Bridge Design - Shallow Foundations

Course Number: CE-02-412

PDH: 10

Approved for: AK, AL, AR, GA, IA, IL, IN, KS, KY, LA, MD, ME, MI, MN, MO, MS, MT, NC, ND, NE, NH, NJ, NM, NV, OH, OK, OR, PA, SC, SD, TN, TX, UT, VA, VT, WI, WV, and WY

New Jersey Professional Competency Approval #24GP00025600
North Carolina Approved Sponsor #S-0695
Maryland Approved Provider of Continuing Professional Competency
Indiana Continuing Education Provider #CE21800088

This document is the course text. You may review this material at your leisure before or after you purchase the course. In order to obtain credit for this course, complete the following steps:

1) Log in to My Account and purchase the course. If you don't have an account, go to New User to create an account.

2) After the course has been purchased, review the technical material and then complete the quiz at your convenience.

3) A Certificate of Completion is available once you pass the exam (70% or greater). If a passing grade is not obtained, you may take the quiz as many times as necessary until a passing grade is obtained (up to one year from the purchase date).

If you have any questions or technical difficulties, please call (508) 298-4787 or email us at admin@PDH-Pro.com.

Division of Engineering Services

BRIDGE DESIGN PRACTICE

4th Edition

2015

Caltrans

State of California
Department of Transportation

CHAPTER 16
DEEP FOUNDATIONS

TABLE OF CONTENTS

16.1 INTRODUCTION 16-1

16.1.1 Types of Piles and Shafts 16-2

16.1.2 Constructability Issues 16-2

16.1.3 General Design Considerations – Pile/Shaft Group 16-3

16.2 ANALYSIS/DESIGN OF PILE/SHAFT GROUPS IN COMPETENT SOIL (DESIGN EXAMPLE) 16-8

16.2.1 Determine Pile Cap Layout and Depth 16-11

16.2.2 Determine Factored Loads for Service, Strength, and Extreme Event Limit States 16-13

16.2.3 Check Pile/Shaft Capacity 16-15

16.2.4 Design Pile Cap for Flexure 16-19

16.2.5 Design Pile Cap for Shear 16-24

16.2.6 Design Pile Cap for Joint Shear 16-28

16.2.7 Communication to Geotechnical Services 16-32

16.3 ANALYSIS/DESIGN OF SHAFT GROUPS IN SOFT/LIQUEFIABLE SOIL UNDER EXTREME EVENT I LIMIT STATE 16-34

16.3.1 Introduction 16-34

16.3.2 Caltrans Design Practice 16-34

16.3.3 Practice Bridge Geometry 16-40

16.3.4 Site Conditions and Foundation Recommendations 16-40

16.3.5 Material Properties 16-49

16.3.6 Minimum Pile-Cap Depth 16-51

16.3.7 Shaft-Group Layout 16-53

16.3.8 Seismic Forces on Shaft-Group Foundations 16-53

16.3.9 Structural Modeling of Shaft-Group Foundations 16-56

16.3.10 Inelastic Static Analysis of Shaft-Group Foundations 16-57

16.4 ANALYSIS AND DESIGN OF LARGE DIAMETER COLUMN-SHAFTS 16-63
16.4.1 Introduction 16-63
16.4.2 Design Practice 16-63
16.4.3 Lateral Stability Check of Type I and II Shafts 16-66
16.4.4 Reinforcement Spacing Requirements of Column-Shafts 16-67
16.4.5 Design Process 16-70
16.4.6 Design Example 16-72
NOTATION 16-106
REFERENCES 16-112

CHAPTER 16
DEEP FOUNDATIONS

16.1 INTRODUCTION

This chapter discusses the design practice of deep foundations, which comprises pile and shaft foundations. A pile is defined as a slender deep foundation unit, entirely or partially embedded in the ground and installed by driving, vibration, or other method. A drilled shaft is defined as a foundation unit, entirely or partially embedded in the ground, constructed by placing concrete in a drilled hole with or without steel reinforcement. Within Caltrans terminology, "pile" is often used as a general term referring to both driven piles and drilled shafts. However, the term "piles" is referred as "Driven Piles" in the AASHTO LRFD Bridge Design Specification (AASHTO, 2012). Both piles and drilled shafts develop their geotechnical capacities from the surrounding soil. Pile/shaft groups in competent soil are addressed in Sections 16.1.3 and 16.2, shaft groups in soft/liquefiable soil are addressed in Section 16.3, and column shafts (Type I and Type II) are addressed in Section 16.4.

Pile/shaft foundations can be an economical/necessary alternative to spread footings, particularly when:

(i) competent soil strata are far from original ground;

(ii) liquefaction and/or lateral-spreading potential exist;

(iii) scour depth is large;

(iv) removal of existing soil is undesirable, e.g., soil contaminated by hazardous material; or

(v) space limitations prohibit the use of spread footings.

The structural system of a pile/shaft group is an array of piles or shafts that are connected to a relatively thick reinforced concrete or composite cap and that work interactively together to support the bridge bents/piers. The forces and moments acting at the base of the bent/pier are directly transferred to the pile cap, and resulting displacements and rotations of the cap generate axial force, shear force, and bending moment in the piles/shafts. Design provisions for driven piles and drilled shafts are specified in AASHTO Articles 10.7 and 10.8, respectively, with corresponding CA Amendments (Caltrans, 2014a). Furthermore, Caltrans Memo to Designers 3-1 (Caltrans, 2014b) provides general guidance for selection and design of the piles or shafts and detailed communication procedures between the Structural Designer (SD) and the Geotechnical Designer (GD).

16.1.1 Types of Piles and Shafts

Application of different types of piles and shafts are discussed in Memo to Designers 3-1 (Caltrans, 2014b). Standard Plan Piles (Class Piles) are structurally predesigned piles or shafts mostly used in pile groups to support columns or at abutments and piers. Upper limits of structural resistance of Standard Plan (Class) Piles in compression and tension, as well as structural details, are given in the Standard Plans. The most common types of driven piles are steel H-Pile (HP) or pipe piles, precast pre-stressed concrete piles, and Cast-in-Steel Shell (CISS) piles. In selection of driven piles, environmental constraints such as acceptable limits of noise and vibration, construction constraints such as required overhead, and geotechnical condition of the soil are of importance.

Drilled shafts also known as Cast-in-Drilled Hole (CIDH) concrete piles are often recommended when:

(i) pile driving is not viable, e.g., when there is interference of pile driving with overhead power or telephone lines or nearby underground utilities;

(ii) large vertical or lateral resistance is required; and

(iii) noise and vibration mitigation plans are either not feasible or too expensive.

However, disposal of hazardous drill spoils may be costly. Drilled shafts may be used in a group similar to driven piles or as large diameter isolated shafts, that is, pile extensions and Types I/II shafts. Memo to Designers 3-1 (Caltrans, 2014b) includes provisions that improve constructability of the shaft, such as the use of temporary/permanent casing and also construction joint in Type-II shafts. For more information on isolated large diameter shafts (Type I and II shafts), refer to Section 16.4.

16.1.2 Constructability Issues

If ground water is anticipated during construction, drilled shafts must be at least 24 in. in diameter, and PVC inspection pipes should be installed to allow Gamma-Gamma Logging (GGL) or Cross-Hole Sonic Logging (CSL) test of the shafts for quality assurance mostly performed by the Foundation Testing Branch of Geotechnical Services. Memo to Designers (MTD) 3-1 (Caltrans, 2014b) illustrates requirements for proper placement of the inspection pipes. Inspection pipes are laid out by the (SD) and must be shown in the structure plans where applicable. Drilled shafts need to allow for additional concrete cover for placement of the rebar cage. Minimum cover requirements for various drilled shaft sizes are shown in Table 16.1-1. The minimum cover is not related to protection of the reinforcing steel (refer to CA Amendment (Caltrans, 2014a) Table 5.12.3-1) but rather as an aid for construction. The minimum cover allows for rebar cage deformations that occur during placement as well as for some tolerance for the final shaft and column location. For non-Standard Plan Piles, irrespective of the actual cover, only 3 in. of cover is assumed effective and used in the structural capacity calculations.

Table 16.1-1 Minimum Cover Requirement for Drilled Shafts

Diameter of Drilled Shaft, *D*	Concrete Cover
16 in. and 24 in. Standard Piles	See Standard Plan B2-3
24 in. $\leq D \leq$ 36 in.	3 in.
42 in. $\leq D \leq$ 54 in.	4 in.
60 in. $\leq D <$ 96 in.	5 in.
96 in. and larger	6 in.

For Type-II shafts use of a construction joint below the column cage will facilitate construction. The plans should show the location of the construction joint and also any permanent casing used to allow workers to prepare the joint. If the joint is more than 20 ft deep, the District should be contacted to obtain classification of the site as gassy/non-gassy from Cal-OSHA Mining and Tunneling Unit as explained in topic 110 of the Highway Design Manual and MTD 3-1 (Caltrans, 2014b).

The most common types of driven piles are steel pipe, steel HP shapes, Cast-in-Steel Shell (CISS), and precast pre-stressed concrete piles. The Structural Designer should check availability with the cost estimating branch if HP sections are to be used. Timber piles are not commonly used in Caltrans' projects unless for temporary construction.

Vibration and noise generated by pile driving should be considered from early stages of the project and when developing the Advanced Planning Study (APS). District should be consulted regarding acceptable levels of noise and vibration based on environmental, geotechnical, and structural constraints. The Project Engineer may present mitigation methods to avoid elimination of driven piles which are usually cheaper than other alternatives.

Redundancy of the steel piles, shells, and casings can affect quality assurance of welding and, therefore, impact the cost and schedule of the project. Definition of Redundant (R) and Non-redundant (N) piles is covered in the Caltrans Standard Special Provisions [49-2.02B(1)(a), 2011] and may differ from the commonly used definition of structural redundancy.

If ground water is anticipated during construction, steel casings may be used to facilitate construction of drilled shafts and to avoid caving problems. Unlike driven shells (used in CISS piles), casings can be installed by vibration or oscillations, and usually, contribution of cased portion of the shaft to geotechnical capacity is negligible. Contribution of casing to confinement or flexural strength and stiffness of the shaft may be considered in design calculations.

16.1.3 General Design Considerations – Pile/Shaft Group

Columns and piers can be supported by a foundation system consisting of a concrete cap attached to a group of piles or shafts. The structural redundancy of pile/shaft group is advantageous. However, excavation and backfill required for construction may impact its selection.

16.1.3.1 Pile/Shaft Spacing

Table 16.1-2 summarizes the current recommendations of AASHTO (AASHTO, 2012) and CA Amendments (Caltrans, 2014a) for pile/shaft spacing in a pile/shaft group, where *D* is the diameter of the pile/shaft.

Table 16.1-2 Pile/Shaft Group Spacing

Type of Piles or Shafts	Minimum center-to-center spacing of piles/shafts	Minimum spacing between face of the pile/shaft to face of the cap (for edge/corner piles/shafts)
Driven Piles	36 in. or 2*D* (whichever greater)	9 in. or 0.5*D* (whichever greater)
Drilled Shafts	2.5*D*	12 in.

The limit of 2.5*D* for drilled shafts shall not be violated for better constructability. Furthermore, use of larger spacing is recommended to avoid interference of adjacent piles/shafts and to economize geotechnical design.

16.1.3.2 Scour Protection

To avoid loss of geotechnical capacity and structural problems caused by washout of the surrounding soil, the pile cap should be deep enough to prevent pile/shaft exposure during service life of the bridge. All components of scour that is degradation, contraction, and local pier scour must be considered in design. The minimum required depth of the cap to eliminate scour problem is shown in Figure 16.1-1.

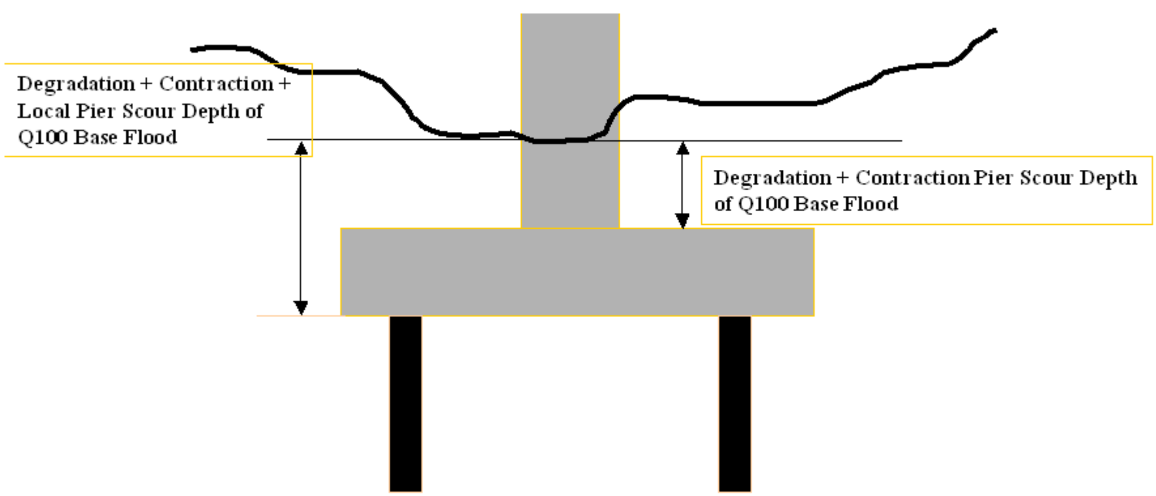


Figure 16.1-1 Required Embedment Depth for Scour Protection

When evaluating the geotechnical/structural capacity of the pile/shaft group, the combination of different components of scour should follow Table 16.1-3 (see Section 3.7.5 of CA Amendments (Caltrans, 2014a).

Table 16.1-3 Percentage of Scour Used in Design for Different Limit States

Limit State	Maximum Aggradation/Degradation and Contraction Scour to be considered for footing design, shown as a % of total	Maximum Local Scour to be considered for footing design, shown as a % of total
Service	100	100
Strength	100	50
Extreme Event	100	0

16.1.3.3 Standard (Class) Piles

Based on structural capacity, piles and shafts are classified as standard and non-standard. Standard piles, including drilled shafts and driven piles, have a pre-calculated structural capacity. Caltrans Standard Plans, Sheets B2-3, B2-5, and B2-8 provide pile details for class 90, 140, and 200 kip standard piles. Class of a pile or shaft refers to Design Compression Strength of the pile/shaft, generally used for Working Stress Design (WSD). Design Tensile Strength (WSD) for the above piles is 0.4 times design compressive strength as shown in the standard plans. The LRFD Nominal Resistance in Compression of Pile Class 90, 140 and 200 is twice the class of the pile, i.e., 180, 280, and 400 kips, respectively. The LRFD Nominal Resistance in Tension is half of the compression. Due to lack of solid information on joint performance, the pile-to-cap connection of standard piles is assumed as a pin connection.

Corrosion mitigation provisions are covered by construction specifications. Therefore, standard plans are valid for both corrosive and non-corrosive sites, except for pipe piles “Alternative W” shown on B2-5 and B2-8. Designers may use this alternative if applicable corrosion allowance is considered, and structural resistance of the reduced cross section of the pile is recalculated and checked based on design life of the structure (commonly 75 years). Standard piles must be checked and redesigned if used for seismic critical applications.

16.1.3.4 General Design Assumptions

A pile/shaft group is an indeterminate structure and is generally subjected to axial force and biaxial moment and shear. The following assumptions are commonly used in analysis of pile/shaft groups:

- Rigid Pile Cap: The pile cap can be assumed as rigid when the length-to-thickness ratio of the cantilever measured from face of the column/pier to the edge of the cap is less than or equal to 2.2 according to Seismic Design Criteria (SDC) 7.7.1.3 (Caltrans, 2013).

- Pile/Shaft-to-Cap Pin Connection: When surrounded by competent soil, the lateral movement of the piles/shafts under lateral loads such as earthquake is very small. Therefore, moments in the pile/shaft can be ignored and a pin connection can be assumed between piles/shafts and the pile cap.

These assumptions will result in a linear distribution of pile/shaft forces and facilitate analysis under lateral forces as explained in this chapter.

16.1.3.5 Analysis for Service and Strength Limit State Loads

The maximum compression ($C_{\max}$) and tension ($T_{\max}$) axial forces applied to a pile/shaft in a symmetrical group are calculated as:

$$C_{\max} = \frac{P}{N} + \left|\frac{M_x C_y}{I_x}\right| + \left|\frac{M_y C_x}{I_y}\right| \qquad (16.1.3.5\text{-}1)$$

$$T_{\max} = \frac{P}{N} - \left|\frac{M_x C_y}{I_x}\right| - \left|\frac{M_y C_x}{I_y}\right| \qquad (16.1.3.5\text{-}2)$$

where P, M_x and M_y are axial force, bending moment about x axis, and bending moment about y axis, respectively, acting at the top of pile (bottom of pile cap). N is the total number of piles/shafts, and I_x and I_y are equivalent moments of inertia of pile/shaft groups in the x and y directions calculated as:

$$I_x = \Sigma\left(N_x \times C_y^2\right) \qquad (16.1.3.5\text{-}3)$$

$$I_y = \Sigma\left(N_y \times C_x^2\right) \qquad (16.1.3.5\text{-}4)$$

In the above equations N_x and N_y are number of piles/shafts in a row parallel to x or y directions, and C_y and C_x are perpendicular distances of the row under consideration from center of gravity of the pile/shaft group, respectively. In the above equations compression is assumed positive.

16.1.3.6 Analysis for Extreme Event (Seismic) Loads

For Extreme Event-I Limit State (seismic) the pile/shaft group is analyzed under column overstrength moment (M_o) and associate shear force (V_o) acting at the base of the column and applied at all different directions. The plastic moment (M_p) at the base of the column should be calculated using fiber method analysis (for example, xSECTION analysis) and considering the seismic induced overturning effect on the column axial force for multi-column bents. The overstrength moment (M_o) is equal to $1.2M_p$. The overstrength moment and shear should be transferred to the bottom of the pile cap for pile/shaft group analysis, and therefore, the moment to be used in pile/shaft analysis will be $M_o + V_o D_f$, where D_f is the depth of the pile cap. Analysis of pile/shaft group for seismic forces depends on the type of the soil. Caltrans SDC 6.2.2 (Caltrans, 2013) classifies soil as competent, marginal, and poor. This classification is based on physical and mechanical properties of the soil, as well as possibility of seismic-associated effects such as liquefaction and lateral spreading. Pile/shaft group analysis for seismic forces in competent soil will be similar to analysis for Service or Strength limit state load combinations. The analysis of pile/shaft group in marginal or poor (soft/liquefiable) soil under Extreme Event I limit state is addressed in Section 16.3.

