passive resistive earth pressure
- $P_{PE} = P_p - \Delta P_{PE}$
- $\Delta P_{PE} = P_p - P_{PE}$
- As an extension of the Coulomb method, the M-O analysis has all of the limitations of pseudostatic analysis plus…

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Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Limitations of Mononobe-Okabe analyses
  - Over-predicts passive forces for $\delta > \Phi/2$
  - Active forces appear to approach infinity when the slope angle β approaches the angle of internal friction Φ. This is because a landslide is eminent. A slope stability program is required to solve such cases. The retaining wall has to support the base of the landslide.
  - Cannot use M-O for clayey backfill or any fill that derives all or part of its strength from cohesion, c'
  - Doesn't work well for dense, highly consolidated granular soils, or layered, nonhomogeneous soils

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Section I – A.4 – Dynamic (Seismic) Earth Pressure

- Wood's analysis is commonly used for rigid walls
- Wood's is an elastic method that assumes soil-structure interaction strains in the elastic range
- Wood's original solution was a computer generated solution based on dynamic analyses
- Computer analyses were converted into dimensionless "thrust" and "moment" factors, F_p and F_m, given peak acceleration, soil density, and Poisson's ratio

Section I – A.4 – Dynamic (Seismic) Earth Pressure

- John H. Wood's Analysis procedure (1973) is included in his California Institute of Technology Ph.D. dissertation
- Wood's dissertation is cataloged as EERL Report 73-05
- To find figures for use with Wood's method, see Kramer's book or you can download a copy of his dissertation from the Earthquake Engineering Research Library (EERL) website.

Section I – A.4 – Dynamic (Seismic) Earth Pressure – Example Questions

1. What lateral earth pressure equation is used to develop the Mononobe-Okabe seismic analysis?
   A. The Rankine Equation
   B. The Boussinesq Equation
   C. The Coulomb Equation
   D. None of the above

Section I – A.4 – Dynamic (Seismic) Earth Pressure – Example Questions

Answer to #1 – (C) The Coulomb Equation. The way to tell Coulomb from Rankine is the wall to soil friction angle δ is used only in Coulomb Equation.

2. What soil backfill lateral earth pressure coefficient is compatible with the Wood's analysis?
   A. K_o
   B. K_a
   C. K_p
   D. C_c

Section I – A.4 – Dynamic (Seismic) Earth Pressure – Example Questions

Answer to #2 – (A), K_o "at rest" earth pressure coefficient. Wood's method is for a rigid wall.

3. What method of seismic earth pressure analysis is used for saturated, soft clay backfill soil?
   A. Mononobe-Okabe
   B. Wood's
   C. Richter
   D. None of the above

Section I – A.4 – Dynamic (Seismic) Earth Pressure – Example Questions

Answer to #3 – (D), None of the above. The Mononobe-Okabe and Wood's methods assume a moist to dry, granular soil with shear strength characterized as a frictional material (i.e., with a Φ). Soft clay backfill requires analysis of the state of stresses using a slope stability analysis program, or a finite element analysis program.

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Section I – B.2 – Seismic Design Categories, Site Class

Site Class	Site Profile Name	Soil Shear Wave Velocity, v_s (ft/sec)	Standard Penetration Resistance, $\bar{N}$ or Nch	Undrained Shear Strength, $\bar{S}_u$ (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	NA	NA
B	Rock	$2{,}500 < \bar{v}_s \le 5{,}000$	NA	NA
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \le 2{,}500$	> 50	> 2,000 psf
D	Stiff soil	$600 < \bar{v}_s \le 1{,}200$	15 to 50	1,000 to 2,000 psf
E	Soft clay soil	$\bar{v}_s \le 600$	< 15	< 1,000 psf
		Any profile with more than 10 ft of soil having the following characteristics: • Plasticity index PI > 20 • Moisture content $w \ge 40\%$, and • Undrained shear strength $S_u < 500$ psf		
F	Soil requires site response analysis	Liquefiable soils, peat, high plasticity clay		

Section I – B.2 – Seismic Design Categories, Site Class

- From study of earthquakes, it was apparent that soft, loose soils amplify ground motions
- Medium rock, Site Class B was established as the standard, with amplifications from its motion applied to site classes C, D, E, and F
- Site Class is based on the upper 100 feet (30 m) of soil and rock from the base of the building
- Soil shear wave velocity, standard penetration blow count, and undrained shear strength are allowed by code for S.C. determination, BUT

Section I – B.2 – Seismic Design Categories, Site Class

- v_s, Soil Shear Wave Velocity is best. Why?
- Stiffness of subsurface materials controls the velocity and energy transferred from the fault rupture to the ground surface
- Shear modulus G is the best soil parameter for characterizing subsurface stiffness
- Shear wave velocity is directly correlated to G
- To find weighted average v_s for upper 100 ft for site class use: $v_s = (100\text{ ft}) / \sum(d_i / v_{si})$—this is <u>not</u> the average, it is technically the "harmonic mean."

