

2015 NEHRP Recommended Seismic Provisions: Training and Instructional Materials

FEMA P-1052/July 2016

Foundation and Liquefaction Design

Ian McFarlane, S.E. and Stephen K. Harris, S.E. Originally developed by Michael Valley, S.E.

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LIQUEFACTION DE
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• Shallow Footing Example FOUNDATION AND
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• Pile Foundation Example
• Design for Differential Settlement Example FOUNDATION AND LIQUEFACTION DESIGN

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Indation Example
for Differential Settlement Example
for Liquefiable Soil Example
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• Design for Liquefiable Soil Example • TABLE OF CONTENTS:
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• Design for Differential Settlement Example
• Design for Liquefiable Soil Example
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**FOUNDATION DESIC
• Proportioning Elements for:**
• Transfer of Seismic Forces **FOUNDATION DESIGN**
• Proportioning Elements for:
• Transfer of Seismic Forces
• Strength and Stiffness **FOUNDATION DESIC
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• Transfer of Seismic Forces
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• Shallow and Deep Foundations
• Elastic and Plastic Analysis FOUNDATION DESIGN

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- and Deep Foundations

And Plastic Analysis

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 3
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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 ath and Transfer of Seismic Forces

vertical pressures - shallow

Overturning moment

Load Path and Transfer of Seismic Forces $\frac{d}{dx}$ and Transfer of Seismic Forces
vertical pressures - deep

**ss Design vs
Design
• Strength Design
– Permitted in 2015
Standard Design vs
esign**
Strength Design
– Permitted in 2015
Standard
– Nominal strength **Design vs
esign**
– Permitted in 2015
– Standard
– Nominal strength
with phi factors
– Allows direct **Allowable Stress
Strength De
• Allowable Stress • S
Design – Traditional/historic** – Traditional/historic **Allowable Stress D

Strength Des

Allowable Stress • Stre

Design – Traditional/historic F

Allowable – Allowable geotechnical values – A

have inherent factor – Company

Company – A

Traditional values – A

Traditional v** Allowable Stress Design vs Strength Design

- Design
	- approach
	- geotechnical values have inherent factor of safety Allowable Stress

	Design - Traditional/historic S

	approach - Allowable

	geotechnical values - A

	have inherent factor - A

	have inherent factor - Co

	of safety fc

	- Requires separate M

	ASD load combination
	- ASD load combination
- - Standard
	- with phi factors
- Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Foundation Design 9

Foun **Example 18**

Strength Design

- Permitted in 2015

Standard

- Nominal strength

with phi factors

- Allows direct

comparison of

foundation capacity comparison of foundation capacity with superstructure capacity

**Allowable Stress Des
Strength Desigrey
• Nominal Strength can be:
• Nominal strength associated verticinated foilure meaborism Allowable Stress Design vs
Strength Design
Nominal Strength can be:
- Nominal strength associated with
anticipated failure mechanism
Nominal strength associated with limitaties** 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 Allowable Stress Design vs Strength Design

- - anticipated failure mechanism
	- on maximum deformation at failure
- nal strength associated with limitation
aximum deformation at failure
ent based on sustained loads rather
Foloding associated with nominal
should be considered
Instructional Material Complementing FEMA 1051, Design Example • 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 rat than the loading associated with nominal strength should be considered

**Allowable Stress Design vs
Strength Design
• Nominal Strength Example
– Strength reduction factor Φ=0.45 for
managemies (begring preseurs) Allowable Stress Design vs
Strength Design
Mominal Strength Example
– Strength reduction factor Φ=0.45 for
compression (bearing pressure)
Mayable Strees Example Allowable Stress Design v:**

Strength Design

• Nominal Strength Example

– Strength reduction factor Φ =0.45 for

compression (bearing pressure)

• Allowable Stress Example

– Factor of safety of ~3.0 Allowable Stress Design vs Strength Design

-
- compression (bearing pressure) **Strength Design
Strength Design
Nominal Strength Example
– Strength reduction factor** Φ **=0.45 for
compression (bearing pressure)
Nowable Stress Example
– Factor of safety of ~3.0**
- - Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 11

Foundation Design Concepts

- **Foundation Design Concepts**
• Most foundation failures are associated with
excessive foundation movement, not loss of
load-bearing capacity excessive foundation movement, not loss of load-bearing capacity **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
- Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 12 load-bearing pressure is encouraged to minimize differential settlement

Design of Concrete Footings

-
- **Design of Concrete Fo
• ACI 318 References
– Chapters 7,8 One way, Two
referenced for detailing require Design of Concrete Footings

ACI 318 References

– Chapters 7,8 – One way, Two-way slabs

referenced for detailing requirements

Chapter 13 – Canaral Feundations** referenced for detailing requirements **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 **Design of Concrete Footings**