16.1.3.7 Design Process

MTD 3-1 (Caltrans, 2014b) lays out the design process for deep foundations. The SD provides factored loads acting on the pile/shaft for different load combinations, and the GD provides tip elevations for compression, tension, and settlement. The settlement tip is calculated based on service-I limit state loads, while compression and tension tips are calculated based on strength and extreme event limit state loads.

The factored weight of the footing (pile cap) and overburden soil should be added to the factored axial force calculated at the base of the column to provide the "gross" factored axial force. The factored weight of the soil from Original Ground (OG) to bottom of the pile cap is subtracted from factored gross axial force to obtain factored "net" axial force. Pile/shaft load calculations are based on net axial force for Service limit state and gross axial force for Strength and Extreme Event limit states.

The lateral tip elevation is provided by SD. The seismic moment and shear are applied at the cut-off point of the pile/shaft, and deflection at the cut-off point is recorded. Then, the length of the pile/shaft is changed, the deflection is recalculated, and the variation of the deflection vs. length of the pile/shaft is drawn. "Critical Depth" of the pile/shaft is the shallowest depth at which any increase in the length of the pile/shaft does not change the cut-off deflection. The critical length is used to specify "lateral tip" on the plans. A determination of the lateral tip elevation is not necessary for pile/shaft groups in competent soil. For pile/shaft groups in marginal or soft/liquifiable soil it is not necessary to use a factor of safety for determination of the lateral tip elevation. MTD 3-1 explains the design process and information to be communicated between the SD and the GD, as well as information to be shown in the Pile Data Table. Examples of different types of piles/shafts are provided in the Attachments of MTD 3-1.

16.2 ANALYSIS/DESIGN OF PILE/SHAFT GROUPS IN COMPETENT SOIL (DESIGN EXAMPLE)

The design process for a column-to-pile cap pile foundation is illustrated through the following example.

Note: A fixed column-pile cap connection was assumed for this design illustration. However, a more efficient pile cap design may utilize a pinned column-pile cap connection.

Given:

A circular column of 6 ft diameter with 26 #14 main bars and #8 hoops at 5 in. is used for a two-span post-tensioned box girder bridge. OG elevation is 48 ft, Finished Grade (FG) elevation is 48 ft, and bottom of the cap elevation is 38.75 ft. Furthermore:

- Concrete material f'_c = 3,600 psi.
- Reinforcement f_y = 60,000 psi.
- Un-factored live loads at the base of the column are listed in Table 16.2-1.
- Plastic moment and shear at the base of the column are calculated as M_p = 15,455 kip-ft and V_p = 716 kips.
- Un-factored dead load and seismic forces at the base of the column are listed in Table 16.2-2.
- Geotechnical information classifies soil as competent (SDC 7.7.1.1). Density of soil is 120 lb./ft^3
- Pile Cap rests on 30 in. CIDH piles (drilled shafts). Estimate of the shaft factored nominal geotechnical resistance is 600 kips compression and 300 kips tension. Shaft moments are assumed negligible (SDC 7.7.1.1). Shaft layout is assumed as shown in Figure 16.2-1.

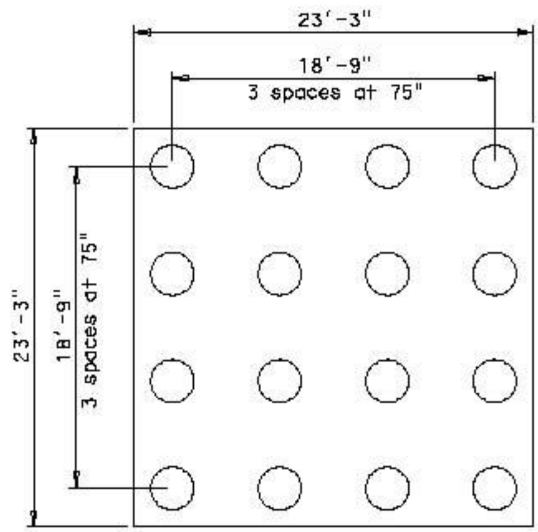


Figure 16.2-1 Shaft Group Layout

Note: When the distance between the center of applied load (column) and the supporting reactions (piles/shafts) is less than twice the depth (per AASHTO 5.6.3.1), a strut and tie model may be used. Caltrans practice allows use of simplified analysis and design of a pile cap in lieu of a strut and tie model.

- The typical section of the shaft is shown in Figure 16.2-2.

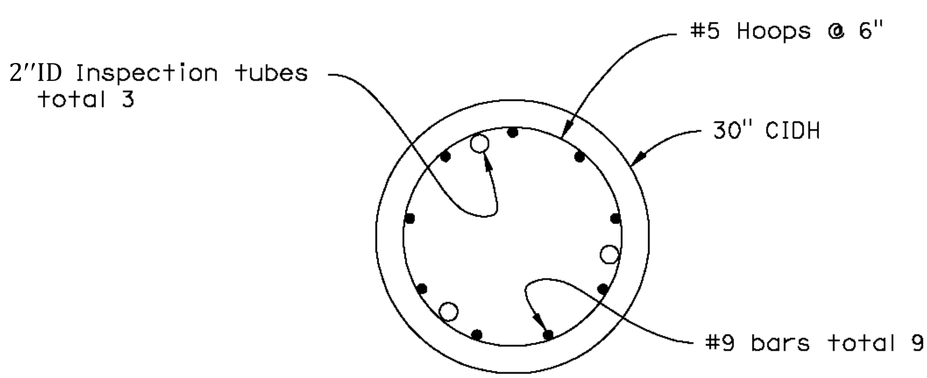


Figure 16.2-2 Shaft Details

Table 16.2-1 Un-factored Live Load Forces at Column Base

	Design Truck			Permit Truck		
Case	I	II	III	I	II	III
M_T (kip-ft)	-203.8	-39.6	-79.8	-344.1	18.7	32.2
M_L (kip-ft)	248.3	1442.2	547.1	169.1	2537.6	351.4
P (kip)	217.3	237.6	479.2	366.5	438.7	760.4
V_T (kip)	0	0	0	0	0	0
V_L(kip)	0	0	0	0	0	0

Note:
Case I: Maximum Transverse Moment (M_T) and associated effects
Case II: Maximum Longitudinal Moment (M_L) and associated effects
Case III: Maximum Axial Force (P) and associated effects

Table 16.2-2 Un-factored Dead Load and Seismic Forces Applied at the Column Base.

Un-factored, without impact Loads	M_T (kip-ft)	M_L (kip-ft)	P (kip)	V_T (kip)	V_L (kip)
DC	61.0	826.1	1164.9		
DW	11.4	167.8	227.4		
EV	0.0	0.0	*312.3*		
PS	0.0	-141.7	-20.9		
Seismic-I+	18545.8	0.0	992.0	859.0	0.0
Seismic-I-	18545.8	0.0	-992.0	859.0	0.0
Seismic-II	0.0	18545.8	0.0	0.0	859.0
Seismic-III+	13115.8	13115.8	496.0	607.0	607.0
Seismic-III-	13115.8	13115.8	-496.0	607.0	607.0

Note: Seismic forces and moments caused by overturning effects are calculated using an assumed overstrength moment and shear force of :
M_o=1.2M_p= 18,545.8 kip-ft, V_o=1.2V_p=859 kips. However, in practice, the magnitude of overstrength moment and shear depends on the applied axial force.

V_T, V_L shown in Tables 16.21-1 and 16.21-2 are forces applied at the top of the footing.

Requirements:

1. Determine pile cap layout and depth.
2. Determine LRFD factored loads for service, strength, and extreme event limit states.
3. Check pile/shaft capacity.
4. Design pile cap for flexure.
5. Design pile cap for shear.
6. Design pile cap for joint shear.

16.2.1 Determine Pile Cap Layout and Depth

A pile/shaft layout of 16 shafts (4 rows of 4 shafts) is assumed as shown in Figure 16.2-1. Per CA Amendments 10.8.1.2, the minimum spacing for CIDH piles is 2.5*D*. The minimum edge distance for CIDH piles is 12 in. clear (AASHTO 10.8.1.2). The shaft spacing is 2.5 × 30 in. = 75 in = 6.25 ft, and minimum overall pile cap width is 75 in. × 3 + (12 in. + 30 in./2) × 2 = 279 in. Pile/shaft layout and pile cap size meet minimum size and spacing criteria.

Note: If Geotechnical Services indicate group effects control, it may be warranted to increase pile/shaft spacing to raise pile/shaft tip elevations.

To ensure the full moment capacity of a column can be developed, the minimum pile cap depth is equal to the minimum clearance from the bottom of pile cap to the bottom mat of cap reinforcement, plus the bar diameters used for the bottom of pile cap reinforcement, plus the required development length of the main column reinforcement.

$D_{\mathrm{ftg,min}} = clr. + 2(d_{bd}) + l'_d$

$D_{ftg,min}$ = minimum pile cap depth.

clr. = minimum clearance from the bottom of pile cap to the bottom mat of pile cap reinforcement = 6 in. (BDD 6.71)

d_{bd} = diameters of the bars used for the bottom of pile cap reinforcement.

l'_d = required development length of the main column reinforcement.

Assuming #11 bars for the pile cap bottom reinforcement: d_{bd} = 1.63 in. (BDD 13-10)

Calculate the development length according to the specifications given by AASHTO 5.11.2.2 and AASHTO 5.11.2.4, which are shown below in section 16.2.1.1.

16.2.1.1 Development of Deformed Bars in Compression

AASHTO states:

$l_{db} \geq 0.63\,(1.693)(60) / (3.6)^{0.5} = 33.7$ in. (AASHTO 5.11.2.2.1-1)

$l_{db} \geq 0.3(1.693)(60) = 30.5$ in. (AASHTO 5.11.2.2.1-2)

AASTHO 5.11.2.2.2 states that the basic development length may be multiplied by applicable modification factors.

AASHTO 5.11.2.2.2: Reinforcement is enclosed within a spiral of not less than 0.25 in. in diameter and not more than a 4 in. pitch, modification factor = 0.75. (This reduction does not apply because we have the main column hoops at 5 in.).

Hooks shall not be considered effective in developing bars in compression. Therefore, development length required for compression is equal to 33.7 in.

16.2.1.2 Development of Standard Hooks in Tension

l_{hb} = 38.0 (1.693) / $(3.6)^{0.5}$ = 33.9 in. (AASHTO Eq. 5.11.2.4.1-1)

Basic development length shall be multiplied by applicable modification factors:

- Concrete Cover: For #11 bar and smaller, side cover (normal to plane of hook) not less than 2.5 in., and for 90 degree hook, cover on bar extension beyond hook not less than 2 in., modification factor = 0.70.

Note: For determining modification factors the specifications refer to the portion of the bar from the critical section to the bend as the "hook," and the portion of the bar from the bend to the end of the bar as the "extension beyond the hook."

- Ties or Stirrups: For #11 bar and smaller, hook enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development l_{dh} not greater than 3 d_b, where d_b is diameter of hooked bar, modification factor = 0.80.

None of the modification factors are applied, since #14 bars have been used for columns. Therefore:

Development of standard hooks in tension = 33.9 in., say 34 in. (Also greater than 8 × 1.88 in. and 6 in.)

Development length for tension (34 in.) controls over the development length for compression (33.7 in.). The required pile cap thickness is calculated as:

$D_{ftg,min} = clr. + 2(d_{bd}) + l\,ft._d$ = 6 in. + 2(1.63 in.) + 34 in. = 43.3 in.

Note: If the development length of the pile/shaft reinforcement is a concern, the pile cap depth should be similarly checked for this reinforcement.

Recommendation for balanced footing/pile cap depth is:

0.7 x D_c = 0.7 x 6.0 ft = 4.2 ft;

Use D_{ftg} = 50 in. = 4.17 ft (SDC 7.6.1-2)

Check minimum pile cap depth for rigid footing assumption. (SDC 7.7.1.3)

$L_{ftg}/D_{ftg} \leq 2.2$

where:

L_{ftg} = cantilever length of pile cap from face of column = (23.25-6.0)/2 = 8.63 ft

D_{ftg} = 50 in. = 4.17 ft

L_{ftg}/D_{ftg} = 2.06; rigid footing assumption **OK**

16.2.2 Determine Factored Loads for Service, Strength, and Extreme Event Limit States

The following three cases of live load forces should be considered in design:

Case I: Maximum Transverse Moment (M_T) and associated effects
Case II: Maximum Longitudinal Moment (M_L) and associated effects
Case III: Maximum Axial Force (P) and associated effects

Analysis results for other applicable loads acting on the pile cap are given in Table 16.2-2. Forces and moments resulting from seismic analysis in transverse, longitudinal, and 45 degree combination thereof are shown as Seismic I(+/-), Seismic II, and Seismic III(+/-). For Seismic I and Seismic III, the + represents the compression column while the – represents the tension column due to overturning forces. A combination of seismic forces should be taken at 15 degree intervals, however, for this example, one location at 45 degrees was used.

E_V was calculated in Table 16.2-2 as:

$(23.25 \text{ ft} \times 23.25 \text{ ft} - (\pi \times (6 \text{ ft})^2/4)) \times (48.0 \text{ ft} - 38.75 \text{ ft} - 4.17 \text{ ft}) \times (0.12 \text{ k/ft}^3) = 312.3 \text{ kips}$

The LRFD load combinations used in foundation design and corresponding load factors (AASHTO Table 3.4.1-1) are summarized in the following table. The upper and lower limits of permanent load factors (γ_p) are shown as *U* and *L* respectively.

Table 16.2-3 LRFD Load Factors

	DC	*DW*	*PS*	*EV*	*HL-93*	*P-15*	Seismic
Strength I-*U*	1.25	1.5	1	1.35	1.75	0	0
Strength I-*L*	0.9	0.65	1	0.9	1.75	0	0
Strength II-*U*	1.25	1.5	1	1.35	0	1.35	0
Strength II-*L*	0.9	0.65	1	0.9	0	1.35	0
Strength III-*U*	1.25	1.5	1	1.35	0	0	0
Strength III-*L*	0.9	0.65	1	0.9	0	0	0
Strength V-*U*	1.25	1.5	1	1.35	1.35	0	0
Strength V-*L*	0.9	0.65	1	0.9	1.35	0	0
Service I	1	1	1	1	1	0	0
Extreme Event I	1	1	1	1	0	0	1

The *PS* load factor of 1.0, as shown in Table 16.2-3, is recommended when the column's cracked moment of inertia is used in analysis. However, for load cases other than Extreme Event-I a load factor of 0.5 may be used; see AASHTO Table 3.4.1-3 (AASHTO, 2012).

In order to determine loads at the bottom of the pile cap, the cap size and depth will be needed. For this example a pile cap depth of 50 in. with a length and width of 23 ft - 3 in. is used.

The overall un-factored pile cap weight (DL) = 23.25 ft × 23.25 ft × 4.17 ft × 0.15 kip/ft^3 = 338 kips

The LRFD load factors are applied to axial force and moments in longitudinal and transverse directions to calculate factored loads for Strength, Service, and Extreme Event limit states, as summarized in Table 16.2-4 below. Loading shown in the table below is for live load case III only.

Table 16.2-4 Case III Maximum Axial Force

Factored Loads	M_T (kip-ft)	M_L (kip-ft)	P (kip)
Strength I-*U*	-46	2100	3459
Strength I-*L*	-77	1668	2599
Strength II-*U*	137	1617	3647
Strength II-*L*	106	1185	2787
Strength III-*U*	93	1143	2621
Strength III-*L*	62	711	1761
Strength V-*U*	-14	1881	3267
Strength V-*L*	-45	1450	2407
Service I	-7	1399	2501
Extreme Event I, Seismic I+	22128	0	3014
Extreme Event I, Seismic I-	22128	0	1030
Extreme Event I, Seismic II	0	22128	2022
Extreme Event I, Seismic III+	15645	15645	2518
Extreme Event I, Seismic III-	15645	15645	1526

Shown below are sample calculations for the factored loads shown in Table 16.2-4:

- Example 1: Calculation of axial force at the bottom of pile cap for Strength-II limit state:

  P = 1.25(1164.9) + 1.25(338) + 1.5(227.4) + 1(-20.9) + 1.35(760.4) + 1.35(312.3) = 3647 kips

- Example 2: Calculation of transverse moment at the bottom of pile cap for Seismic I+:

  M_T = 1(18545.8 + 4.17 × 859.0) = 22,128 kip-ft

- Example 3: Calculation of gross axial force at the bottom of pile cap for Service I limit state:

  P = 1(1164.9) + 1(338) + 1(227.4) + 1(-20.9) + 1(479.2) + 1 (312.3) = 2501 kips

However, the net Service I loads should be reported to Geotechnical Services as shown in the footnotes of Table 16.2-9. The net axial force is calculated by subtracting the weight of the overburden soil from gross axial force.

Soil Weight = 23.25 ft × 23.25ft × (48 ft - 38.75 ft) × 0.12k/ft ^{3} = 600 kips

P_{net} = 2501 – 600 = 1,901 kips

Note: ▭ represents loads used in Table 16.2-9, Foundation Design Data Sheet.

Similarly, the net permanent loads are to be calculated and reported to Geotechnical Services.

P = 1(1164.9) + 1(338) + 1(227.4) + 1 (-20.9) + 1(312.3) = 2,022 kips

P_{net} = 2,022 – 600 = 1,422 kips

16.2.3 Check Pile/Shaft Capacity

The general equation for moment in a rigid pile cap under seismic demand (SDC 7.7.1-2) is written as:

$$M_o^{col} + V_o^{Col} \times D_{ftg} + \Sigma M_{(i)}^{pile} - R_s \times (D_{ftg} - D_{Rs}) - \Sigma(C_{(i)}^{pile} \times C_{(i)}) - \Sigma(T_{(i)}^{pile} \times C_{(i)}) = 0$$

Since the pile cap is surrounded by competent soil, the simplified foundation model may be used; Eq.16.1.3.5-1 and Eq.16.1.3.5-2 shall apply. The moment of inertia of the pile/shaft group in both directions is calculated as:

Table 16.2-5 – Transverse Pile/Shaft Layout

	C_y (ft)	# piles/shafts	$N \times C_y^2$
Row 1	-9.38	4	352 ft^2
Row 2	-3.13	4	39 ft^2
Row 3	3.13	4	39 ft^2
Row 4	9.38	4	352 ft^2
		$I_x = \Sigma (N \times C_y^2) =$	782 ft^2

Similarly, $I_y = I_x$ = 782 ft^2

Strength and Service loads shown below in Table 16.2-6 are for Case II, which is the controlling load case for shaft loading for this example. The last two columns list maximum force (P_{max}) and minimum force (P_{min}) in the piles under the various loading conditions. Negative forces show tension in the shafts.