Section I – B.2 – Seismic Design Categories, Site Class

- Note: Site Class E can be achieved in two ways
  1. v_s < 600 feet per second
  2. If the 10 feet of soil directly beneath the building is a soft, wet, plastic, low shear strength clay. This is defined by a plasticity index, PI > 20; a water content, w ≥ 40%; and an undrained shear strength, s_u < 500 pounds per square foot.
- Site Class F soils are very soft and loose, and defined as soil with liquefaction potential, peat (organic soil, P_t), and high plasticity clay (liquid limit, LL ≥ 50).
  – Site Class F soils require a site response analysis

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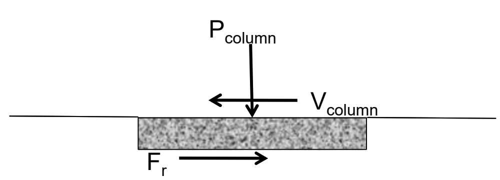
Section II – H.1 – Spread Footings

- Lateral forces applied to a structure are eventually transferred to its foundation
- When building or bridge foundations are spread footings they resist lateral forces in three ways:
  1. Frictional sliding resistance on the bottom of footing
  2. Passive earth pressure resistance on the footing end
  3. An overturning moment superimposed on the footing's vertical bearing stress
- Consider sliding resistance on footing bottom

Section II – H.1 – Spread Footings

- Sliding resistance, $F_r = P_{column} (⅔ \tan \Phi) > V_{column}$
- Sliding resistance of concrete poured on bearing soil is reduced by the factor ⅔. If the concrete bottom is confirmed to be rough, this factor could be increased to 0.8.



Section II – H.1 – Spread Footings

- The figure showing $F_r > V_{column}$ looks pretty simple
- The problem is determining what column load is present when the lateral force V_{column} is acting
- Most codes use a percentage of the dead load to resist lateral shears (i.e., 80 or 90%)
- Some load cases have uplift on certain columns
- All load combinations should be checked before counting on bottom of footing sliding resistance

Section II – H.1 – Spread Footings

- Using passive soil resistance on the end of a footing to resist lateral column shears has two primary problems
  1. To generate full passive soil resistance, the footing has to develop significant, often excessive, lateral movement.
  2. Depending on location of the footing, soil in front of the footing may have to be excavated at some future date. If lateral column shear forces occur during a period when lateral passive soil has been removed, there will be a problem.

Section II – H.1 – Spread Footings

- To resolve the first issue, passive soil resisting force on the end of a footing is calculated as $P_{passive} = (½\ \gamma\ H^2\ Kp)\ (W)$
- Where H is the footing depth
- W is the footing width in the same units as H
- γ is the moist unit weight of the soil adjacent to the end of the column footing

Section II – H.1 – Spread Footings

- Passive earth pressure coefficients can be unrealistically large for dense sandy soils
- To avoid this: do not use an internal friction angle, $\Phi > 36^o$ to calculate K_p or δ
- To limit lateral footing deflection that depends on passive earth pressure to resist sliding, divide the computed $P_{passive}$ force by a factor of safety of 2 to 3

Section II – H.1 – Spread Footings

- Lateral forces resisted by a spread footing can result in an overturning moment superimposed on the footing's vertical bearing stress
- So long as tension soil stresses are not generated when P/A ± Mc/I is applied to the footing everything is fine
- If soil tension stresses are indicated, the effective size of the footing bearing area is reduced to match compression stress area and the resulting increased stresses are computed

Section II – H.1 – Spread Footings

- Use of P/A ± Mc/I results in a trapezoidal soil pressure distribution on the bottom of footing
- FHWA design procedures allow the designer to calculate a modified uniform pressure distribution by use of the Meyerhof Method
- Given a footing of width B by length L, with a moment about the L axis, the Meyerhof Method, reduces footing width B' = B – 2e, where the eccentricity e = M/P
- Design bearing stress, $\sigma_{bearing}$ = P / (B')(L)

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Section II – H.2 – Piles

- When building or bridge foundations are pile supported, they may be assumed to resist lateral forces in three ways
  1. Pile bending resistance where the piles act like vertically orientated cantilevered beams
  2. Battered piles where a component of axial resistance acts in the horizontal direction
  3. Pile groups that can act like a *bolt group* (P/A ± Mc/I) or as a complex combination of bending and shear in piles and caps