ACI 318 References

- Chapters 7,8 – One way, Two-way slabs

referenced for detailing requirements

- Chapter 13 – Seismic Provisions

- Chapter 22 – Sectional Strength ACI 318 References

– Chapters 7,8 – One way, Two-way slabs

referenced for detailing requirements

– Chapter 13 – Seismic Provisions

– Chapter 22 – Sectional Strength
	-
	- Ier 13 General Foundations
ter 18 Seismic Provisions
ter 22 Sectional Strength
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	-

CONTICALLY
 INSTRUMENT COMPLETE:

The Complementing FEMA 1051, Design Examples Foundation Design - 14

The Condation Design - 14

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 14
 Reinforced Concrete Footings: Basic Design **Criteria** (concentrically loaded)

(b) Critical section for one-way shear (a) Critical section for flexure column and edge of steel base plate (typical) extent of footing (typical) d

Outside face of concrete column or line midway between face of steel

(c) Critical section for two-way shear

Ilastic, after uplift

Complements

Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 15 Footing Subject to Compression and Moment: Uplift Nonlinear

(e) Some plastification

(f) Plastic limit

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

• 3,000B psf (Square) Shallow Footing Examples

Soil parameters:

- Medium dense sand
- (SPT) $N = 20$

Bearing Capacity

-
- 4,000B' psf (Rectangular)
-
- Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 17

Instructional Material Complementing FEMA 1051, Design Examples Found • Medium dense sand

• (SPT) N = 20

• Encorrection Searing Capacity

• 3,000B psf (Square) (Set

• 4,000B' psf

• A = footing width (ft)

• B = footing width (ft)

• B' = average width of

compressed area (ft) • (SPT) $N = 20$ • Fr

Bearing Capacity Allov

• 3,000B psf (Square) (Set

• 4,000B' psf • 2

• B = footing width (ft)

• B' = average width of

• B' = average width of

• mpressed area (ft)

• ϕ =0.45 compressed area (ft) • ϕ=0.45
-

- Density = 125 pcf
- Friction angle = 33o

Allowable Bearing (Settlement) **9 Examples
•** Density = 125 pcf
• Friction angle = 33°
Allowable Bearing
(Settlement)
• 2,000 psf B<20 ft
• 1,000 psf B<40 ft • Density = 125 pcf
• Friction angle = 33°
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

- **Combining Loads:**
• Maximum downward load:
 $1.2D + 0.5L + E$ $1.2D + 0.5L + E$ **Combining Load**
• Maximum downward load:
1.2D + 0.5L + E
• Minimum downward load:
0.9D + E • Maximum downward load:

• Maximum downward load:

• Minimum downward load:
 $0.9D + E$

• Definition of seismic load effect E:
 $E = r1QE1 + 0.3 r2QE2 + (-0.2 SDSE)$
-

 $0.9D + E$

In of seismic load effect E:
r1QE1 + 0.3 r2QE2 +/- 0.2 SDSD
1.0 ry = 1.0 and SDS = 1.0
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 21 ximum downward load:
1.2D + 0.5L + E
imum downward load:
0.9D + E
inition of seismic load effect E:
E = r1QE1 + 0.3 r2QE2 +/- 0.2 SDSD
rx = 1.0 ry = 1.0 and SDS = 1.0 ximum downward load:

1.2D + 0.5L + E

imum downward load:

0.9D + E

inition of seismic load effect E:

E = r1QE1 + 0.3 r2QE2 +/- 0.2 SDSD

rx = 1.0 ry = 1.0 and SDS = 1.0

Reactions

Reduction of Overturning Moment

- **Reduction of Overturning Moment**
• NEHRP Provisions allow base overturning
moment to be reduced by 25% at the soil-
foundation interface moment to be reduced by 25% at the soilfoundation interface **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 overturnin**
- Instructional Material Complementing FEMA 1051, Design Examples
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 23 loads are the resultants of base overturning moment, whereas column moments are resultants of story shear • 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
-

Additive Load w/ Largest eccentricity

- 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 applying the 0.75 multiplier for overturning effects to the axial loads, and neglecting the weight of the foundation and overlying soil, **Additive Load weccentric
• Combining loads on footing applying the 0.75 multiplies
• effects to the axial loads, a
• P = 688 kips
• Mxx = -6,717 ft-kips eccentricity**

• Combining loads on footings A-⁴

applying the 0.75 multiplier for o

effects to the axial loads, and ne

weight of the foundation and ove

• P = 688 kips

• Mxx = -6,717 ft-kips

• Myy = -126 ft-kips (• 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
•
-
-
- Instructional Material Complementing FEMA 1051, Design Examples
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 24

Counteracting Load w/ Largest e

- **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 6, including the overturning factor, and neglecting the weight of the footing and overlying soil, **Counteracting Load**

• Again combining loads on

6, including the overturning

neglecting the weight of the overlying soil,

• $P = 332$ kips

• Mxx = -5,712 ft-kips • Again combining loads on footin
6, including the overturning factor
neglecting the weight of the foot
overlying soil,
• $P = 332$ kips
• Mxx = -5,712 ft-kips
• Myy = -126 ft-kips (negligible) • Again combining loads on footings A-8
6, including the overturning factor, and
neglecting the weight of the footing ar
overlying soil,
• $P = 332$ kips
• $Mxx = -5,712$ ft-kips
• $Myy = -126$ ft-kips (negligible)
-
-
- kips
5,712 ft-kips
126 ft-kips (negligible)
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 25