Table 16.2-6 – Case II Maximum Longitudinal Moment

Factored Loads: kips, ft	M_T (kip-ft)	M_L (kip-ft)	P (kip)	P_c/N (kip)	$M_x \times c_y/I_x$ (kip)	$M_y \times c_x/I_y$ (kip)	P_{max} (kip)	P_{min} (kip)
Strength I-*U*	24	3666	3036	190	0.3	44.0	234.0	145.5
Strength I-*L*	-7	3235	2176	136	-0.1	38.8	174.9	97.1
Strength II-*U*	118	4568	***3213***	201	1.4	54.8	***257.0***	144.5
Strength II-*L*	87	4136	2353	147	1.0	49.6	197.7	96.4
Strength III-*U*	93	1143	2620	164	1.1	13.7	178.6	148.9
Strength III-*L*	62	711	1761	110	0.7	8.5	119.3	100.8
Strength V-*U*	40	3089	2941	184	0.5	37.1	221.4	146.3
Strength V-*L*	9	2658	2081	130	0.1	31.9	162.1	98.1
Service I	33	2294	2259	141	0.4	27.5	169.1	113.3
Extreme Event I								
Seismic-I(+)	22128	0	3014	188	265.5	0.0	453.9	-77.2
Seismic-I(-)	22128	0	1030	64	265.5	0.0	329.9	-201.2
Seismic II	0	22128	2022	126	0.0	265.5	391.9	-139.2
Seismic III+	15645	15645	2518	157	187.7	187.7	***532.8***	-218.1
Seismic-III-	15645	15645	1526	95	187.7	187.7	470.8	***-280.1***

16.2.3.1 Check Geotechnical Requirements

The CA Amendments 10.5.5.2.4 and 10.5.5.3.3 specify strength reduction factors (ϕ) for strength and extreme event limit states as 0.7 and 1, respectively.

Compare factored loads on piles/shafts to factored resistance for strength limit state:

For compression 257 kips < (0.7) × 600 = 420 kips **OK**
For tension 0 kips < (0.7) × 300 = 210 kips **OK**

Compare factored loads on piles/shafts to factored resistance for extreme event limit state:

For compression 533 kips < (1.0) × 600 = 600 kips **OK**
For tension 280 kips < (1.0) × 300 = 300 kips **OK**

Therefore, shafts meet LRFD geotechnical requirements.

16.2.3.2 Check Pile/Shaft Structural Requirements

Strength Limit State:

- Pile/Shaft Tension Capacity
  $= \phi P_n = \phi (A_{st} \times f_y)$ (AASHTO 5.7.6.1)
  = 0.9 (9 bars × 1in.^{2}/bar) 60 ksi = 486 kips > 0 kips **OK**

Note: No tension in shafts for Strength limit state.

- Pile/Shaft Compression Capacity
  $\phi P_n = 0.75\{0.85[0.85 \times f'_c (A_g - A_{st}) + f_y A_{st}]\}$ (AASHTO 5.7.4.4)
  $= 0.75\{0.85[0.85 \times 3.6(706.9\text{-}9) + 60(9)]\} = 1706$ kips > 257 kips

  1706 kips > 257 kips **OK**

Extreme Event Limit State (Seismic):

$\phi = 0.9$; for shear (SDC 3.2.1)

- Pile/Shaft Tension Capacity
  $\phi P_n = 1$ (9 bars × 1in.^{2}/bar) 60 ksi = 540 kips > 280 kips **OK**

- Pile/Shaft Compression Capacity
  $\phi P_n = 1\{0.85[0.85 \times 3.6(706.9\text{-}9) + 60(9)]\} = 2274$ kips > 533k **OK**

16.2.3.3 Piles/Shafts Shear Capacity

Caltrans' practice is not to check the shear capacity for pile/shaft groups in competent soil. However, formal guidance material for this policy is currently not available for LRFD design. The following is an example calculation showing how the pile shear check calculation may be performed.

Ignoring the pile cap passive pressure on the front face as well as the friction on the two side faces of the pile cap, the pile/shaft shear may be conservatively approximated as the total shear divided by the number of piles/shafts. Assuming the maximum shear will occur at the top of the pile/shaft, the maximum factored shear force is as follows:

Seismic III-: $V_u = 1\,(607^2 + 607^2)^{0.5}/16 = 53.7$ kips

The structural shear capacity of a reinforced concrete pile/shaft can be calculated as follows:

$V_n = V_c + V_s + V_p \leq 0.25 f'_c b_v d_v + V_p$ (AASHTO 5.8.3.3)

$V_c = 0.0316\beta(f'_c)^{0.5}(b_v)(d_v)$

$V_s = [A_v (f_y)(d_v) (\cot\theta + \cot\alpha) \sin\alpha] / s$

$V_p = 0$ (no pre-stressing in pile/shaft)

$b_v = D = 30$ in.

$d_v = 0.9\, d_e = 0.9$ (21.8 in.) = 19.6 in.

$D_r = D - 2$ (clr + hoop d_{bd} + long $d_{bd}/2$) = 30 – 2(3 + 0.69 + 1.25/2) = 21.4 in.

$d_e = D/2 + D_r/\pi = 30/2 + 21.4/\pi = 21.8$ in. (AASHTO C5.8.2.9-2)

$\alpha = 90°$

$A_v = 0.31\text{in.}^2 \times 2 = 0.62\text{ in.}^2$, $s = 6$ in. (#5 hoops at 6 in. spacing)

Check for minimum transverse reinforcement:

$A_v \geq 0.0316\,(f'_c)^{0.5}(b_v)s/f_y = 0.0316(3.6)^{0.5}(30)(6)/60 = 0.18$ in.^{2} **OK**

N_u = 280.1 kips (shear will be controlled by maximum tensile member)

M_u = 0 (minimal moment demand assumed at top of pile/shaft)

A_{ps} = 0 (no pre-stressing steel in pile/shaft)

$\varepsilon_s = [(|M_u|/d_v) + 0.5\,N_u + |V_u - V_p| - A_{ps}\,f_{po}]/[(E_s\,A_s + E_p\,A_{ps})]$

(AASHTO 5.8.3.4.2-4)

$\varepsilon_s = [0.5\,(280.1) + |53.7|]/[(29{,}000)(4.5)] = 0.00148$

$$\beta = \frac{4.8}{1 + 750\varepsilon_s}$$ (AASHTO 5.8.3.4.2-1)

$$\beta = \frac{4.8}{1 + 750(0.00148)} = 2.27$$

$\theta = 29 + 3500\varepsilon_s$ (AASHTO 5.8.3.4.2-3)

$\theta = 29 + 3500(0.00148) = 34.2^{o}$

$A_s = 9(1\text{ in.}^2)/2 = 4.5$ in.^{2}

$V_c = 0.0316(2.27)(3.6)^{0.5}(30)(19.6) = 80.02$ kips

$V_s = 0.62(60)(19.6)[\cot(34.2°)+\cot(90°)]/6 = 178.8$ kips

$V_n = 80.02 + 178.8 = 258.8$ kips $< 0.25(3.6)(30)(19.6) = 529.2$ kips **OK**

$V_r = \phi\,V_n = 0.9\,(258.8) = 232.9$ kips > 53.7 kips **OK**

It is worth noting that a more refined analysis that accounts for the passive pressure on the front face as well as the friction on the two side faces may be warranted if the pile shear demand exceeds its structural shear capacity.

16.2.3.3 Pile/Shaft Moment Capacity

For the pile group in competent soil, due to small pile deflections, the bending moment demand has been assumed not to control, and therefore, does not require further analysis.

16.2.4 Design Pile Cap for Flexure

The critical section of the pile cap for flexure is at the face of the column as shown in Figure 16.2-3. Since the column has a circular cross-section, it is transformed into an effective square section for pile cap analysis with equivalent column width of: $(28.26)^{0.5} = 5.32$ ft

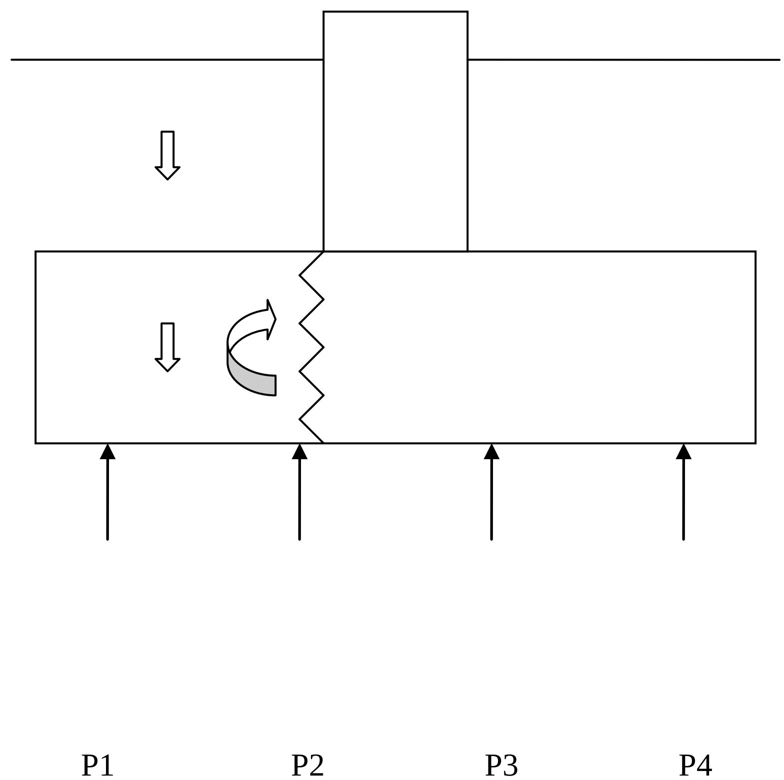


Figure 16.2-3 Pile Cap Loading

$M_{cap\ (transverse)} = (P1_{transv} \times X1_{transv}) + (P2_{transv} \times X2_{transv}) - \gamma Wft \times Xft_{transv} - \gamma Ws \times Xs_{transv}$

Due to symmetry: $X1_{transv} = X1_{long}$, $X2_{trans} = X2_{long}$, $Xft_{transv} = Xs_{transv} = Xft_{long} = Xs_{long}$

where:

$X1$ = distance from face of column to row 1 of shafts

= (6.25/2 + 6.25) – 5.32/2 = 6.715 ft

X2 = distance from face of column to row 2 of shafts

= 6.25/2 – 5.32/2 = 0.465 ft

Pi = (No. shafts per row x) × $[P/N + |M_x \times c_y/I_x|]$

Xft_{transv} = 0.5 distance from face of column to edge of pile cap

= 1/2((23.25-5.32)/2) = 4.48 ft

W_{ft} = weight of pile cap portion (from face of column to edge of cap)

= ((23.25-5.32)/2)(23.25)(50/12)(0.15) = 130.3 kips

Ws = weight of soil portion (from face of column to edge of cap)

= ((23.25-5.32)/2)(23.25)(48-38.75-4.17)(0.12) = 127.1 kips

γ = load factor, as specified in CA Amendments Tables 3.4.1-1 and 3.4.1-2

The maximum compression forces and the corresponding moment, M_{cap}, as shown in Table 16.2-6, are for Case II loading. Loading Case I and III should also be checked but are not shown here.

Table 16.2-6 Pile/Shaft Compression Forces and Corresponding M_{cap} (Strength and Service)

	Transverse			Longitudinal		
Factored Loads:	*P1* (kip)	*P2* (kip)	M_{cap} (kip-ft)	*P1* (kip)	*P2* (kip)	M_{cap} (kip-ft)
Strength I-*U*	760	759	3958	935	818	5159
Strength I-*L*	544	544	2870	699	596	3935
Strength II-*U*	809	805	4306	**1022**	876	**5774**
Strength II-*L*	592	590	3214	787	654	4549
Strength III-*U*	660	657	3235	710	673	3581
Strength III-*L*	443	441	2142	474	452	2356
Strength V-*U*	737	736	3793	884	785	4799
Strength V-*L*	521	520	2700	648	563	3574
Service I	566	565	2912	675	602	**3658**

Example Calculation: For Strength II, max *P1* force and M_{cap} in longitudinal direction:

$P1_{long}$ = (#shafts per row 1) × $[P/N + |M_x \times c_y/I_x|]$ = (4) × [3213/16 + (4568×9.38)/782]

≅ 1022 kips

γ (dead load components) = 1.25

γ (earth vertical pressure) = 1.35

M_{cap} = (1022 × 6.715) + (876 × 0.465) – 1.25(130.3)(4.48) – 1.35(127.1)(4.48)

≅ 5,774 kip-ft

In this example, tension forces were developed only under seismic loads. Maximum Compression and Tension Forces for Extreme Event I loading are shown in Table 16.2-7. The sign indicates tension forces. The corresponding M_{cap} values are also shown.

Table 16.2-7 Shaft Compression Forces and Corresponding M_{cap} (Seismic)

Factored Loads:	Transverse *P1* (kip)	Transverse *P2* (kip)	Transverse M_{cap} (kip-ft)	Longitudinal *P1* (kip)	Longitudinal *P2* (kip)	Longitudinal M_{cap} (kip-ft)
Maximum Compression						
Seismic-I(+)	**1815**	1107	**11551**	753	753	4256
Seismic-I(-)	1319	611	7990	257	257	694
Seismic II	505	505	2475	1567	859	9771
Seismic III(+)	1380	880	8524	1380	880	8524
Seismic-III(-)	1132	632	6744	1132	632	6744
Maximum Tension						
Seismic-I(+)	-309	399	-3040	753	753	4256
Seismic-I(-)	-805	-97	**-6602**	257	257	694
Seismic II	505	505	2475	-557	151	-4821
Seismic III(+)	-122	379	-1794	-122	379	-1794
Seismic-III(-)	-370	131	-3575	-370	131	-3575

Example Calculation: For Seismic-I(+), M_{cap} in transverse direction:

γ (seismic) = 1.0

$$M_{cap} = (1815 \times 6.715) + (1107 \times 0.465) - 1(130.3)(4.48) - (127.1)(4.48)$$

$$\cong 11{,}551 \text{ kip-ft}$$

Maximum moments acting on the pile cap at the face of the column for Seismic-I(+), Seismic-I(-), and Service I and are shown in Tables 16.2-6 and 16.2-7. Dividing by the 23.25 ft pile cap width, the maximum design moments and associated values per unit width are calculated as:

Strength Limit State: Strength II M_T = 185 kip-ft/ft; **M_L = *248* kip-ft/ft**

Extreme Event Limit State: Seismic-I (+) **M_T = 497 kip-ft/ft;** M_L = 183 kip-ft/ft

Extreme Event Limit State: Seismic-I (-) **M_T = -284 kip-ft/ft;** M_L = 29 kip-ft/ft

Service Limit State: M_T = 125 kip-ft/ft; **M_L = 157 kip-ft/ft**

Assuming that #9 (d_{bd} = 1.25 in.) bars with 3 in. cover and #11 (d_{bd} = 1.63 in.) bars with 6 in. cover are used for top and bottom mat reinforcement, the minimum effective depths (d_e) of the pile cap for the top and bottom mats are calculated as 45.13 in. and 41.55 in., respectively.

Critical sections for moment and shear calculations are:

- Bending moment at the face of the column (AASHTO 5.13.3.4).
- One-way shear at distance d_v from the face of the column (AASHTO 5.8.3.2).
- Two-way (punching) shear on the perimeter of a surface located at distance $d_{v,avg}$ from the face of the column (AASHTO 5.13.3.6).

where d_v is effective shear depth of the section, and $d_{v,avg}$ is the average of effective shear depths for both directions.

16.2.4.1 Pile Cap Bending Moment Check: Bottom Bars Due to Maximum Pile/Shaft Compression

Assuming 3 in. side concrete cover and using 46-#11 bars for bottom mat, the spacing of the rebar is calculated as:

s = (23.25(12)-2(3)-1.63)/(46-1) = 6 in.

The calculated spacing is less than maximum spacing of 12 in. specified in AASHTO 5.10.8, and it is acceptable.

The area of steel contributing to unit width of the pile cap is: (1.56)(12)/6=3.12 in.^{2}, therefore the depth of the concrete stress block is calculated as:

$$a = c = \frac{(3.12)(60)}{(0.85)(3.6)(12)} = 5.1 \text{ in.}$$

The bending moment capacity for non-seismic loading is computed as follows:

$M_r = \phi M_n$ = (0.9)(3.12)(60)(41.55 – 0.5 × 5.1)(1/12) = 547.6 kip-ft/ft > 248 (AASHTO 5.7.3.2) **OK**

where ϕ = 0.9 is based upon tensile controlled reinforcement (AASHTO 5.5.4.2)

$\varepsilon_t = 0.003(d_e - c)/c \cong 0.003(d_e - c)/c$ = 0.003(41.55-5.1)/5.1 = 0.0214 > 0.005

(AASHTO Fig.C5.7.2.1-1)

Therefore, the section is tensile controlled, ϕ = 0.9 **OK**

For flexural capacity under seismic loading, the moment capacity for capacity protected members is determined from expected material properties. (SDC 3.4)

where expected f'_c = 5 ksi, f_y = 68 ksi

$$a = \frac{(3.12)(68)}{(0.85)(5.0)(12)} = 4.16 \text{ in.}$$

M_{ne} = (1)(3.12)(68)(41.55 – 0.5 × 4.16)(1/12) = 697.8 kip-ft/ft > 497

OK

16.2.4.2 Pile Cap Bending Moment Check: Top Bars Due to Maximum Pile/Shaft Tension:

Assuming 3 in. side concrete cover and using 46 #9 bars for top mat, the spacing of the rebar is calculated as:

s = (23.25(12) – 2(3) – 1.25)/(46 – 1) = 6 in.

The calculated spacing is less than maximum spacing of 12 in. specified in AASHTO 5.10.8, and it is acceptable.

The area of steel contributing to unit width of the pile cap is: (1)(12)/6 = 2 in.^{2}, and therefore, the depth of the concrete stress block and resisting moment are calculated as:

$$a = \frac{(2.0)(60)}{(0.85)(3.6)(12)} = 3.27 \text{ in.}$$

Flexural capacity check is needed for seismic loading only as top reinforcement is not in tension due to strength and service loading. The moment capacity is determined from expected material properties. (SDC 3.4)

$$a = \frac{(2.0)(68)}{(0.85)(5.0)(12)} = 2.7 \text{ in.}$$

M_{ne} = (1)(2)(68)(45.13 – 0.5 × 2.7)(1/12) = 496.2 kip-ft/ft > 284 **OK**

Therefore, selected number of bars is adequate for strength, however AASHTO Eq. 5.7.3.3.2 requires minimum amount of reinforcement to be provided for crack control.

Crack control is for service load case and is neglected for top bars, as top bars are not in tension for service loading condition.