Section II – H.2 – Piles

- Consider pile bending resistance to lateral loads, piles can be rigid (fence posting) or flexible.
- Laterally loaded piles are a soil-structure interaction class of problem
- Deflections, shears, and bending moments of laterally loaded piles can be solved by a hand calculation method by Broms (1964) or a finite-difference computer program such as L-Pile developed by Reese et al. (1984) for flexible piles.
- L-Pile treats the soil as a beam on nonelastic foundation springs by use of p-y curves at nodes

Section II – H.2 – Piles

- Broms’ method – see FHWA NHI-05-042 (2006) available on the internet
- FHWA’s 2006 pile manual says
  - “The method (Broms) calculates the ultimate soil resistance to lateral load as well as the maximum moment induced in the pile. Broms' method can be used to evaluate fixed or free head conditions in either purely cohesive or purely cohesionless soil profiles. The method is not conducive to lateral load analyses in mixed cohesive and cohesionless soil profiles.”
- In the FHWA manual, Broms’ method has 14 steps and is 14 pages long.

Section II – H.2 – Piles

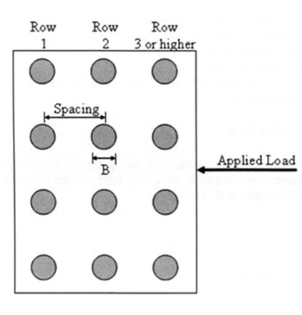
- FHWA recommends L-Pile for complex soil profiles
- Broms' and L-Pile calculate the lateral capacity of a single pile. If a pile group is required, AASHTO 2014 lateral load group reduction factors are used (see below).
- AASHTO lateral GRFs are for driven and drilled piles and for piles in bending (not for fence-posted short rigid piles). Note: lateral and vertical GRF's are different.

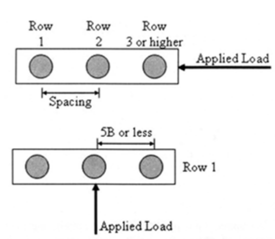
Table 10.7.2.4-1—Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from Hannigan et al., 2006)

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.8	0.4	0.3
5*B*	1.0	0.85	0.7

Section II – H.2 – Piles

- Group reduction factor determined by the center to center pile spacing, *z*, in the direction of loading.





Note: Loading perpendicular to a single row of piles was formerly >2.5B for GRF = 1.0; now, 5B required for GRF = 1.0 and 3B = 0.8

Section II – H.2 – Piles

- Formerly method of installation reduction factor
  - For driven displacement piles, use no reduction in sandy soils and use reduction for clayey soil
  - For drilled piles or drilled shafts, use values in table (assumes disturbance caused by drilling)
  - For jetted piles, use 0.75 of the value determined after applying the group reduction factor listed in table
- AASHTO says minimum pile spacing 2.5B, drilled pile construction sequence required if spacing<6B

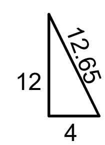
Section II – H.2 – Piles

- Piles can be assumed to be fixed at a depth called the *equivalent point of fixity* or EPF
- EPF is defined as the point where the pile can be assumed to be fixed such that the pile head deflection computed by the structural model and the L-Pile model match indicating equivalent stiffness
- The pile's bending moment calculated at the EPF is often approximately twice the maximum moment determined by the L-Pile program

Section II – H.2 – Piles

- Next we can use battered piles to resist lateral foundation loadings
- Use of battered piles often assumes that they have axial load resistance only
- Given a 100-ton pile battered at 4 on 12 (horizontal to vertical), what is its allowable vertical and lateral load capacity?

$P_{vertical}$ = (12/12.65)(200 kips)
$P_{vertical}$ = 190 kips
$P_{horizontal}$ = (4/12.65)(200 kips)
$P_{horizontal}$ = 63 kips

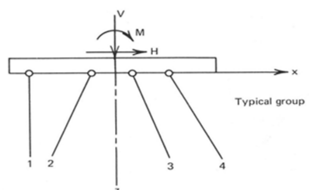


Section II – H.2 – Pile Groups

- Groups of piles can be used to resist lateral loads
- Pile groups are commonly analyzed at three levels of complexity:
  1. The simple static method
  2. The load deflection curve method (i.e., p-y method using a program like GROUP a cousin of L-Pile)
  3. Custom computer analyses considering stiffness interactions between piles, a flexible cap and soil

Section II – H.2 – Pile Groups

- The Simple Static Method – "Bolt Group Analogy"