Elastic Response

- Elastic Resp

 Objective is to set L

and W to satisfy

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

Additive Combination

Given
$$
P = 688
$$
 k, $M = 6717$ k-fit
\n $e = M / P = 9.76$ ft
\nTry $L = 40$ ft, $B = 9$ ft
\n $L/6 < e < L/2$ therefore elastic with some uplift
\n $L' = 3 (L/2-e) = 30.7'$
\n $q_{max} = 2 P / [3 B(L/2-e)]$
\n $q_{max} = 4.98$ 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 Bearing Capacity:
 $Q_{ns} = 4,000B' = 4,000 \text{ min (B, L'/2)}$
 $Q_{ns} = 4,000 \text{ min (9, 30.7/2)} = 36,000 \text{ psf}$

Design bearing capacity:
 $\phi Q_{ns} = 0.45 \text{ (36,000 psf)}$
 $\phi Q_{ns} = 16.2 \text{ ksf} > 4.98 \text{ ksf}$

Design bearing capacity: $\phi Q_{\text{ns}} = 0.45$ (36,000 psf) OK by elastic analysis

Plastic Response

- **Plastic Resp**
• Same objective as
for elastic
response for elastic response **Plastic Resp**
• Same objective as
for elastic
response
• Smaller footings
can be shown OK
thus
- can be shown OK thus

Counteracting Case
• Given $P = 332$ k; $M = 5712$ k-ft
• $e = M/P = 17.2$ ft
• $T_{P}U = 40$ ft $B = 9$ ft **Counteracting Case**

Siven $P = 332$ k; $M = 5712$ k-ft
 $P = M / P = 17.2$ ft

Fry $L = 40$ ft, B = 9 ft

Fry elastic solution:
 $- L' = 3(40/2 - 17.2) = 8.4$ ft
 $-$ qmax = 2 **P** / [3 **B**(L/2-**e**)] = 8.82 ksf

Bearing Capacity: Counteracting Case

- **Counteractine

 Given** $P = 332$ **k;** $M = 5712$ **k-ft

 e = M / P** = 17.2 ft

 Try *L* = 40 ft, B = 9 ft

 Try elastic solution:

 $1' = 3(40/2 17.2) = 8.4$ ft • 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

emax = 2 **P** / [3 **B**/l /2-9] = 1 • 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

• Bearing Capacity:

– Qns = 4,000 min (9,8.4/2
- $e = M / P = 17.2$ ft
-
- -
- Gounteracting Case

Siven $P = 332$ k; $M = 5712$ k-ft
 $P = M / P = 17.2$ ft

Fry $L = 40$ ft, B = 9 ft

Fry elastic solution:
 $L' = 3(40/2 17.2) = 8.4$ ft
 $-$ qmax = 2 P / [3 B(L/2-e)] = 8.82 ksf

Bearing Capacity:

Ons = 4.0
- -
- $I = 2 \text{ P} / [3 \text{ B}(L/2-e)] = 8.82 \text{ ks}$

Capacity:
 2 A ,000 min (9,8.4/2) = 16,800 psf
 $= 0.45$ (16.8 ksf) = 7.56 ksf < qmax NG

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 30 Siven $P = 332$ k; $M = 5712$ k-ft
 $\cdot = M / P = 17.2$ ft

Fry $L = 40$ ft, B = 9 ft

Fry 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 $\mathbf{v} = \mathbf{M} / \mathbf{P} = 17.2 \text{ ft}$

Fry *L* = 40 ft, B = 9 ft

Fry 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

Counteracting Case

Settlement Analysis

-
- **Settlement Analysis
• Long term settlement verification
• Load combination: D+0.5L per Appendix C
Commentery (ASCE 3) Settlement Analysis
• Long term settlement verification
• Load combination: D+0.5L per Appendix C
Commentary (ASCE 7)** Commentary (ASCE 7) **Settlement Ar

• Long term settlement verifi

• Load combination: D+0.5l

Commentary (ASCE 7)

• P = 340 kips

• Q_{sustained} = 340,000 / (9' x 4 • 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 • Long term settlement verifi

• Load combination: D+0.5L

Commentary (ASCE 7)

• P = 340 kips

• Q_{sustained} = 340,000 / (9' x 4

• Q_{allowable} = 2,000 psf

• **OK for settlement**
-
- KIPS
 $I_1 = 340,000 / (9' \times 40') = 945 \text{ psf}$
 $I = 2,000 \text{ psf}$
-
- OK for settlement

Additional Checks

- Additional Checks
• Moments and shears for reinforcement
• Should be checked for the overturning case should be checked for the overturning case 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
- moments and shears in concrete
- Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 33
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 33 **• Moments and shears for reinforcement