To check the crack control requirement for the bottom reinforcement (Strength II), the cracking moment ($M_{cracking}$) is calculated as smaller of M_{cr} and 1.33 M_u as follows:

Modulus of rupture = f_r = (0.24)(3.6)$^{0.5}$ = 0.455 ksi

Gross section modulus = (12)(50)^2/6 = 5000 in.^{3}/ft

$$M_{cr} = \gamma_1 \gamma_3 f_r S_x = 1.6(0.7)(0.455)(5{,}000) = 2{,}548 \frac{\text{kip-in.}}{\text{ft}} = 212 \frac{\text{kip-ft}}{\text{ft}}$$

1.33M_u = 1.33(248) = 329.8 kip-ft/ft

Therefore, M_{cr} = 212 kip-ft/ft governs, and $M_r = \phi M_n$ = 547.6 kip-ft/ft > 212

OK

AASHTO 5.7.3.4 requires maximum limits of rebar spacing for crack control.

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

Assuming γ_e = 1 (class-I exposure) and d_c = 6+(1.63+1.63/2) = 8.45 in.:

$$\beta_s = 1 + \frac{8.45}{(0.7)(50 - 8.45)} = 1.29$$

Cracked section is used to calculate tensile stress in steel reinforcement under service loads:

$$f_{ss} = \frac{(n)(M)(d - x)}{I_{cr}}$$ (Chen and Duan 2000)

E_c = 1820(3.6)$^{0.5}$ = 3453 ksi (AASHTO C5.4.2.4-1)

n = 29000/3453 = 8.40

M (maximum service moment) = M_L = 157 kip-ft

$d = d_e$ = 41.55 in.

x can be solved by a quadratic equation, where for a rectangular section:

$B = [n \times A_s + (n - 1) \times A_s']/b$

$C = 2[n \times d \times A_s + (n - 1) \times d' \times A_s']/b$

$$x = \sqrt{B^2 + C} - B = 10.9$$

$I_{cr} = (1/3) \times b \times x^3 + n \times A_s \times (d - x)^2 + (n - 1) \times A_s' \times (x - d')^2$ = 30,400 in.^{4}

Using the above information f_{ss} is calculated as:

$$f_{ss} = \frac{(8.40)(157 \times 12)(41.55 - 10.9)}{30{,}400} = 16.0\,\text{ksi}$$

The maximum spacing is checked per AASHTO Eq. 5.7.3.4-1:

$$\frac{(700)(1)}{(1.29)(16)} - (2)(8.45) = 17.0\,\text{ksi} > 6.0\,\text{in.}$$ **OK**

Therefore, 46#11 bars are acceptable for the bottom mat.

Note: For square pile caps the reinforcement will be distributed uniformly across the entire width of the cap. (AASHTO 5.13.3.5)

16.2.5 Design Pile Cap for Shear

According to AASHTO 5.13.3.6.1, both one-way and two-way shear shall be considered in pile cap design:

- The critical section for one-way action extends in a plane across the entire width and is located at a distance as specified in 5.8.3.2 (that is mostly at distance d_v from the face of the column).

- The critical section for two-way action is perpendicular to the plane of the pile cap and is located so that its perimeter b_o is a minimum but not closer than $0.5d_v$ to the perimeter of the concentrated load or reaction area.

where $d_v = d - 0.5a = 41.55 - 0.5(5.1) = 39$ in. = 3.25 ft

d_v should be greater than both $0.9d$ = 37.1 in. and $0.72h$ =36 in.

(AASTHO 5.8.2.9)

use d_v = 39 in. = 3.25 ft

The d_v value calculated above is based upon the strength loading case and is conservatively used for both the strength and seismic shear capacity calculations in this example.

16.2.5.1 Direct (One-Way) Shear

The applied one-way shear acting at distance d_v away from the face of the column will engage one shaft row. The maximum shear force V_u is 687 kips for strength and 1558 kips for the extreme event.

Strength: $V_u = 1022 - 1.35(127.1) - 1.25(130.3) = 687.4$ kips

Seismic I+: $V_u = 1815 - 1.0(127.1) - 1.0(130.3) = 1558$ kips

Note: For circular columns the distance d_v can be taken from the face of an equivalent square column.

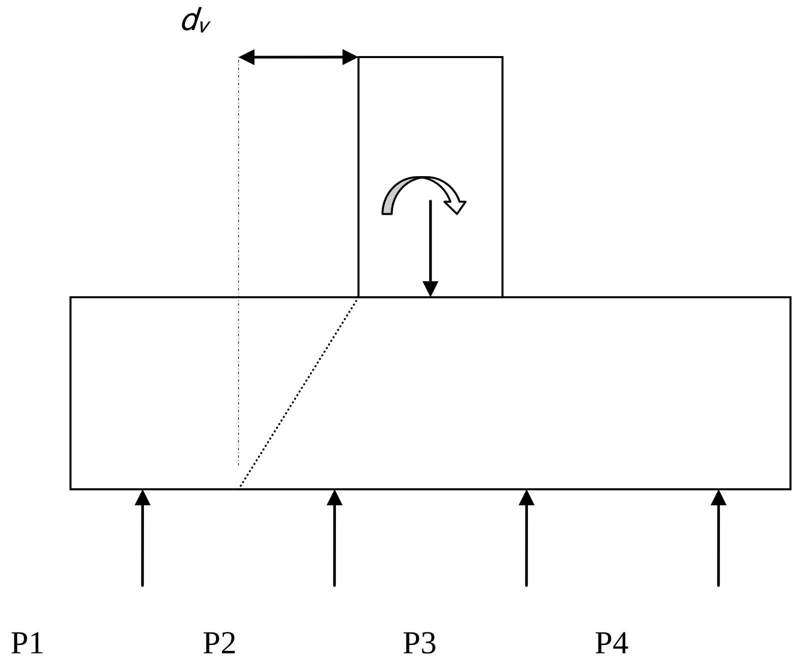


Figure 16.2-4 One-Way Shear

Therefore, use V_u= 1558 / 23.25 = 67 kip/ft

The maximum shear resistance of the section (considering shear reinforcement contribution) is limited to 0.25 $f'_c b_v d_v + V_p$ (AASHTO 5.8.3.3-2)

$V_{n,\,max}$ = 0.25(3.6)(12)(39) + 0 = 421.2 kip/ft > 67 kip/ft **OK**

Shear resistance of concrete (V_c) is $0.0316\beta(f'_c)^{0.5}b_v d_v$ (AASHTO 5.8.3.3-2)

Shear resistance of steel (V_s) is $A_v \times f_y \times d_v \times \cot\theta/s$ (AASHTO C5.8.3.3-1)

where β = 2, θ = 45.0° (AASTHO 5.8.3.4.1)

$V_c = (0.0316)(2)\sqrt{3.6}(12)(39) = 56.1 \text{ kip/ft} < 67 \text{ kip/ft}$, shear reinforcement is required.

Assuming #5 bars at 12 in. both ways:

V_s = (0.31in.^{2}/ft)(60ksi)(39 in.)(cot(45°)) / (12 in.) = 60.5 kip/ft

Factored nominal resistance of the steel and concrete:

$\phi V_n = \phi\,(V_c + V_s)$

ϕV_n = 0.9(56.1 + 60.5) = 104.94 kip/ft > 67 kip/ft **OK**

Check Minimum Transverse Reinforcement: (AASHTO 5.8.2.5)

$A_v \geq 0.0316\sqrt{f'_c}\, b_v \times s/f_y$

$0.31 \text{ in.}^2/\text{ft} \geq 0.0316\sqrt{3.6}\,(12 \text{ in.})(12 \text{ in.})/60 = 0.144 \text{ in.}^2/\text{ft}$ **OK**

Check Maximum Spacing of Transverse Reinforcement:

If $v_u < 0.125 f'_c$, then $s_{max} = 0.8\, d_v$ or 24 in. (AASHTO 5.8.2.7-1)

$v_u = V_u/[\phi\,(b_v)\,(d_v)]$

$v_u = 67$ kip/ft /[0.9 (12 in.) (39 in.)] = 0.16 ksi < (0.125 × 3.6 ksi)

$s_{max} = 0.8d_v = 0.8$ (39 in.) = 31.2 in., use $s_{max} = 24$ in. > 12 in. **OK**

16.2.5.1 Punching (Two-Way) Shear

The critical section is located at the distance of 0.5 × $d_{v,avg.}$ from face of the column as shown in Figure 16.2-5.

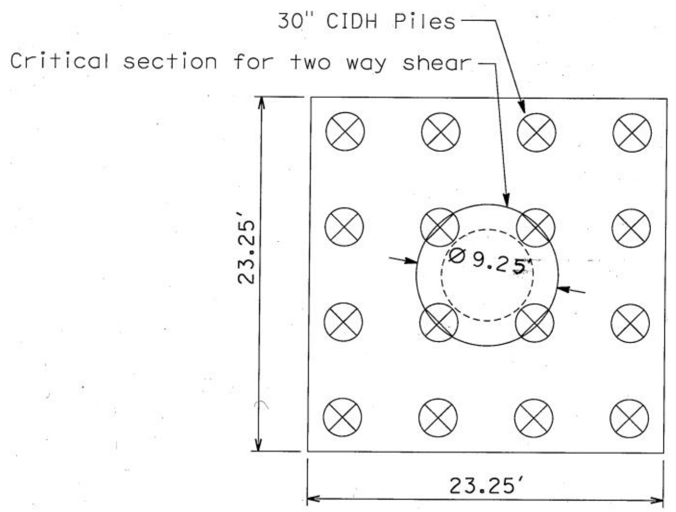


Figure 16.2-5 Pile Cap Critical Section

The actual punching shear force acting on the critical surface is calculated by subtracting the force resulting from the piles/shafts acting on critical surface from the axial force (P_u) of the column.

Determine the controlling punching shear force (P_u).

Strength II: P_u = [1.25(1164.9) + 1.5(227.4) + 1(-20.9) + 1.35(760.4)] = 2803

Seismic I+: P_u = [1(1164.9) + 1 (227.4) + 1(-20.9) + 1(992)] = 2363

Therefore, use P_u = 2803 kips, ϕ = 0.9

As the 4 shafts shown above are not fully within the effective zone, only a portion of these shafts should be removed from the punching shear force. The number of shafts within the critical surface will be approximated as 2 of 16 shafts.

$$P_{2-way} = 2803(1 - \frac{2}{16}) = 2453$$

Using conservative assumption of $d_{v,avg}$ = 39 in. = 3.25 ft, results in $b_0 = \pi$ (6 + 3.25) = 29.1 ft = 348.7 in. For 2-way action with transverse reinforcement, the nominal shear resistance shall be taken as:

$$V_n \le 0.192\sqrt{f'_c}(348.7)(39)$$

$$V_n = V_c + V_s \le 0.192\sqrt{f'_c}(b_o)(d_v)$$

$V_n \le$ 4,954 kips

$\phi V_{n\ max}$ = 0.9(4,954) = 4,459 kips

$$V_c = (0.0316)(2)\sqrt{3.6}(348.7)(39) = 1{,}631 \text{ kips} \qquad \text{(AASHTO 5.13.3.6.3-2)}$$

$$V_s = \frac{A_v \times f_y \times d_v}{s}\text{; for } \theta = 45° \qquad \text{(AASHTO 5.13.3.6.3-4)}$$

A_v = 0.31 × (348.7/12) = 9 in.^{2}

V_s = 9(60)(39)/(12) = 1755 kips

Nominal resistance of the steel and concrete:

$\phi V_n = \phi V_c + \phi V_s$ = 0.9(1631+1755) = 3,047 kips ≤ 4,459 kips

$\phi V_n \le \phi V_n$ max

3,047 kips ≤ 4,459 kips

$\phi V_n > P_{2\text{-}way}$

3,047 kips > 2,453 kips **OK**

Note: For large capacity piles, localized pile punching shear failure and the development of flexural reinforcement beyond the exterior piles should be investigated. (SDC 7.7.1.6)

16.2.6 Design Pile Cap for Joint Shear

Footing joint shear is evaluated in accordance with SDC 7.7.1.4.

Principal Compression, $p_c \le 0.25 \times f'_c = 0.25 \times (3.6) = 0.9$ ksi

Principal Tension, $\rho_t \le 12\sqrt{f'_c} = 12\sqrt{3600} = 720 \text{ psi} = 0.72 \text{ksi}$

$$p_t = \frac{f_v}{2} - \left(\left(\frac{f_v}{2}\right)^2 + v_{jv}^2\right)^{1/2}$$

$$p_c = \frac{f_v}{2} + \left(\left(\frac{f_v}{2}\right)^2 + v_{jv}^2\right)^{1/2}$$

$$f_v = \frac{P_{col}}{A_{jh}^{ftg}}$$

$$v_{jv} = \frac{T_{jv}}{B_{eff}^{ftg} \times D_{ftg}}$$

$$A_{jh}^{ftg} = (D_c + D_{ftg}) \times (B_c + D_{ftg})$$

$$B_{eff}^{ftg} = \sqrt{2} \times D_c$$

$$T_{jv} = T_c - \Sigma T_{(i)^{pile}}$$

$\Sigma T_{(i)}{}^{pile}$ = summation of the hold down force in the tension piles/shafts

Note: The column tensile force (T_c) can be determined from the xSECTION output file or CSiBridge. After determining T_c associated with M_p, multiply by 1.2 to determine T_c associated with M_o.

Check Maximum Compressive Column:

$P_c = P_u$ = [1(1164.9) + 1(227.4) + 1(-20.9) + 1(992)] = 2363 kips

For Maximum Compressive Column, T_c (at $M = M_p$) = 2578 kips (from CSiBridge)

T_c (at $M = M_o$) = 1.2 × 2578 kips = 3094 kips

Use $\Sigma T_{(i)}{}^{pile} = 0$; conservatively ignore tensile piles.

Note: If tensile piles are used, only the tensile piles within the joint shear area should be considered.

$$f_v = \frac{P_c}{A_{jh}^{ftg}} = \frac{2{,}363}{(72+50)(72+50)} = \frac{2{,}363}{14{,}884} = 0.159$$

T_{jv} = 3,094 - 0 = 3,094

v_{jv} = 3,094/(101.8 × 50) = 0.608

$$p_c = \frac{0.159}{2} + \left(\left(\frac{0.159}{2}\right)^2 + 0.608^2\right)^{1/2} = 0.69 \text{ ksi} < 0.9 \text{ ksi}$$

$$p_t = \frac{0.159}{2} - \left(\left(\frac{0.159}{2}\right)^2 + 0.608^2\right)^{1/2} = |-0.53| \text{ ksi} < 0.72 \text{ ksi}$$

Check Maximum Tensile Column:

$P_c = P_u = [1.0(1164.9) + 1.0(227.4) + 1.0(-20.9)-1.0(992)] = 379$ kips

For Maximum Tensile Column, T_c (at $M = M_p$) = 3000 kips (from CSiBridge)

T_c (at $M = M_o$) = 1.2 × 3000 kips = 3600 kips

$\Sigma T_{(i)}^{pile} = 0$ kips

$$f_v = \frac{P_c}{\left(A_{jh}^{ftg}\right)} = \frac{379}{14{,}884} = 0.025$$

$T_{jv} = 3600 - 0 = 3600$

$v_{jv} = 3600 / (101.8 \times 50) = 0.707$

$$p_c = \frac{0.025}{2} + \left(\left(\frac{0.025}{2}\right)^2 + 0.707^2\right)^{1/2} = 0.720 \text{ ksi} < 0.9 \text{ ksi}$$

$$p_t = \frac{0.025}{2} - \left(\left(\frac{0.025}{2}\right)^2 + 0.707^2\right)^{1/2} = \left|-0.694\right| \text{ ksi} < 0.72 \text{ksi}$$

If $\rho > 3.5\sqrt{f'_c}$ then T heads are required in stirrups: (SDC 7.7.1.7)

$$3.5\sqrt{f'_c} = (3.5)\sqrt{3600} = 210 \text{ psi} = 0.21 \text{ksi}$$

Use $p_t = 0.694$ ksi > 0.21 ksi **NG**

Therefore, *T* headed stirrups are required in pile cap to account for joint shear effects. The pile cap region within $D_c/2$ from the face of the column should have *T* heads at the bottom of stirrups.

See the pile cap reinforcement detail, Figure 16.2-6, for *T* headed stirrup locations.

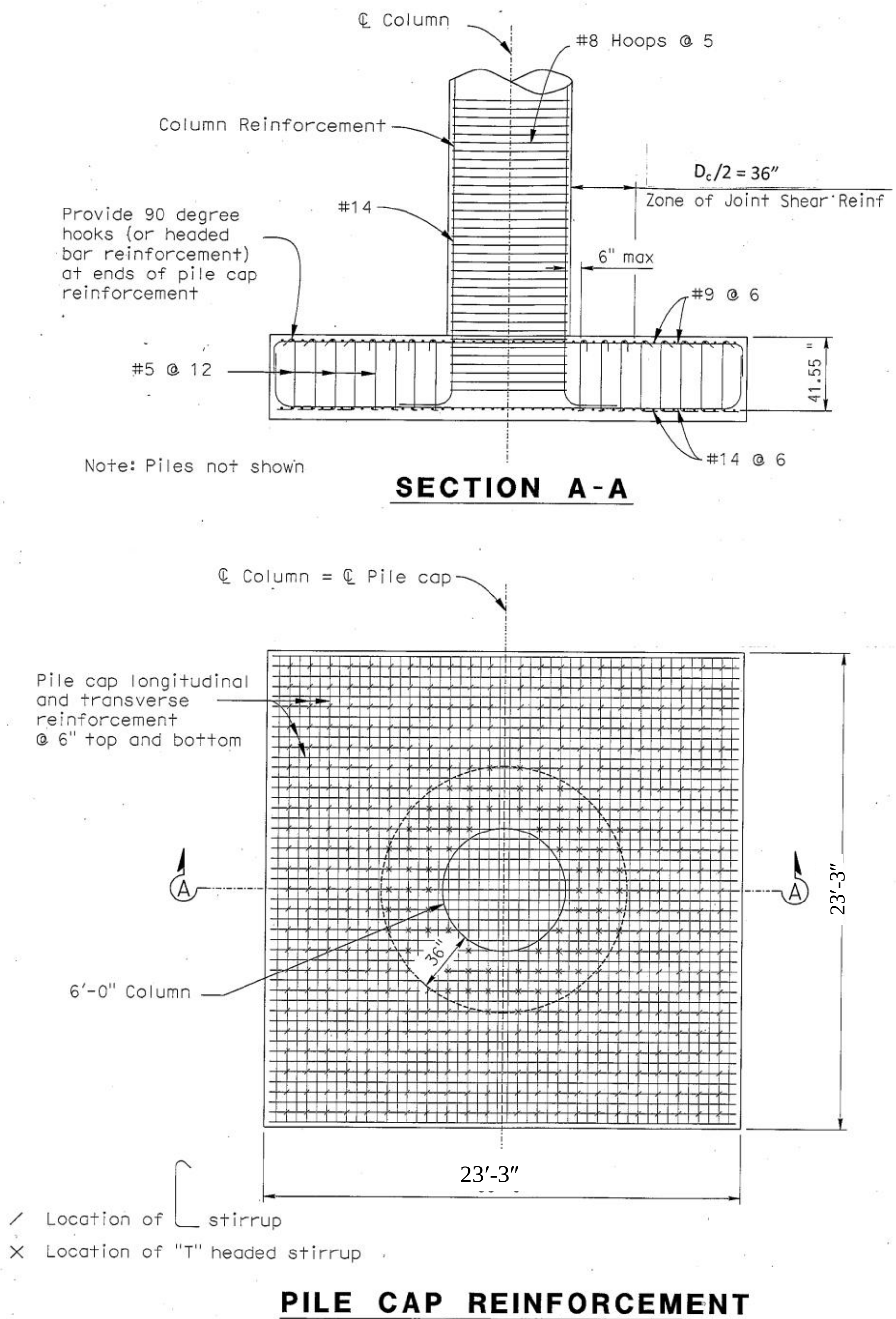


Figure 16.2-6 Reinforcement Layout

Note: Fully lapped stirrups with 180 degree hooks at opposite ends may be used in lieu of *T*-headed stirrups (SDC 7.7.1.7). For other seismic design and detailing requirements, refer to Caltrans’ Seismic Design Criteria (Caltrans, 2013).