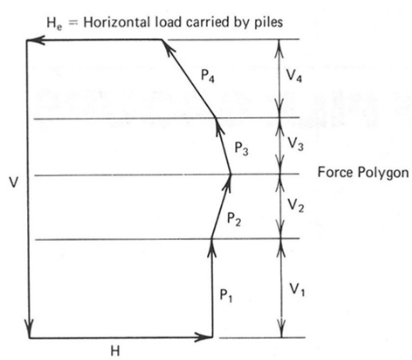
$$V_i = \frac{V}{n} + \frac{M x_i}{\sum_{j=1}^{n} (x_j^2)}$$

Assuming:

1. Pile cap is rigid
2. All piles equally share vertical load
3. No interaction of piles
4. No interaction of soil with piles or cap
5. Statically determinate, P/A ± Mc/I
6. If there is a residual horizontal force, it is shared equally by all piles

Figure from Poulos and Davis, 1980

Section II – H.2 – Pile Groups



1. Forces in piles are resolved by a vector diagram
2. Batters on piles can be adjusted to make residual $H_e = 0$
3. Another assumption used is to assume that battered piles resist lateral forces and vertical piles resist V and M

Figure from Poulos and Davis, 1980

Section II – H.2 – Pile Groups

- The GROUP Program analyzes pile groups for axial, lateral, moment, and torque loadings in 2 or 3 dimensions
- Piles can be vertical or battered, fixed head, pinned head, or elastically restrained head in pile cap
- Piles can vary in size, stiffness, length, and spacing
- Cap can settle, translate, and rotate
- Cap is a RIGID BODY

Section II – H.2 – Pile Groups

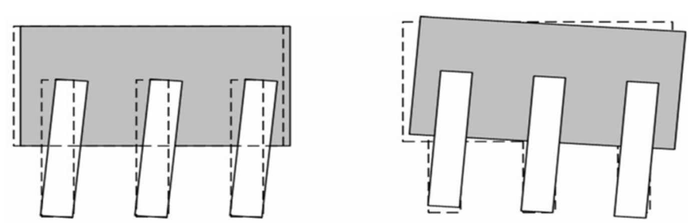
- GROUP considers nonlinear soil response to axial loading, torsional loading, and lateral loading (p-y curves)
- Piles are assumed to be widely spaced (≥ 8 diameters). Does not consider close spaced piles in general.
- Interaction behavior of close spaced piles of the second type are considered by GROUP.
- There are structural and soil considerations involved in complete pile group interaction analysis.

Section II – H.2 – Pile Groups

- Close pile spacing: important pile group issue.
  - Close-spaced piles have two types of interaction: (1) Pile-Soil-Pile Interaction, and (2) Cap redistribution of axial, lateral, and moment loadings
  - Pile-Soil-Pile Interaction is when individual piles in the group are less efficient than an isolated pile, called the *shadow effect*. This first type of interaction cannot be solved by GROUP; input from engineer required. 2016 GROUP will suggest p-modifiers, but still requires engineer evaluation.
  - Cap redistribution is when loadings on individual piles in the group are changed to be compatible with displacements and rotations of the pile cap. This second type of interaction can be solved by the GROUP program if piles are properly modeled.

Section II – H.2 – Pile Groups

- Complex analyses of laterally loaded pile groups consider stiffness interactions between piles, a flexible cap and soil



(a) Rotation of piles within cap 2012 AASHTO (b) Rotation of piles and cap together

Section II – H.2 – Pile Groups

- Pile fixity in cap and rotation in cap affects lateral pile deflection (ASCE Geotechnical Special Publication 132, Duncan, Robinette, Mokwa)
- Duncan et al., conclude that piles perfectly fixed against rotation deflect about 25% as much as pinned head piles.

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Section II – H.2 – Drilled Piers and Drilled Shafts

- Drilled piers (i.e. drilled shafts) and drilled piles are deep foundations that look like driven piles, but they are different.
- Driven piles displace soil as they are driven into the ground which can densify sandy soils.
- Installing drilled piers/shafts can result in soil loss from sidewalls which reduces soil density.
- Lateral load tests can reduce uncertainty in determining lateral resistance of drilled piers, allowing larger values of LRFD resistance factors.

Structural Design Standards Relevant for Foundation Design

- In order of precedence of controlling requirements for forces in foundation design
  – For buildings
    - *International Building Code* (IBC 2012 Edition) without supplements
    - *Minimum Design Loads for Buildings and Other Structures*, 2010 (ASCE 7-10, third printing)
  – For highway structures—*AASHTO LRFD Bridge Design Specifications, 7th edition, 2014.*

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Structural Design Standards
Relevant for Foundation Design

- ACI 318-11, 2011 Building Code Requirements for Structural Concrete
- Notes on ACI 318-11 Building Code—includes solved examples
- Any modern geotechnical textbook that you are using. It is not good strategy to take an unfamiliar book to the SE exam.