• Moments and shears for reinforcement

• Plastic soil stress gives upper bound on

moments and shears in concrete

• Horizontal equilibrium:** $H_{max} < \phi \mu(P+W)$ **

in this case friction** 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

Elastic solution pressures (ksf)

Central Mat Bearing

Design bearing capacity:

 $\phi Q_{\text{ns}} = 0.45$ (27.12 ksf) = 12.21 ksf OK

Central Mat Bearing

Verify equilibrium:

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 39 $(12.21)(6.78)(95) = 7864$ kips ≈ Axial Load (1749 kips) $(7849)(5.42) = 42,542$ ft-kips ≈ Off-axis moment (42,544 ft-kips)

Central Mat Bearing

Verify equilibrium:

 $(12.21 \text{ ksf})(6.78 \text{ ft})(95 \text{ ft}) = 7864 \text{ kips} \approx \text{Axial}$ Load (1749 kips)

Iibrium:
 $(6.78 \text{ ft})(95 \text{ ft}) = 7864 \text{ kips} \approx \text{Axial}$
 9 kips
 $)(5.42 \text{ ft}) = 42,542 \text{ ft-kips} \approx \text{Off-axis}$
 $2,544 \text{ ft-kips}$

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 40 (7849 kips)(5.42 ft) = 42,542 ft-kips $≈$ Off-axis moment (42,544 ft-kips)

Central Mat Flexural Reinforcement

**Central Mat Flexural F
• Use moment
contours to define
areas of flexural** contours to define areas of flexural $\qquad \qquad \circ$ reinforcement density

Central Mat Shear Design

- **Central Mat Shear
• Critical section at d/2
(two-way) and d
(one-way)** (two-way) and d (one-way) **Central Mat Sheandler**

• Critical section at d/2

(two-way) and d

(one-way)

• Be aware of size-

effect in thick

foundation elements
- effect in thick foundation elements

Pile/Pier Foundations

View of cap with column above and piles below

• Pile Stiffness: **Pile/Pier Founda

Pile Stiffness:**

– Short (Rigid)

– Intermediate

– Long – Intermediate – Long • Cap Influence Pile/Pier Foundations

- -
	-
	-
- Group Action
-
-
- Soil Stiffness **dations

Soil Stiffness

– Linear springs –

nomographs e.g.

NAVFAC DM7.2 lations

il Stiffness

Linear springs –

nomographs e.g.
NAVFAC DM7.2

Nonlinear springs –** NAVFAC DM7.2
- Uence

IPILE or similar

IPILE or similar

Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Foundation Design 44 **ndations**

Soil Stiffness

– Linear springs –

nomographs e.g.

NAVFAC DM7.2

– Nonlinear springs –

LPILE or similar

analysis LPILE or similar analysis

Sample p-y Curves

Passive Pressure

Pile Shear: Two Soil **Stiffnesses**

Pile Moment 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

-
- **Pile Design
• Site Class E
• Substantially more
reinforcement Pile Design
•** Site Class E
• Substantially more
• "Full" spiral for 7D reinforcement **Pile Design
•** Site Class E
• Substantially more
• "Full" spiral for 7D
• Confinement at
• boundary of soft and
-
- **Pile Design

•** Site Class E

 Substantially more

 "Full" spiral for 7D

 Confinement at

boundary of soft and

firm soils (7D up and boundary of soft and firm soils (7D up and 3D down)

Other Topics for Pile Foundations

-
- **Other Topics for Pile Foundation**
• Foundation Ties: $F = P_G(S_{DS}/10)$
• Pile Caps: high shears, rules of thumb; k **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 for 3D strut and tie methods in future **Other Topics for Pile Foundatio**
• Foundation Ties: $F = P_G(S_{DS}/10)$
• Pile Caps: high shears, rules of thumb;
for 3D strut and tie methods in future
• Liquefaction: another topic $\frac{|\overline{SMQ}|}{|\overline{SMQ}|}$
• Kinematic interacti **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 laye
- ISM2
- Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 52

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 $S_{DS}/10$
- $\begin{array}{l} \text{Ricat} \text{in} \ \text$ •Oftentimes use grade beams or thickened slabs on grade

Example: Differential Settlement

- Table 12.13-3
- Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Comp when subject to imposed settlement

Structural Requirements

- **Structural Requirements
• Analysis is required to use shallow
• Ale less of arouty support permitted** foundations **Structural Requirements
• Analysis is required to use shallow
• No loss of gravity support permitted
• Residual member strength at least 2/3 of
• undergood persinal strength**
-
- **Structural Requirements

 Analysis is required to use shallow

 No loss of gravity support permitted

 Residual member strength at least 2/3 of

undamaged nominal strength

 If demands avecad nominal strength** undamaged nominal strength
	- II member strength at least 2/3 of
ged nominal strength
hands exceed nominal strength
deration of nonlinear behavior is
red Analysis is required to use shallow

	oundations

	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 consideration of nonlinear behavior is required