16.2.7 Communication to Geotechnical Services

The following attachments provide examples of communication processes between the Structural Designer and Geotechnical Services Refer to MTD 3-1 (2014b) and MTD 1-35 (2008).

Attachment-I:

Example of Preliminary Information to be sent from Structural Designer to Geotechnical Services:

Table 16.2-8 Preliminary Foundation Design Data Sheet

Support No.	Foundation Type(s) Considered	Estimate of Maximum Factored Compression Loads (kips)
Abut 1	Spread Footing	
Bent 2	30 in. CIDH Piles	3500
Abut 3	Spread Footing	

Attachment-II:

General Foundation and Load Information to be sent from Structural Designer to GD for LRFD Strength and Extreme Event Limit States Load Data:

Table 16.2-9 Foundation Design Data Sheet

Foundation Design Data Sheet								
Support No.	Design Method	Pile Type	Finished Grade Elevation (ft)	Cut-off Elevation (ft)	Pile Cap Size (ft)		Permissible Settlement under Service Load (in.)*	Number of Piles per Support
					B	L		
Abut 1	WSD						1.0 or 2.0	
Bent 2	LRFD	30 in. CIDH	48	39	23.25	23.25	1.0	16
Abut 3	WSD						1.0 or 2.0	

**Note*: Based on Caltrans' current practice, the total permissible settlement is 1 in. for multi-span structures with continuous spans or multi-column bents, 1 in. for single span structures with diaphragm abutments, and 2 in. for single span structures with seat abutments. Different permissible settlement under service loads may be allowed if a structural analysis verifies that required level of serviceability is met.

Table 16.2-10 Foundation Design Loads

Foundation Design Loads											
Support No.	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Abut 1				N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Bent 2	1901	N/A	1422	3647	257	0	0	3014	533	0	280
Abut 3				N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

Note: For Geotechnical Services:

- Support loads shown are per column.
- Service I loads are reported as net loads.
- Strength and Extreme loads are reported as gross loads.

Loads from Table 16.2-10 are shown in bold and highlighted in Sections 16.2.2 and 16.2.3.

Load tables may be modified to submit multiple lines of critical load combinations for each limit state, if necessary.

16.3 ANALYSIS/DESIGN OF SHAFT GROUPS IN SOFT/LIQUEFIABLE SOIL UNDER EXTREME EVENT I LIMIT STATE

16.3.1 Introduction

The behavior of a pile/shaft group depends on the characteristics of its surrounding soil. This problem pertains to a class of Soil-Structure Interaction (SSI) problems. The lateral behavior of a pile/shaft group is governed by the soil near the ground surface, whereas its axial behavior is governed by the soil at a deeper depth. These two behaviors are practically independent of each other (Parkers and Reese, 1971). The axial behavior of a pile/shaft group embedded in a soft/liquefiable soil is similar to that embedded in a competent soil; whereas its lateral behavior depends on the type of soil in which it is embedded. For a pile/shaft group embedded in a competent soil, the soil near the ground surface provides substantial soil resistance to the lateral displacement of the pile cap, which results in small displacement and moment demands in the piles/shafts. For a pile/shaft group embedded in a soft/liquefiable soil, the majority of this soil resistance is lost, which results in large displacement and moment demands in the piles/shafts. The main objective of this section is to illustrate the analysis and design of shaft groups in soft/liquefiable soil under Extreme Event I Limit State (seismic loading).

16.3.2 Caltrans Design Practice

Foundation components of Ordinary Standard Bridges shall be designed to remain essentially elastic when resisting the column's overstrength moment, the associated overstrength shear, and the axial force at the base of the column (Caltrans, 2013). Bridge features that lead to complex response during seismic events, *e.g.*, irregular geometry, unusual framing, and/or unusual geologic conditions, are considered non-standard. The current Caltrans design practice for Ordinary Non-Standard Bridges is that the formation of plastic hinges in piles/shafts is not desirable, and piles/shafts should remain elastic during the design seismic hazard. For a soft or liquefiable soil (unusual geologic conditions), this might be uneconomical due to the excessive curvature demand imposed on the piles/shafts. Project-specific design criteria may permit plastic hinging at the top of the piles/shafts with a maximum displacement ductility demand of 2.5 (estimated at the bottom of the pile cap); the formation of a second set of plastic hinges at some distance below the bottom of the pile cap shall not be permitted (Caltrans, 2013). Shaft groups with permitted plastic hinging at the top of the shafts should be designed to meet the performance and strength criteria described in the following subsections.

16.3.2.1 Shaft-Group Foundation Performance Criteria

16.3.2.1.1 Demand Ductility Criteria (SDC 4.1.2)

The displacement ductility demand of a shaft group, μ_D, is defined as $\mu_D = \Delta_D / \Delta_Y$, where Δ_D is the global displacement demand of the shaft group and Δ_Y is the yield displacement of the shaft group from its initial position to the formation of the first plastic hinge in the shafts (SDC 2.2.3). Shaft groups with permitted plastic hinging at the top of the shafts shall have a maximum displacement ductility demand of 2.5.

For ordinary standard bridges, the global displacement demand, Δ_D, is typically estimated using a linear elastic analysis (equivalent static or dynamic) of the bridge assuming effective section properties for ductile members (SDC 2.2, 5.2, and 5.6). For a shaft-group foundation, however, the forces transferred to the foundation are limited by the overstrength moment capacity of the column, M_o^{col}. The global displacement demand of a shaft group, Δ_D, shall therefore be defined as the lateral displacement (measured at the bottom of the pile cap) resulting from the application of the column's overstrength moment and associated overstrength shear at the base of the column: see Figure 16.3-1.

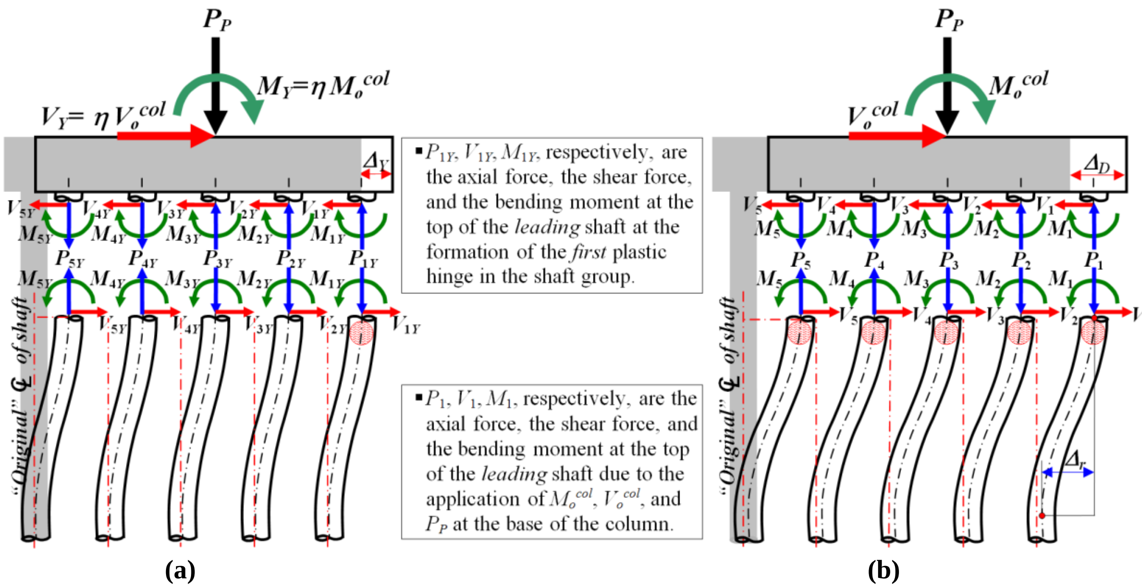


Figure 16.3-1 Schematic Views of a Shaft-Group Foundation with Permitted Plastic Hinging for Two Loading Cases

In the above figure: (a) corresponds to the formation of the first plastic hinge at the top of a shaft, which occurs at a lateral force $V_Y = \eta V_o^{col}$ and bending moment $M_Y = \eta M_o^{col}$, where η is a constant (for shafts with permitted plastic hinging $\eta \le 1$ and for elastic shafts $\eta > 1$); and (b) corresponds to the application of the columns' overstrength moment and associated overstrength shear at the base of the column.

16.3.2.1.2 Capacity Ductility Criteria (SDC 4.1.3)

The local displacement ductility capacity of an isolated shaft within a shaft group μ_c is defined as $\mu_c = \Delta_c / \Delta_y^{shaft}$, where Δ_y^{shaft} is the idealized yield displacement of the shaft at the formation of the first plastic hinge (SDC 3.1.4.1), and Δ_c is the local displacement capacity of the shaft at its collapse limit state. The value of Δ_c is calculated for an equivalent member that approximates a guided-guided column (SDC 3.1.3, 3.1.4, and 3.1.4.1). See Figure 16.3-2.

$\Delta_{c1} = \Delta_{y1}^{shaft} + \Delta_{p1}$; $\Delta_{c2} = \Delta_{y2}^{shaft} + \Delta_{p2}$; (SDC 3.1.3-1)

$\Delta_{y1}^{shaft} = \phi_{Y1} \times L_1^2/3$; $\Delta_{y2}^{shaft} = \phi_{Y2} \times L_2^2/3$; (SDC 3.1.3-7)

$\Delta_{p1} = \theta_{p1} \times (L_1 - L_{p1}/2)$; $\Delta_{p2} = \theta_{p2} \times (L_2 - L_{p2}/2)$; (SDC 3.1.3-8)

$\theta_{p1} = \phi_{p1} \times L_{p1}$; $\theta_{p2} = \phi_{p2} \times L_{p2}$; (SDC 3.1.3-9)

$\mu_{c1} = \Delta_{c1}/\Delta_{y1}^{shaft}$; $\mu_{c2} = \Delta_{c2}/\Delta_{y2}^{shaft}$; (SDC 3.1.4-2)

where L_1, L_2 are the distances from the two points of maximum moments to the point of contra-flexure (assumed equal, *i.e.*, $L_1 \cong L_2 = L/2$); L is the distance between the two points of maximum moments; ϕ_{Y1}, ϕ_{Y2} are the idealized yield curvatures of the top and lower plastic hinges, respectively (assumed equal, *i.e.*, $\phi_{Y1} \cong \phi_{Y2}$); ϕ_{p1}, ϕ_{p2} are the idealized plastic curvatures of the top and lower plastic hinges, respectively (assumed equal, *i.e.*, $\phi_{p1} \cong \phi_{p2}$); Δ_{p1}, Δ_{p2} are the idealized local plastic displacement capacities due to the rotations of the top and lower plastic hinges, respectively; θ_{p1}, θ_{p2} are the plastic rotation capacities of the top and lower plastic hinges, respectively; L_{p1}, L_{p2} are the equivalent analytical lengths of the top and lower plastic hinges, respectively. The top plastic hinge of a shaft within a shaft group is analogous to that of a column, whereas the lower plastic hinge of a shaft is analogous to that of a non-cased type I pile shaft; therefore, L_{p1}, L_{p2} are given by:

$L_{p1} = 0.08\, L_1 + 0.15\, f_{ye}\, d_{bl} \geq 0.3\, f_{ye}\, d_{bl}$ (SDC 7.6.2.1-1)

$L_{p2} = D + 0.08\, L_2$, (Analogous to SDC 7.6.2.3-1 for non-cased type I shaft)

where d_{bl} and f_{ye} are the nominal diameter and the expected yield stress of the longitudinal reinforcement of the shaft, respectively, and D is the shaft diameter.

Individual shafts within a shaft group shall have a minimum local displacement ductility capacity of 3 (*i.e.*, $\mu_{c1} \geq 3$; $\mu_{c2} \geq 3$) to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to them (SDC 3.1.4.1).

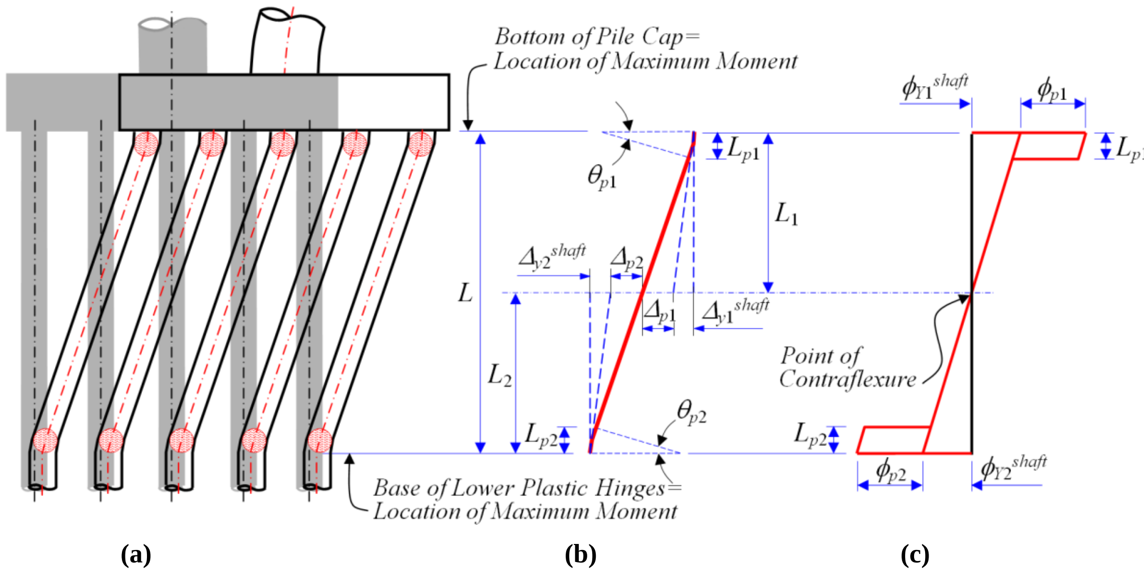


Figure 16.3-2 Plastic Hinges of a Shaft-Group Foundation

In the above figure: (a) corresponds to a schematic view of a shaft-group foundation subjected to lateral loading near collapse; (b) corresponds to a segment of a shaft between the two points of maximum moment idealized as a guided-guided column; and (c) corresponds to an idealized curvature diagram of the shaft segment shown in (b).

16.3.2.1.3 Global Displacement Criteria (SDC 4.1.1)

The global displacement demand of a shaft group, Δ_D, shall be less than its global displacement capacity, Δ_C: see Figure 16.3-3. The global displacement capacity of a shaft group, Δ_C, is defined as the lateral displacement (measured at the bottom of the pile cap) corresponding to the first plastic hinge reaching its plastic rotation capacity (SDC 3.1.3).

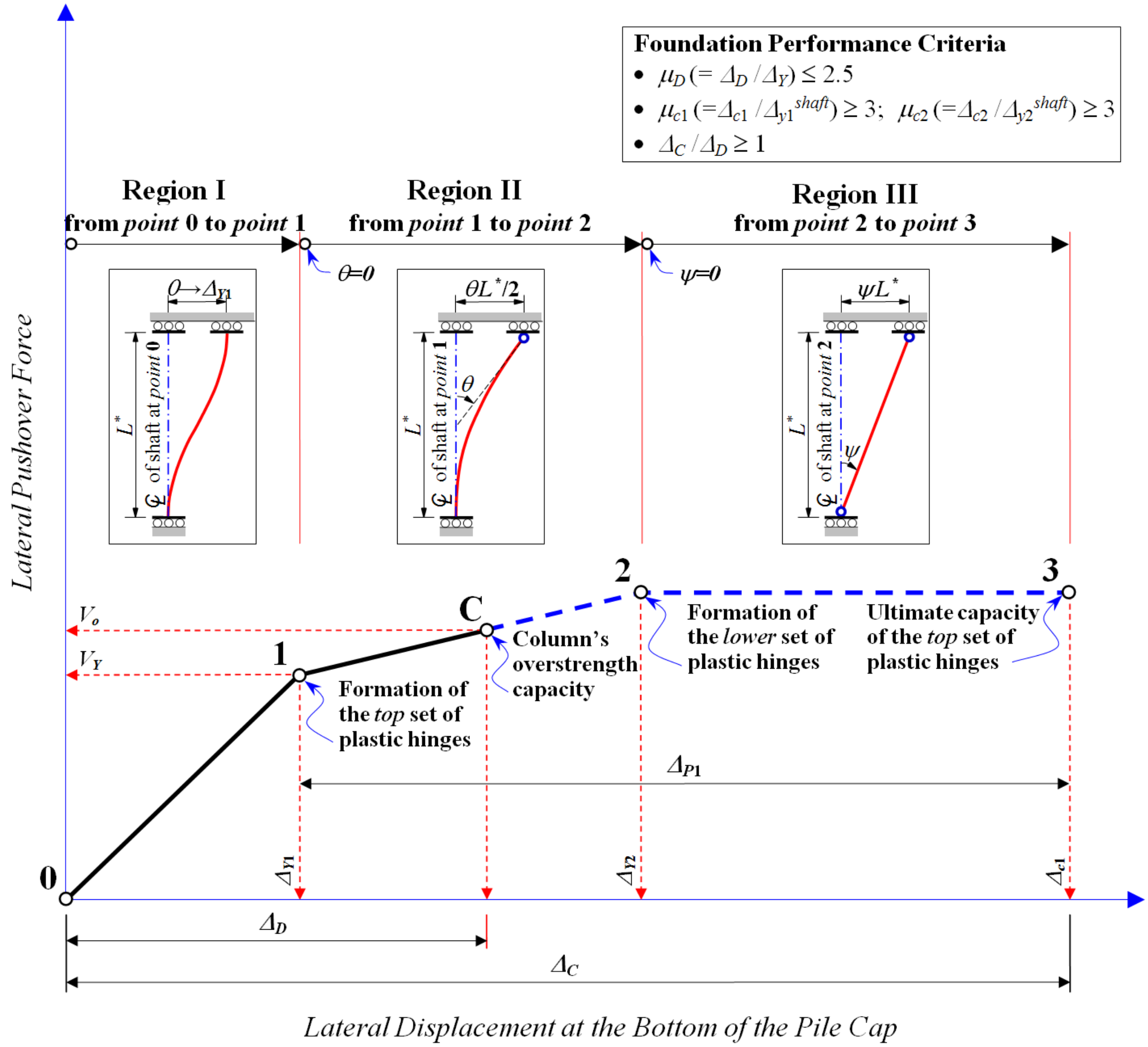


Figure 16.3-3 An Idealized Force-Deflection Curve of a Shaft Group in Soft/Liquefiable Soil

Note: The models shown in the inset diagrams approximately depict the behavior of the shaft group, assuming that the liquefied-soil stiffness is negligibly small.