Floor Framing Plan

Instructional Material Complementing FEMA 1051, Design Examples

Instructional Material Complementing FEMA 1051, Design Examples

Foundation Design - 56 Floor Framin
• Steel SMF
frame building frame building Floor Framin

• Steel SMF

frame building

• Two conditions

studied

leterier studied Floor Framing

Steel SMF

rame building

Two conditions

studied

— Interior

— Perimeter

— Steel SMF • Steel SMF

frame building

• Two conditions

studied

– Interior

– Perimeter

• Expected

Settlement,
 $\delta = R^n$ Settlement, ¹ $\delta_{v} = 8"$

Rotation Demand

Condition 1

- **Condition**
• SMF members and
connection can sustain
0.04 rad connection can sustain 0.04 rad. **Condition

• SMF members and

connection can sustain

0.04 rad.

• Must assess:

– Simple shear tab Condition**

SMF members and

connection can sustain

0.04 rad.

Must assess:

- Simple shear tab

connection to web

Crowity beem $\begin{array}{r} \text{connection can sustain} \\ \text{0.04 rad.} \\ \text{Must assess:} \\ \text{– Simple shear tab} \\ \text{connection to web} \\ \text{– Gravity beam} \\ \text{– Column weak axis} \end{array}$
- - connection to web
	-
	-

Check of W16x36 Beam

Check of W16x36 Beam
• Consider the beam fixed-ended with an imposed displacement imposed displacement

 $\begin{array}{c}\n\text{Oksi}(448in^4)(8in)/(25 ft \times 12 in/ft)^3 = 46 kips\n\text{Oksi}(448in^4)(8in)/(25 ft \times 12 in/ft)^2\n\hline\n-in = 577 kip - ft\n\text{Instructional Matorial Complomoning FEMA 1051, Design Examples}\n\end{array}$ Foundation Design - 59 $V_{\Lambda} = 12EI\Delta/L^3$ $M_{\Lambda} = 6EI\Delta/L^2$

W FEMA

Add gravity loading

- Add gravity loading
• Compute the gravity shear and moment
–85 psf dead load, 50 psf live load Add gravity loading

Sompute the gravity shear and moment

- 85 psf dead load, 50 psf live load

- 4.17 feet tributary width
	-

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 60 – 4.17 feet tributary width

$$
V_u = 46 + 11 = 57 \text{ kips}
$$

$$
M_u = 577 + 45 = 622 \text{ kip} - ft
$$

Compare to design values
• Shear capacity $\phi V_n = 141$ kip
– Beam is adequate for shear **Compare to design value**
Shear capacity $\phi Vn = 141$ kip
– Beam is adequate for shear
Flexural capacity $\phi Mn = 174$ kip-ft **Compare to design values**
• Shear capacity $\phi Vn = 141$ kip
- Beam is adequate for shear
• Flexural capacity $\phi Mn = 174$ kip-ft
- Demand exceeds flexural capacity Compare to design values

- -
- -
- and exceeds flexural capacity

Ithe beam will yield

action will yield

actional Material Complementing FEMA 1051, Design Examples Foundation Design 61

Foundation Design 61 **Compare to design values**

Shear capacity ϕV_n = 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

- connec **Compare to design values**

Shear capacity $\phi V_n = 141$ kip

- Beam is adequate for shear

Flexural capacity $\phi Mn = 174$ kip-ft

- Demand exceeds flexural capacity

- Either the beam will yield in flexure or the

connection connection will yield

- **Check Conner

 Shear

capacity
** $\phi V_n = 78.3 \text{ kin}$ capacity $\phi Vn = 78.3$ kip **Check Conner

• Shear

capacity
** $\phi Vn = 78.3$ **kip

• Flexural

capacity
** $\phi Mn = 21$ **kin-ft**
- capacity $\phi Mn = 21$ kip-ft • Shear

capacity
 $\phi Vn = 78.3$ kip

• Flexural

capacity
 $\phi Mn = 21$ kip-ft

• Connection

will yield in

flexure
- will yield in flexure

- Check Conne

 Check plate

thickness to

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

- Check Conne

 Maximum plate

thickness is

accentable thickness is acceptable Check Connec

Maximum plate

hickness is

acceptable

– But *not* if it

were 50 ksi

steel
	- were 50 ksi steel

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

Check Conne

• Check plastic

• rotation to provide

• collanse resistance rotation to provide collapse resistance per ASCE 41 Check Connec

Check plastic

otation 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

-
- **Bi-Axial bending of column F**
• Considered acceptable, because:
– SMF column is known to be able to re **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 any moment the beam can impart **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
	- moment of weak axis connection is

	iall as to be considered negligible

	Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 66 so small as to be considered negligible

Condition 2

Check of W24x62 Girder

Check of W24x62 Girder
• Consider the beam fixed-ended with an imposed displacement imposed displacement