Figure 16.3-3 shows a schematic plot of the idealized force-deflection curve (solid lines) of a shaft group in soft/liquefiable soil. The broken lines correspond to a hypothetical case, where the column is retrofitted such that its plastic moment capacity is sufficiently large to force the formation of the lower set of plastic hinges. The lateral behavior of a shaft group can be divided into three distinct regions. Region I, from point 0 to point 1, where the lateral behavior is approximated by a guided-guided shaft, with $L^* = L$; Region II, from point 1 to point 2, where the lateral behavior is approximated by a guided-free shaft (assuming that the top set of plastic hinges form simultaneously). The additional lateral displacement (measured at the bottom of the pile cap) beyond point 1 is ($\theta\ L^*/2$), where $L^* = L\text{-}Lp_1/2$, and θ (rad) is the rotation of the top of the shafts beyond point 1 (θ = 0 at point 1); Region III, from point 2 to point 3, where the lateral behavior is approximated by a pinned-free shaft (assuming that the lower set of plastic hinges form simultaneously). The additional lateral displacement (measured at the bottom of the pile cap) beyond point 2 is ($\psi\ L^*$), where $L^* = L - Lp_1/2 - Lp_2/2$, and ψ (rad) is the additional rotation of the top of the shafts beyond point 2. The points 0, 1, 2, and 3 shown in Figure 16.3-3 identify the boundaries of these three regions.

The global displacement capacity of a shaft group is given by $\Delta_C = \Delta_{Y1} + \Delta_{P1}$, where Δ_{Y1} is the global yield displacement of the shaft group from its initial position to the formation of the first plastic hinge, and Δ_{P1} is the global plastic displacement of the shaft group corresponding to the first plastic hinge reaching its plastic rotation capacity. The value of Δ_{Y1} is obtained from the inelastic static analysis of the shaft group (discussed in Section 16.3.10 in this Chapter). The value of Δ_{P1} can be estimated based on the model describing the behavior of the shaft group in region II, *i.e.*, $\Delta_{P1} = \theta_{p1}\ (L - L_{p1}/2)/2$.

It is worth noting that point 3 shown in Figure 16.3-3 can fall either before or after point 2 (corresponding to the formation of the lower set of plastic hinges). The expression $\Delta_{P1} = \theta_{p1}\ (L - L_{p1}/2)/2$ is derived for the case where point 3 falls before point 2. If point 3 falls after point 2, both region II and region III contribute to the expression for Δ_{P1}. The expression $\Delta_{P1} = \theta_{p1}\ (L - L_{p1}/2)/2$, however, can still be conservatively used to verify that $(\Delta_C / \Delta_D) > 1$. The reason here is that point 2 has to fall after point C since the formation of any of the lower plastic hinges is not permitted before the column reaches its overstrength capacity; and subsequently, point 3 has to fall after point C, *i.e.*, $(\Delta_C / \Delta_D) > 1$. The use of the expression $\Delta_{P1} = \theta_{p1}\ (L - L_{p1}/2)/2$ simplifies the analysis since it eliminates the need for tracking the formation of the lower set of plastic hinges.

16.3.2.2 Shaft-Group Foundation Strength Criteria

16.3.2.2.1 Minimum Lateral Strength (SDC 3.5)

Shaft groups with permitted plastic hinging shall have a minimum lateral flexural capacity (based on the expected material properties) to resist a lateral force V_Y of not less than 10% of the dead load on the shaft group P_P, *i.e.*, $V_Y / P_P \geq 0.1$, where V_Y corresponds to the formation of the first plastic hinge in the shaft group: see Figure 16.3-1(a).

16.3.2.2.2 P-Δ Effects (SDC 4.2)

The Caltrans Seismic Design Criteria has established a conservative limit for lateral displacements induced by axial loads for columns meeting the ductility demand limits. This limit for columns shall be adopted for shafts within a shaft group since the lateral soil springs of a liquefied layer are most likely yielded before the formation of plastic hinges at the top of the shafts. For a ductile shaft approximated as a guided-guided column, this limit is defined by:

$P_{dl} \times \Delta_r/2 \leq 0.20 \times M_p^{shaft}$,

where P_{dl} is the axial force in an individual shaft attributed to dead load (with no overturning effects); M_p^{shaft} is the idealized plastic moment capacity of the shaft associated with P_{dl}; Δ_r is the relative lateral offset (of the displacement demand) between the top and the lower points of maximum moment along the shaft: see Figure 16.3-1(b).

16.3.2.2.3 Force Demands (SDC 6.2.3.1)

Foundation elements shall be designed to resist the column's overstrength moment, M_o^{col}, the associated overstrength shear, V_o^{col}, and the axial load, P_P. The moment and the shear demands for the pile cap and the shafts shall be determined from a static analysis of the shaft-group foundation under the actions of M_o^{col}, V_o^{col}, and P_P at the base of the column.

The capacity of concrete components to resist all seismic force demands, except for shear, shall be based on the expected material properties (SDC 3.2.1) with a strength reduction factor of $\phi = 1$. The seismic shear capacity of all concrete components (ductile and capacity-protected) shall be conservatively based on the nominal concrete and steel strengths (SDC 3.2.1) with a strength reduction factor of $\phi = 0.9$.

For shaft-group foundations with permitted plastic hinging, verifying the moment demand of the ductile shafts is not required, since the moment demand shall not exceed the plastic moment capacity of the shafts. The moment demand of the capacity-protected pile cap shall not exceed its expected nominal moment capacity, M_{ne} (SDC 3.4).

16.3.3 Practice Bridge Geometry

Figure 16.3-4 shows schematically the elevation and the typical section of a four-span cast-in-place pre-stressed concrete box-girder bridge having a total length of 525 ft (Yashinsky et al., 2007). The bridge crosses a dry-bed canyon at zero skew. The bridge is supported on two seat-type abutments on pile foundations, and three single-column piers on shaft-group foundations. Each shaft-group foundation consists of a 25 × 25 × 4.5 ft pile cap on a 5 × 5 array of 24-in. diameter, 60-ft long drilled shafts spaced at 5.25 ft in both directions. The length of the drilled shafts is limited to 30 times the shaft diameter to ensure constructability and quality control (Caltrans, 2014b).

16.3.4 Site Conditions and Foundation Recommendations

Figure 16.3-5 shows the idealized soil profile at Pier 3, which consists of a 24-ft thick layer of medium dense sand underlain by a 60-ft thick layer of dense sand. Groundwater was encountered at a depth of 9 ft. The estimated future long-term scour (degradation and contraction) elevation is 91.5 ft. The potential local pier scour depth corresponding to the 100-year base flood shall not be considered in the Extreme Event I Limit State (seismic): see CA Amendments (Caltrans, 2014a) Table 3.7.5-1. The 15 ft thick layer of saturated medium dense sand below the pile cap is determined to be liquefiable under the design seismic hazard. Figure 16.3-6 shows the Acceleration Response Spectrum curve for this project site.

It should be noted that the project site is underlain by cohesionless soil layers and shallow groundwater; therefore, the use of 16-in. diameter CIDH piles is not recommended due to potential construction difficulties (*e.g.*, caving of sand into the drilled hole, and the difficulties associated with the cleaning and inspection of the bottom of the drilled hole). Thus, only 24-in. (or larger) diameter CIDH piles should be used (Caltrans, 2014b).

A series of analyses is performed on the 24-in. diameter shafts selected for this practice bridge in order to provide the necessary geotechnical information for foundation design (Malek and Islam, 2010)

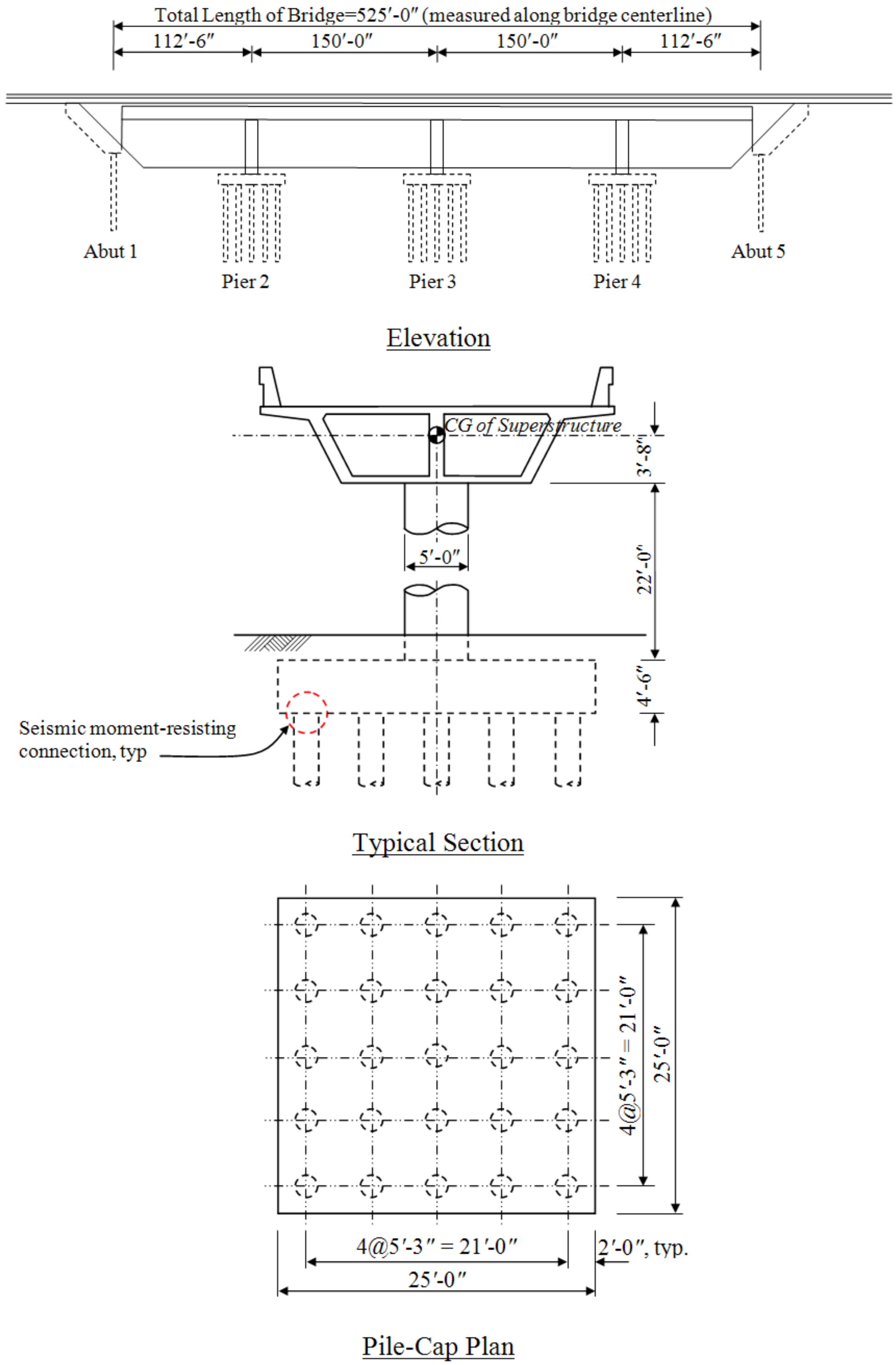


Figure 16.3-4 Geometry of the Practice Bridge

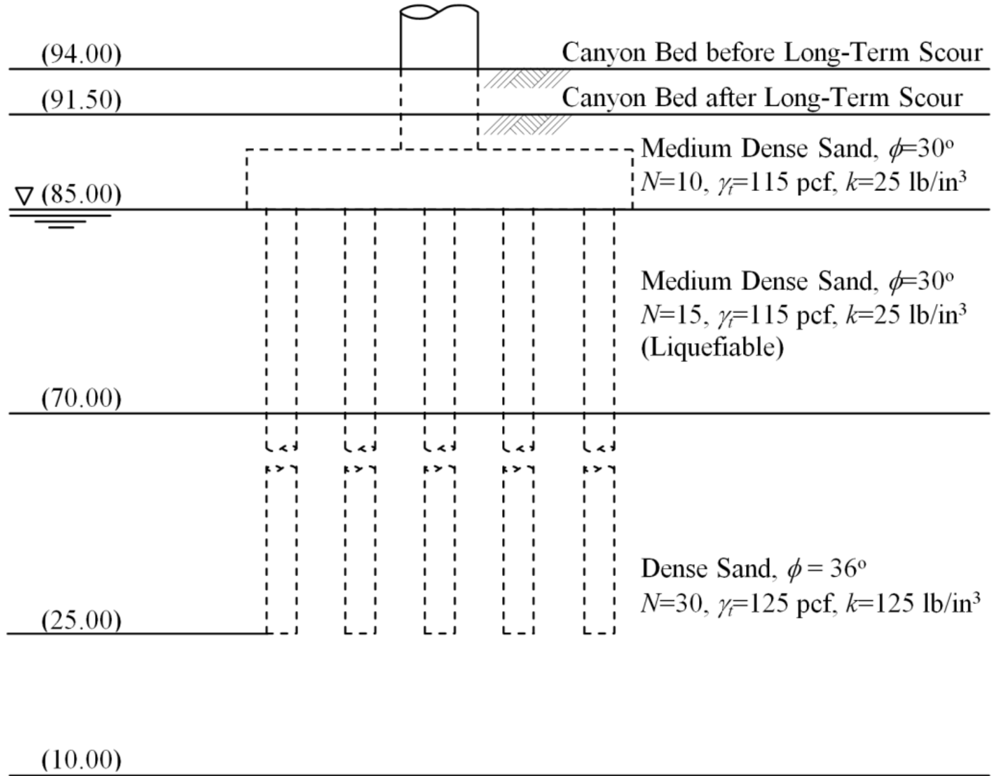


Figure 16.3-5 Idealized Soil Profile for the Project Site

The following notation is used in the idealized soil profile: *N* is the standard blow count, γ_t (lb./ft^3) is the unit weight of sand, ϕ (degrees) is the friction angle, and k (lb./in.^{3}) is a soil modulus parameter for sand.

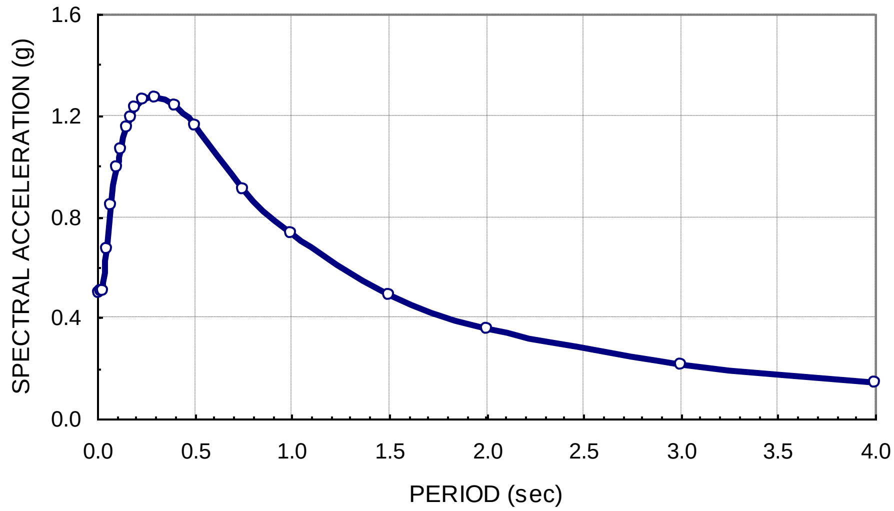


Figure 16.3-6 Acceleration Response Spectrum Curve for the Project Site

The computer program LPILE (Reese *et al.*, 2007) is used to develop the soil lateral reaction versus the lateral deflection (*p*-*y*) curves (also known as lateral soil springs). The *p*-*y* curves are generally non-linear and vary along the length of shafts. The soil lateral reaction, *p*, is the lateral load per unit length of a given diameter shaft and is obtained by integrating the unit lateral stress around the shaft. The force *p* acts in the opposite direction of the lateral deflection *y*: see Figure 16.3-7. Several methods are used by Geotechnical Services to develop the *p*-*y* curves for the liquefied soil layer. A preliminary analysis has indicated that the *p-reduction factor* method is the appropriate method for this project. In this method, the *p*-*y* curve for a liquefied soil layer is obtained by reducing the *p-values* of the corresponding non-liquefied soil layer using a reduction factor, which depends on the density of the liquefied soil layer. For the idealized soil profile shown in Figure 16.3-5, a *p-reduction factor* of 0.1 is selected since the liquefied soil layer has a density that falls within the lower range of the medium density. The recommended *p*-*y* curves for an individual 24-in. diameter shaft installed in the soil profile shown in Figure 16.3-5 are presented in Figure 16.3-8.

It is worth mentioning that the lateral displacement of a pile cap mobilizes the soil in front of it. This results in a build-up of passive pressure, which depends on the amount of lateral displacement of the pile cap. This passive pressure is typically provided by Geotechnical Services in the form of a passive resistance-displacement relationship. The designer, however, should evaluate the amount of displacement required to produce the ultimate passive resistance of the soil. For this practice bridge, the 6.5-ft thick soil layer overlain on the liquefiable layer is relatively thin and likely to crack during liquefaction; therefore, the pile cap passive pressure is negligible.

The *t-z method* (a well-known method of soil-structures interaction) is used by Geotechnical Services to develop the load transfer curves for the axial side resistance (*t-z* curves) and the end bearing resistance (*Q-z* curve). The *t-z* curves are a set of non-linear curves, which vary along the length of a shaft and represent the axial load transfer per unit length of a given diameter shaft as a function of the vertical shaft displacement at the corresponding depth. All soil within and above the liquefiable zone shall be considered not to contribute to axial compressive resistance (AASHTO 10.7.4). The *Q-z* curve is a non-linear curve representing the axial soil reaction in end bearing as a function of the axial displacement at the shaft tip: see Figure 16.3-9. The recommended *t-z* and *Q-z* curves for an individual 24-in. diameter shaft installed in the soil profile shown in Figure 16.3-5 are presented in Figures 16.3-10 and 16.3-11, respectively.

It should be noted that both the tip resistance (end bearing) and the side resistance (skin friction in cohesionless soil or adhesion in cohesive soil) develop in response to the vertical displacement of the shaft. However, the peak value for the side resistance typically occurs at a smaller vertical displacement than the peak value for the tip resistance. Geotechnical Services usually discards the tip resistance for service and strength limit states, particularly in wet conditions, where soft compressible drill spoils and questionable concrete quality are both possible at the tip of the shaft. For extreme event I limit state, however, Geotechnical Services may include some fraction of the tip resistance.

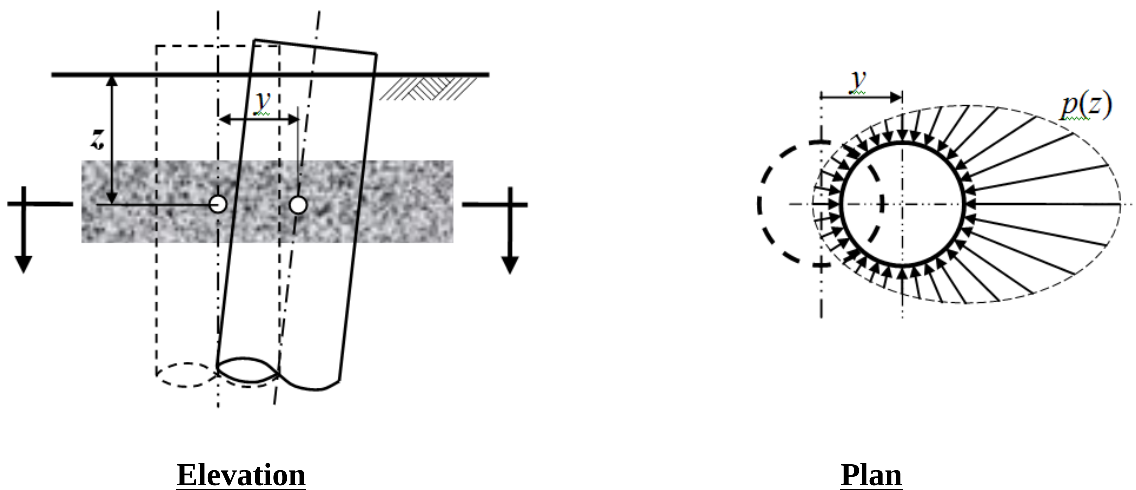


Figure 16.3-7 A Schematic Diagram Showing the Distribution of Soil Stress Reaction $p(z)$ at a Depth z due to a Lateral Deflection y of an Individual Shaft

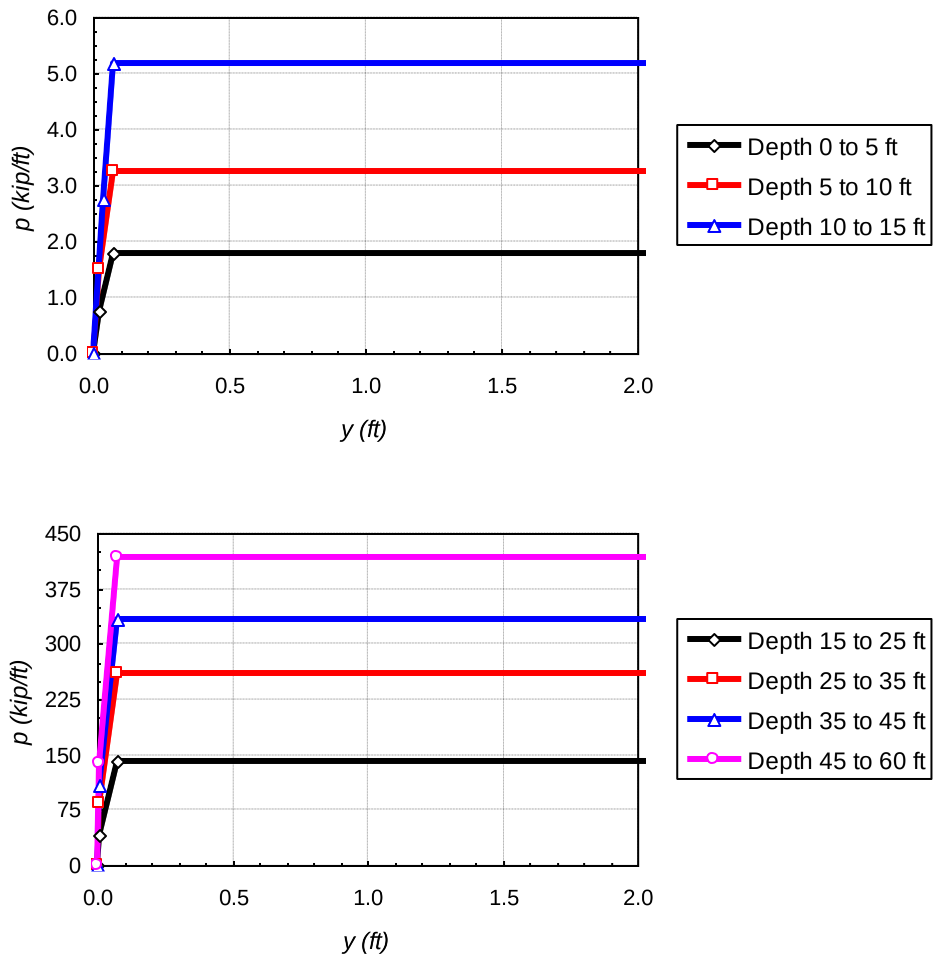


Figure 16.3-8 Idealized Soil *p-y* Curves for an Individual 24-in. Diameter Shaft at Various Depths Measured from the Bottom of the Pile Cap

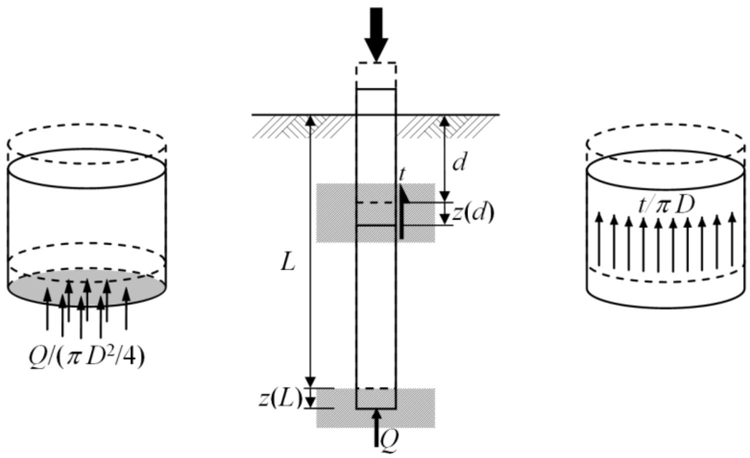


Figure 16.3-9 Soil Stress Reaction

The above figure is a schematic diagram showing the distribution of soil stress reaction *t* along a unit length of the shaft at a depth *d* due to a vertical displacement *z(d)*. Also shown is the soil reaction *Q* at the shaft tip due to a vertical displacement *z(L)*.

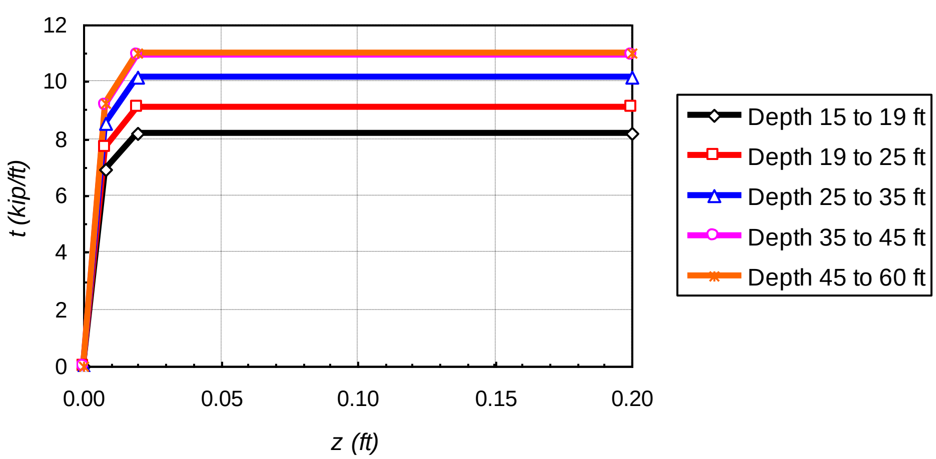


Figure 16.3-10 Idealized Soil *t-z* Curves

The above figure illustrates the idealized soil *t-z* curves for an individual 24-in. diameter shaft at various depths measured from the bottom of the pile cap.

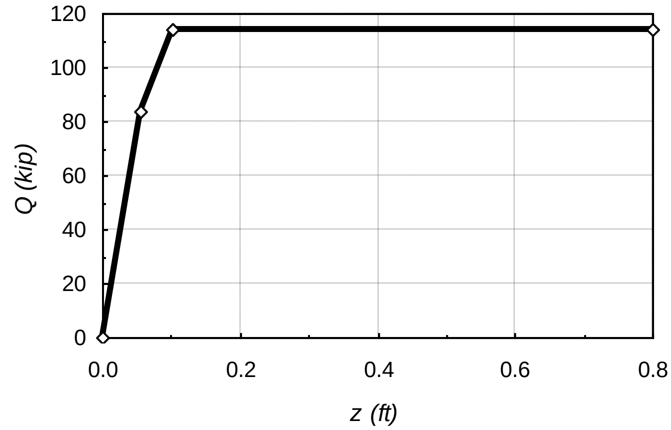


Figure 16.3-11 Idealized Soil *Q-z* Curve for an Individual 24-in. Diameter Shaft

The lateral capacity of a closely spaced shaft group is less than the sum of the capacities of the individual shafts within the group. This behavior can be attributed to overlapping of shear zones within the group. The *p-y* curves are typically developed for an individual shaft. In order to apply these *p-y* curves to a shaft group, a scale factor is applied to the load component, *p*, of the *p-y* curve. This scale factor is referred to as the P-multipliers for the lateral capacity of closely spaced shafts. For this practice bridge, the P-multipliers are based on the CA Amendments 10.7.2.4 (Caltrans, 2014a). Interpolating the results for a center-to-center spacing of 2.625 shaft diameters, and accounting for the side-by-side effects, the P-multipliers corresponding to row 1 (leading), row 2, and rows 3 and higher, are 0.572, 0.392, and 0.284, respectively: see Figure 16.3-12(a).

The vertical capacity of a shaft group in sand is less than the sum of the capacities of the individual shafts due to overlapping zones of shear deformation in the soil surrounding the shafts and loosening of soil during construction (AASHTO 10.8.3.6.3). Figure 16.3-12(b) shows a schematic diagram of the overlapping zones of influence for individual shafts under vertical loads. Figure 16.3-12(c) shows a schematic diagram of the stress conditions below the tips of individual shafts and below the shaft group modeled as a block foundation. The *t-z* and *Q-z* curves are typically developed for an individual shaft. In order to apply these *t-z* and *Q-z* curves to shaft groups, a scale factor is applied to the load component *t* of the *t-z* curves, and to the load component *Q* of the *Q-z* curve. This scale factor is referred to as the Group Efficiency Factor (GEF). Based on AASHTO 10.8.3.6.3, the GEF for a shaft group in compression corresponding to a center-to-center spacing of 2.625 shaft diameters in cohesionless soil is 0.679.

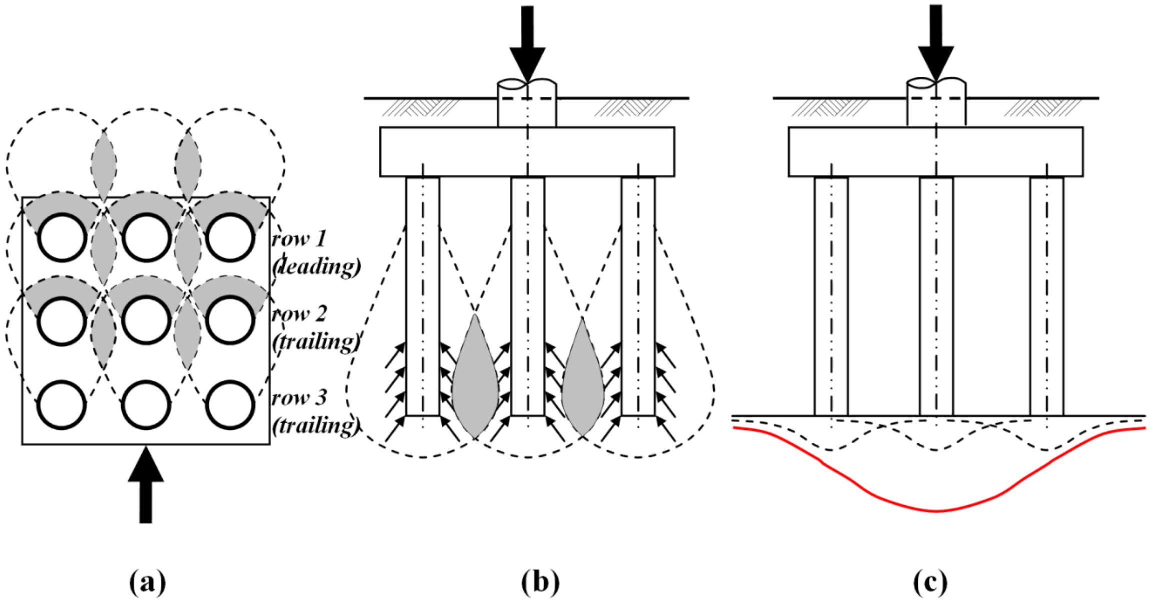


Figure 16.3-12 Overlapping Zones of Closely Spaced Shaft G Foundation

The above figure is of schematic diagrams showing: (a) overlapping zones of influence for a closely spaced shaft group under a lateral load; (b) overlapping zones of influence for closely spaced frictional shafts under an axial compression load; and (c) vertical stress condition below tips of individual shafts (broken lines) and below the entire shaft group modeled as a block foundation (solid line) under axial compression load.

16.3.5 Material Properties

16.3.5.1 Reinforcing Steel ASTM A706 (Grade 60) Properties (SDC 3.2.3)

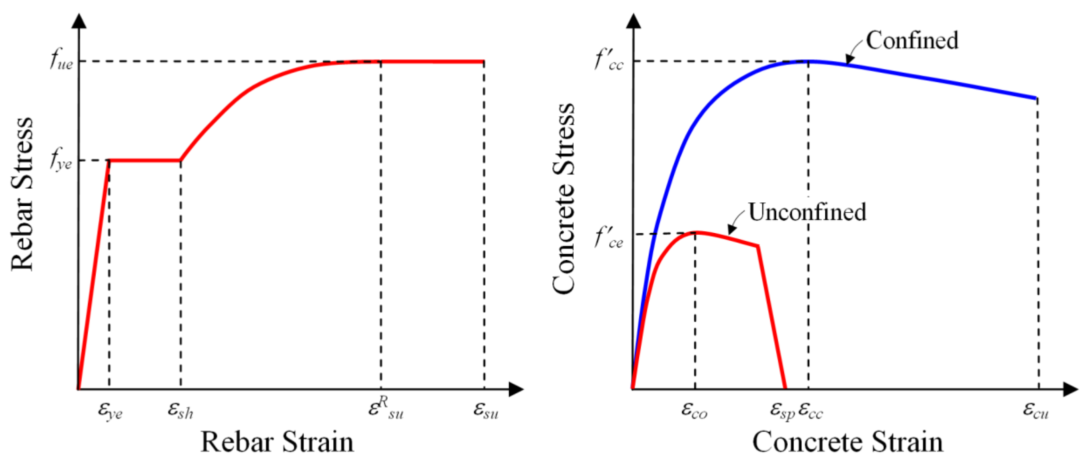
Modulus of elasticity		E_s = 29,000 ksi
Specified minimum yield strength		f_y = 60 ksi
Expected yield strength		f_{ye} = 68 ksi
Specified minimum tensile strength		f_u = 80 ksi
Expected tensile strength		f_{ue} = 95 ksi
Nominal yield strain		ε_y = 0.0021
Expected yield strain		ε_{ye} = 0.0023
Ultimate tensile strain:	for #10 and smaller bars	ε_{su} = 0.12
	for #11 and larger bars	ε_{su} = 0.09
Reduced ultimate tensile strain:	for #10 and smaller bars	ε^R_{su} = 0.09

	for #11 and larger bars	ε^R_{su} = 0.06
Onset of strain hardening:	for #8 bars	ε_{sh} = 0.015
	for #10 and 11 bars	ε_{sh} = 0.0115

16.3.5.2 Normal-Weight Portland Cement Concrete Properties (SDC 3.2.6)

Specified 28-day compressive strength of concrete	f'_c = 3.6 ksi
Expected concrete compressive strength (= 1.3 $f'_c \geq$ 5 ksi	f'_{ce} = 5 ksi
Unit weight of reinforced concrete	w = 0.150 kip/ft³
Unit weight of plain concrete	w_c = 0.14396 kip/ft³
Modulus of elasticity (= 33,000 × $(w_c)^{1.5}$ × $\sqrt{f'_{ce}}$)	E_c = 4030.5 ksi
Ultimate unconfined compression (spalling) strain	ε_{sp} = 0.005
Unconfined concrete strain at maximum compressive strength	ε_{co} = 0.002

For confined concrete, the confined compressive strain, ε_{cc}, and the ultimate compression strain, ε_{cu}, are defined by Mander's constitutive stress-strain model: see Figure 16.3-13.



(a) **(b)**

Figure 16.3-13 Constitutive Models

The above figure shows constitutive models for: (a) park's stress-strain model for rebar; and (b) mander's confined and unconfined stress-strain models for concrete.

16.3.6 Minimum Pile-Cap Depth

The depth of a pile cap shall be sufficient to:

(i) ensure the development of the longitudinal column reinforcement (AASHTO 5.11.2.2 and 5.11.2.4);

(ii) satisfy the column–pile cap joint shear requirements (SDC 7.7.1.4);

(iii) resist the one-way action shear (AASHTO 5.13.3.6.2);

(iv) resist the two-way action shear (AASHTO 5.13.3.6.3); and

(v) satisfy the rigidity requirement if the infinitely rigid pile-cap assumption is made when calculating the axial force demand in the piles/shafts (SDC 7.7.1.3).