 $M_0 = 23.973$ kip – in = 1.998 kip – ft $\Delta = (0)(29000KSI)(1,3)$ $\frac{4}{(8in)}$ /(25 ft x 12 in /ft)² $\Delta = (12)(29000$ KSU $)(1)$ $\frac{4}{(8in)}$ /(25 ft x 12 in/ft)³ = 160 kins

Add gravity loading

- Add gravity loading
• Compute the gravity shear and moment
– 85 psf dead load, 50 psf live load
	-
	- **Add gravity loading**
20 Depropeed Sompute the gravity shear and moment
25 psf dead load, 50 psf live load
208 sq. ft. each Two equal point loads of 208 sq. ft. each **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
** $\mu_g = 1.2(208 \times 85) + 0.5(208 \times 50) = 26 \; kips$

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

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

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

$$
2(208 \times 85) + 0.5(208 \times 50) = 26 \text{ kips}
$$
\n
$$
g = 26 \text{ kips}
$$
\n
$$
u_g l/3 = (26)(25 \text{ ft})/3 = 217 \text{ kip} - \text{ft}
$$
\n
$$
V_u = 160 + 26 = 186 \text{ kips}
$$
\n
$$
M_u = 1,998 + 217 = 2,215 \text{ kip} - \text{ft}
$$
\nInstructional Material Complementing FEMA 1051, Design Examples

\nFoundation Design-69

Compare to design values
• Shear capacity ϕV_n = 306 kip
– Girder is adequate for shear **Compare to design value:**
Shear capacity $\phi Vn = 306$ kip
– Girder is adequate for shear
Flexural capacity $\phi Mn = 482$ kip-ft **Compare to design values**
• Shear capacity $\phi Vn = 306$ kip
- Girder is adequate for shear
• Flexural capacity $\phi Mn = 482$ kip-ft
- Demand exceeds flexural capacity Compare to design values

- -
- -
- and exceeds flexural capacity

Ithe girder will yield

action will yield

actional Material Complementing FEMA 1051, Design Examples Foundation Design 70 **Compare to design values**

Shear capacity ϕV_n = 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

conn **Compare to design values**

Shear capacity $\phi Vn = 306$ kip

- Girder is adequate for shear

Flexural capacity $\phi Mn = 482$ kip-ft

- Demand exceeds flexural capacity

- Either the girder will yield in flexure or the

connec connection will yield

- **Check Conne
• Shear capacity
** $\phi V n = 133$ **kip
Shert eletted** $\phi Vn = 133$ kip
	- holes req'd
- $\phi Mn = 63$ kip-ft
- yield in flexure

Check Connection

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

Check Connection

- **Check Conner
• Maximum plate
thickness is
accentable** thickness is acceptable
	- were 50 ksi steel

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

Check Connection

**Check Conner
• Check plastic
• rotation to provide
• collanse resistance** rotation to provide collapse resistance per ASCE 41 Check Connec

Check plastic

otation to provide

collapse resistance

per ASCE 41

— Acceptable

Conclusions

- **Conclusions**
• The building can sustain the differential settlements imposed
• Acceptable to use shallow foundations by the liquefaction **Conclusions**
• The building can sustain the differential settlements is
by the liquefaction
• Acceptable to use shallow foundations
– This assessment was primarily concerned with
connection ductility **Conclusions**

The building can sustain the differential settlements impos

by the liquefaction

Acceptable to use shallow foundations

— This assessment was primarily concerned with

connection ductility

— Other systems **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 system building can sustain the differential settlements imposed

be liquefaction

eptable to use shallow foundations

his assessment was primarily concerned with

onnection ductility

of ther systems may have different concerns
 building can sustain the differential settlements imposed

he liquefaction

eptable to use shallow foundations

his assessment was primarily concerned with

onnection ductility

of the systems may have different concerns

-
- connection ductility eptable to use shallow foundations

his assessment was primarily concerned with

onnection ductility

of the reversals causing the checked for shear and remain

essentially elastic

• Non SMF steel moment connections may n
	- - essentially elastic
- Instructional Material Complementing FEMA 1051, Design Examples
Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 75 ductility
	-

Foundations on Liquefiable Soil

-
- **Foundations on Liquefiable Soil
• Effects of Liquefaction on Structures
– Differential settlement leading to damage** Foundations on Liquefiable Soil
Effects of Liquefaction on Structures
— Differential settlement leading to damage
— Letarel arreging leading to damage to in the superstructure 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
	- foundations and superstructure

Performance Goals of Provisions

- **Performance Goals of Provisions**
• Structures are intended to resist collapse due
the liquefaction effects
MCE around metions the liquefaction effects **The Coals of Proference Coals of Proference Coals of Proference Structures are intended to resist
The liquefaction effects
- MCE_G ground motions.
Distinct from remainder of Provis 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 **Proformance Goals of Pr**
Structures are intended to resist
he liquefaction effects
– MCE_G ground motions.
Distinct from remainder of Provis
– 2/3 of MCE_R ground motions
	-
- - $-2/3$ of MCE_R ground motions

Nonlinear Behavior

- **Monlinear Behavior
• Settlement or lateral spreading due to
• liquefaction may cause nonlinear behavior
• This behavior must be addressed where** liquefaction may cause nonlinear behavior **• Nonlinear Behavior
• Settlement or lateral spreading due to
liquefaction may cause nonlinear behavior
• This behavior must be addressed where
applicable
• Complete poplinear applyie pot required**
- applicable
- Ite nonlinear analysis not required,
1 that nonlinear behavior is assessed
iately. **• Settlement or lateral spreading due to

• Settlement or lateral spreading due to

• This behavior must be addressed where

• policable

• Complete nonlinear analysis not required,

provided that nonlinear behavior is as** provided that nonlinear behavior is assessed appropriately.

Liquefaction

- **Liquefaction
• Occurs in loose, saturated sands and silts
with poor drainage** with poor drainage Liquefaction
Dccurs in loose, saturated sand
with poor drainage
– Differential settlement
	-

Lateral Spreading

**Lateral Spreading
• Can occur when liquefaction occurs adjacent
to a scarp, channel or riverbank.** to a scarp, channel or riverbank.

Example: Lateral Spreading

-
-

Structural Requirements

- **Structural Requirements

 Analysis is required to demonstrate

acceptable pile behavior

 Neplipear behavior is permitted** acceptable pile behavior **Structural Requirements
• Analysis is required to demonstrate
acceptable pile behavior
• Nonlinear behavior is permitted
– No loss of gravity support permitted Structural Requirements

Analysis is required to demonstrate

acceptable pile behavior

Vonlinear behavior is permitted

— No loss of gravity support permitted

— Residual member strength at least 2/3 of

— undemoged nami**
- -
- SS OT Gravity Support permitted
Iual member strength at least 2/3 of
maged nominal strength
riptive detailing for ductility
and shear capacity must not be
eded
Instructional Material Complementing FEMA 1051, Design Example **Structural Requirements**

Analysis is required to demonstrate

acceptable pile behavior

Vonlinear behavior is permitted

— No loss of gravity support permitted

— Residual member strength at least 2/3 of

undamaged nomin undamaged nominal strength Analysis is required to demonstrate
acceptable pile behavior
Vonlinear behavior is permitted
— No loss of gravity support permitted
— Residual member strength at least 2/3 of
undamaged nominal strength
— Prescriptive detai Propertigate pile behavior

Monlinear 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
	-
	- exceeded

Pile Foundation

- **Pile Foundation**
• Lateral displacement 24 in
— Exceeds 18 in limit **Pile Foundati**

- Exceeds 18 in limit

- Exceeds 18 in limit

- Cocurs over depth of 30 ft. **Pile Foundation**
• Lateral displacement 24 in
– Exceeds 18 in limit
• Occurs over depth of 30 ft. **Pile Foundatio**

• Lateral displacement 24 in

– Exceeds 18 in limit

• Occurs over depth of 30 ft.
-
- 250k

Pile Diameter

-
- **Pile Diameter**
• Improved flexural ductility at low axial loads
• Choose diameter, such that $\frac{250k}{4 \cdot 5!} < 0.1$; • Improved flexural ductility at low as
• Choose diameter, such that $\frac{250k}{A_g f_c'}$ < λ $($ λ $($ λ _{λ} $)$

\n- Improved flexural ductility at low axial loads
\n- Choose diameter, such that
$$
\frac{250k}{Agf_c'} < 0.1
$$
; *File Diameter* $\geq 2 \sqrt{\frac{250k}{\pi(5ksi)(0.1)}} = 25.2$ in
\n- Choose 30 in. diameter
\n
\nMA $\sqrt{\text{op}}$ *Instructional Material Complomoting FEMA 1051, Design Examples Foundation Design 84*

Flexural Demand

- **Flexural Demand**
• Consider fixed-fixed behavior between the
bottom of the cap and the top of the
competent soil bottom of the cap and the top of the competent soil. Flexural Demand

Donsider fixed-fixed behavior between the

bottom of the cap and the top of the

— Half of P-Delta moment occurs at two

locations
	- locations

$$
M_{u} = \frac{P_{u}\Delta}{2} = \frac{250k(24in)}{2}
$$

= 3,000 *kip* – *in*
<sup>Instantuctional Material Complementary FEMA 1051, Design Examples
<sup>Instantational Material Complementary FEMA 1051, Design Examples
^{Formation Design - 85}</sup></sup>

Axial-Moment Interaction

light reinforcing and concrete strength of 5ksi

Rotation and Curvature

Property Rotation and Curvature
• Rotation is computed from displacement and
effective length effective length

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

 $p \hspace{1cm} 1$ \mathcal{I} \mathcal{I}

The diameter
 γ_2 pile diameter
 $\varphi = \frac{\theta_t}{l_p} = \frac{0.0667 rad}{15 in} = 0.0044 in^{-1}$

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design - 87 • Rotation is computed from displacement and

• Flective length
 $\theta_t = \frac{24in}{360in} = 0.0667 \text{ rad.}$

• Curvature depends on presumed hinge

length: $\frac{V_2}{2}$ pile diameter
 $\theta_t = \frac{0.0667 \text{ rad}}{0.0667 \text{ rad.}}$ length: ½ pile diameter $t = 0.000$ /ruu -1

Moment-Curvature

- **Moment-Cur

 Strength at**

imposed curvature

remains above imposed curvature 7,000 remains above 67% limit
- Imposed curvature $\frac{1}{2}$
is less than limit due to bar fracture $\frac{2}{3,000}$

Moment-Curvature

- **Moment-Cur**

 Consider whether

smaller pile

diameters would smaller pile diameters would be acceptable
- Smaller diameters $\frac{1}{2}$ $\frac{1}{8,000}$ result in earlier bar fracture
	-
	- design

Detailing for Ductility
ACI 318-14, Sections 18.7.5.2 through
Spiral spacing
— ½ member dimension (7.5 in.)
— Six time bar diameter (6 in.)
— 6 in. Detailing for Ductility

- **Detailing for Ductility
• ACI 318-14, Sections 18.7.5.2 through .4**
• Spiral spacing **Detailing for Denote Spiral spacing**
- Xapiral spacing
- $\frac{1}{4}$ member dimension (7. **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.)**
- -
	-
	-
- **Detailing for Du**
ACI 318-14, Sections 18.7.5.
Spiral spacing
— ½ member dimension (7.5
— Six time bar diameter (6 in
— 6 in.
/olumetric ratio (larger of the • ACI 318-14, Sections 18.7.5.2 through .4

• Spiral spacing
 $- \frac{1}{4}$ member dimension (7.5 in.)
 $-$ Six time bar diameter (6 in.)
 $-$ 6 in.

• Volumetric ratio (larger of the following):
 $\frac{\rho_s = 0.12 f_c^2 / f_{yt} = 0.01$

The **bar diameter (6 in.)**
\ntric ratio (larger of the following):
\n
$$
\rho_s = 0.12 f_c'/f_{yt} = 0.01
$$
\n
$$
\rho_s = 0.45 \left(\frac{A_g}{A_{ch}} - 1\right) \frac{f_c'}{f_{yt}} = 0.021
$$
\nInstructional Material Complementary FEMA 1051, Design Examples

\nFoundation Design - 90

Detailing for Ductility

**Detailing for Ductility
• Choose #5 spirals and compute required
spacing
** $\frac{d^2}{dt^2}$ spacing

\n- Choose #5 spirals and compute required spacing\n
$$
\rho_s = V_s / V_{ch} = A_s \pi d_{ch}/s\pi \left(\frac{d_c}{2}\right)^2 = \frac{4A_s}{d_{ch}s}
$$
\n
$$
s = \frac{4A_s}{d_{ch}\rho_s} = \frac{4(0.31)}{(24)0.021} = 2.5 \text{ in}
$$
\n
\n- Use pitch of 2½ inches\n
	\n- Required from the top of the pile to 7 pile
	\n- diangatore (4.7′, 6″) below the interface of the
	\n\n
\n

-
- $S = \frac{4A_S}{d_{ch}\rho_S} = \frac{4(0.31)}{(24)0.021} = 2.5$ in
th of 2½ inches
d from the top of the pile to 7 pile
rs (17'-6") below the interface of the
ple soil and the competent soil below.
Instructional Material Complementing FEMA • Choose #5 spirals and compute required
spacing
 $\rho_s = V_s/V_{ch} = A_s \pi d_{ch}/s\pi \left(\frac{d_c}{2}\right)^2 = \frac{4A_s}{d_{ch}s}$
 $s = \frac{4A_s}{d_{ch}\rho_s} = \frac{4(0.31)}{(24)0.021} = 2.5 \text{ in}$
• 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

-
- **Shear Strength
• Compute shear demand at probable moment
– Taken conservatively as 1.25 times Shear Strength

Shear Strength

- Taken conservatively as 1.25 times

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

 $V_u = 2M_{pr}/l = 2(1.25 \times 6.730 \text{ kip} - \text{in})/360 \text{ in} = 47 \text{ k}.$

- **Instructional Material Complementing FEMA 1051, Design Examples**

Instructional Material Complementing FEMA 1051, Design Examples Foundation Design 92 • Compute shear demand at probable moment

– Taken conservatively as 1.25 times

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

• Compute shear strength ignoring concrete

within plastic hinge
 within plastic hinge $\varphi V_n = (0.75)(2)(0.31)(60ksi)(0.8 \times 30)/2.5 = 232k$ • Compute shear demand at probab – Taken conservatively as 1.25 times mominal moment
 $v_u = 2M_{pr}/l = 2(1.25 \times 6.730 \text{ kip} - \text{in})/360 \text{ in}$

• Compute shear strength ignoring demotion within plastic hinge
 $\varphi V_n = (0.75)(2)(0.31$
-

Pile Section Detail

Questions

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