16.3.6.1 Pile Cap-Column Proportions

The SDC recommends that the depth of the pile cap, D_{ftg}, be greater than or equal to 0.7 times the column diameter, D_c (SDC 7.6.1). Maintaining this ratio should produce a reasonably well-proportioned structure and satisfy the joint-shear requirements. For this practice bridge:

$$D_{ftg} \geq 0.7D_c \geq 0.7 \times 5 \geq 3.5 \text{ ft} \qquad \text{(SDC 7.6.1-1)}$$

16.3.6.2 Development Length Requirements

The minimum pile cap depth, $D_{ftg,min}$, is defined as:

$$D_{ftg,min} = \textit{Concrete Cover} + 2 \times d_{bd} + l'_d$$

where concrete cover on reinforcement is 6 in. for a pile cap on concrete shafts (Caltrans, 1990), d_{bd} is the deformed diameter of the bottom-mat reinforcement (d_{bd} = 1.44 in. for #10 bars (Caltrans, 1984)), and l'_d is the development length of the longitudinal column reinforcement, which is calculated in the following subsections.

16.3.6.2.1 Development of Deformed Bars in Compression (AASHTO 5.11.2.2)

The development length, l_d, for deformed bars in compression is equal to the product of the basic development length specified in AASHTO 5.11.2.2.1 and the applicable modification factors specified in AASHTO 5.11.2.2.2; but not less than 8 in.

The basic development length, l_{db}, for the #11 column's longitudinal rebar (having a nominal diameter d_b = 1.41 in. (Caltrans, 1984) in compression is the larger of:

$l_{db} \geq 0.63\ d_b\ f_y / \sqrt{f'_c} \geq 0.63 \times 1.41 \times 60/\sqrt{3.6} \geq 28.1$in. [**Governs**]

(AASHTO 5.11.2.2.1-1)

$l_{db} \geq 0.30\ d_b\ f_y \quad \geq 0.30 \times 1.41 \times 60 \quad \geq 25.4$ in. (AASHTO 5.11.2.2.1-2)

Confinement requirement: Column reinforcement is enclosed within #8 hoops, which are not less than 0.25 in. in diameter and spaced at a 4.0 in. pitch; therefore, a modification factor of 0.75 shall apply (AASHTO 5.11.2.2.2) as follows:

$l_d = 0.75 \times 28.1 = 21.1$ in. (which is greater than the minimum value of 8 in.)

16.3.6.2.2 Development of Standard Hooks in Tension (AASHTO 5.11.2.4)

The development length, l_{dh}, in inches, for deformed rebar in tension terminating in a 90^o standard hook shall not be less than:

(i) the product of the basic development length, l_{hb}, and the applicable modification factors;

(ii) 8 bar diameters, but not less than 6 in.

The basic development length, l_{hb}, for the #11 column's longitudinal hooked rebar of yield strength, f_y, not exceeding 60 ksi, is given by:

$l_{hb} = 38\ d_b / \sqrt{f'_c} = 38 \times 1.41/\sqrt{3.6} = 28.2$ in. (AASHTO 5.11.2.4.1-1)

The basic development length shall be multiplied by the following modification factors:

- Cover requirement: For #11 rebar and smaller, side cover (normal to plane of hook) not less than 2.5 in., and for 90° hook, cover on bar extension beyond hook not less than 2 in., a modification factor of 0.7 shall apply (AASHTO 5.11.2.4.2).

- Confinement requirement: For #11 rebar and smaller, hooks enclosed vertically or horizontally within ties or stirrup-ties spaced along the full development l_{dh} at a spacing not exceeding $3d_b$, where d_b is the nominal diameter of the hooked rebar, a modification factor of 0.8 shall apply (AASHTO 5.11.2.4.2).

The development length, l_{dh}, for #11 bars in tension terminating in a standard hook is:

$l_{dh} = 0.7 \times 0.8 \times 28.2 = 15.8$ in. (which is more than 6 in. and $8d_b = 8 \times 1.41 = 11.28$ in.)

The development length for compression, l_d = 21.1 in., governs over the development length for tension, l_{dh} = 15.8 in. Therefore, the minimum pile cap thickness is given by:

$D_{ftg,min} = clr + 2(d_{bd}) + l'_d = 6$ in. $+ 2 \times 1.44$ in. $+ 21.1$ in. $= 30$ in.

16.3.6.3 Rigid Pile-Cap Requirement

If the rigid pile cap assumption is made to estimate the axial forces in the shafts, the SDC requires that $L_{ftg}/D_{ftg} \leq 2.2$ (SDC 7.7.1.3), where L_{ftg} is the cantilever length of the pile cap measured from the column face to the edge of the pile cap. For this practice bridge:

$L_{ftg} = (25 - 5)/2 = 10$ ft

$D_{ftg,min} = L_{ftg} / 2.2 = 10/ 2.2 = 4.5$ ft (SDC 7.7.1.3-1)

16.3.7 Shaft-Group Layout

According to the CA Amendments 10.8.1.2, the minimum center-to-center spacing for shafts is 2.5 times the shaft diameter; and the minimum "clear" edge-distance for shafts is 12 in. For this practice bridge, the shafts are spaced at 5.25 ft, which is greater than the minimum shaft spacing of 2.5 × 2 ft = 5 ft. The selected 12-in. "clear" edge distance for shafts in this practice bridge meets the minimum "clear" edge-distance requirement.

16.3.8 Seismic Forces on Shaft-Group Foundations

In order to determine the force demand on the pile cap (a capacity protected member), a 20% overstrength magnifier is applied to the plastic moment capacity at the base of the column, *i.e.*, $M_o^{col} = 1.2 \times M_p^{col}$, to account for the material strength variations between the column and the pile cap and the possibility that the actual moment capacity of the column may exceed its estimated value (SDC 4.3.1). Note that for Extreme Event I Limit State, the load factor for permanent loads, γ_p, is 1.0 (CA Amendments 3.4.1).

16.3.8.1 Axial Force Effects

The axial force effects due to the self-weight of superstructure and the future wearing surfaces are both obtained from a static analysis of the bridge and are given by:

Self-weight of box-girder, integral pier cap, and concrete barrier		=	1153.587 kip
Self-weight of future wearing surfaces (35 lb./ft^2)		=	131.250 kip
Self-weight of column	$= 0.15$ lb./ft$^3 \times \pi (5\text{ ft})^2/4 \times 22$ ft	=	64.795 kip
Axial load at column base	= 1153.60 kip + 131.25 kip + 64.795 kip	=	1349.632 kip
Self-weight of pile cap	$= 0.15$ lb./ft$^3 \times 25\text{ ft} \times 25\text{ ft} \times 4.5$ ft	=	421.875 kip
Overburden soil weight	$= 0.115$ lb./ft^3 $(25\text{ ft} \times 25\text{ ft} - \pi (5\text{ ft})^2/4)(2\text{ ft})$	=	139.234 kip

Axial load on pile cap, P_P = 1349.632 kip + 421.875 kip + 139.234 kip = 1910.741 kip

Average axial load/shaft = 1910.741 kip / 25 shafts = 76.430 kip

16.3.8.2 Plastic Moment Capacities for Ductile Concrete Members

The plastic moment capacity for the column's and the shaft's cross-sections are estimated by moment-curvature analyses using the computer program *x*-SECTION based on the expected material properties (SDC 3.3.1). The results of the analyses are summarized in Table 16.3-1. Figure 16.3-14 shows the cross-section details of the column and the shaft, as well as the pile cap (a capacity-protected member).

Table 16.3–1 Summary of the Output Results of the *x*-SECTION Program

	Column	Shaft
Axial load (kip)	1349.63	76.43
Cross-Sectional Area, A (ft^2)	19.63	3.142
Idealized Cracked Moment of Inertia, I_{cr} (ft^3)	14.51	0.209
Idealized Plastic Moment Capacity, M_p (kip-ft)	9812.0	313.2

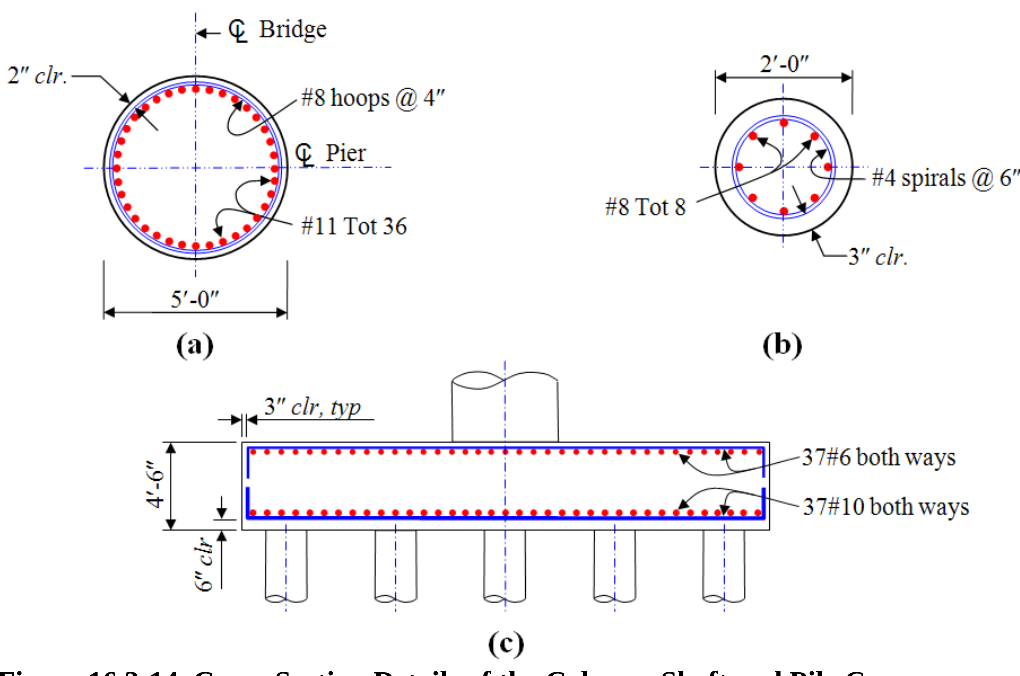


Figure 16.3-14 Cross-Section Details of the Column, Shaft and Pile Cap

The figure above shows the cross-section details of: (a) the column; (b) the shaft; and (c) the pile cap for estimating the plastic moment capacity.

16.3.8.3 Shear Force Effects

Figure 16.3-15 shows the response of Pier 3 under lateral loading in both the transverse and the longitudinal directions. Neglecting the effect of the weight of the column on the plastic moment capacity for the column's cross-section, the overstrength moment at the top and the bottom of the column is given by:

$$M_o^{col@top} = M_o^{col@bot} = 1.2 \times M_p^{col} = 1.2 \times 9812 = 11774.4 \text{ kip-ft}$$

The column's overstrength shear in the transverse and the longitudinal directions can be estimated as follows:

$$V_o^{col}(Trans) = M_o^{col@bot}/H = 11774.4/(22 \text{ ft} + 3 \text{ ft } 8 \text{ in.}) = 458.74 \text{ kips}$$

$$V_o^{col}(Long) = (M_o^{col@top} + M_o^{col@bot})/H_c = (11774.4 + 11774.4)/(22 \text{ ft}) = 1070.4 \text{ kip}$$

Since $V_o^{col}(Long) > V_o^{col}(Trans)$, $\therefore$ longitudinal direction controls.

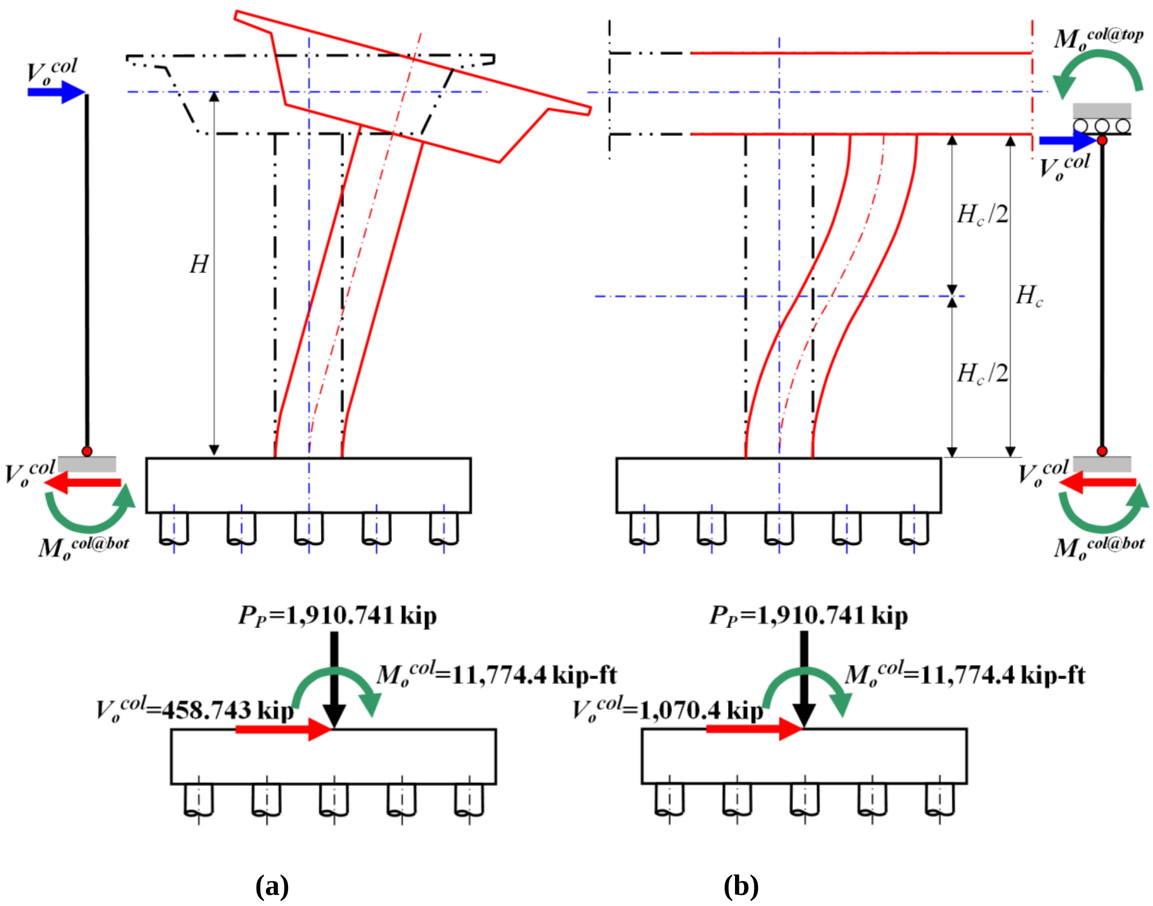


Figure 16.3-15 Schematic Views of the Deformed Shape of Pier 3

The above figure shows schematic views of the deformed shape of Pier 3 in (a) the transverse direction and (b) the longitudinal direction, due to a lateral pushover force V_o^{col}.

16.3.9 Structural Modeling of Shaft-Group Foundations

A shaft-group foundation is a three-dimensional problem, which can be approximately modeled as a two-dimensional problem by assuming the pile cap is infinitely rigid in the direction perpendicular to the longitudinal analysis plane, *i.e.*, the *transverse* direction. For this practice bridge, the shaft-group foundation is divided into five strips of equal width parallel to the longitudinal axis. Each shaft-group foundation strip consists of 25 × 5 × 4.5 ft pile-cap strip on a 5 × 1 array of 24-in. diameter shafts spaced at 5.25 ft: see Figure 16.3-16. Note that the depth of the pile-cap strip equals the depth of the pile cap (= 4.5 ft), but its width is only one-fifth the width of the pile cap, *i.e.*, (1/5) × 25 = 5 ft. The column is modeled as a 5-ft wide frame member endowed with one-fifth the actual cross-sectional properties of the column, *i.e.*, $A^{col}/5$, $I_{cr}^{col}/5$, $M_p^{col}/5$.

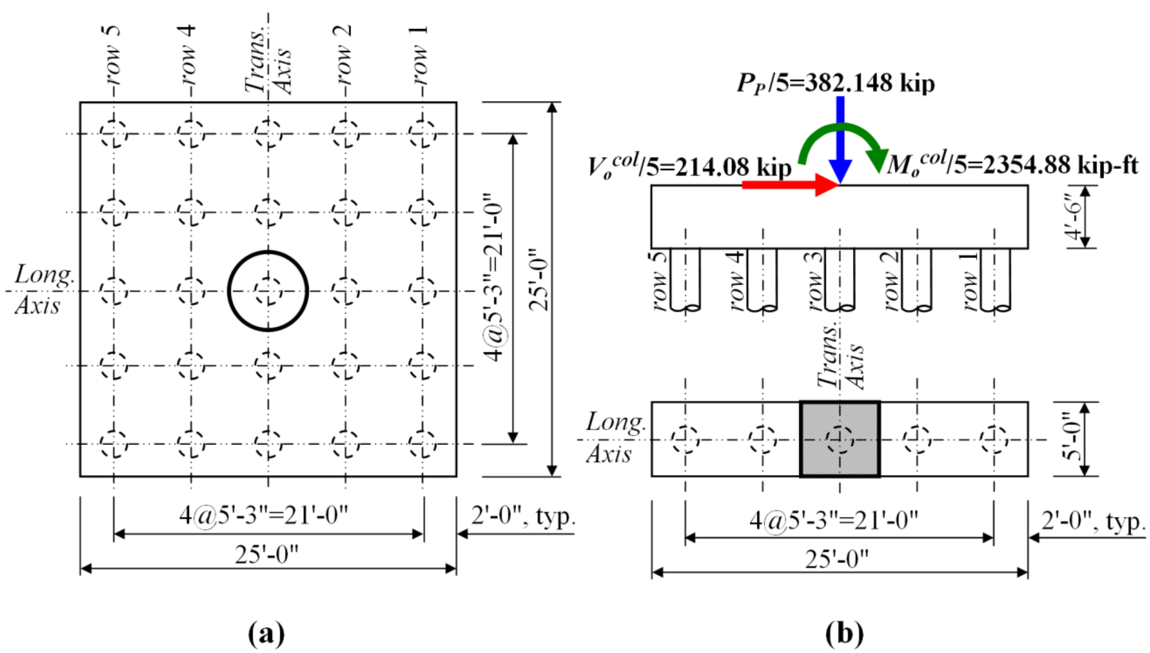


Figure 16.3-16 Shaft Group Foundation Schematic Plan Views

The above figure shows: (a) a schematic plan view of a shaft-group foundation; (b) schematic views of the plan and the elevation of a *two-dimensional shaft-group* foundation strip model of the shaft-group foundation.

16.3.10 Inelastic Static Analysis of Shaft-Group Foundations

For this practice bridge, the inelastic static analysis of the shaft-group foundation strip in the longitudinal direction is performed using three design tools, namely: *w*-FRAME (Seyed, 1995), CSiBridge (CSI, 2015), and LPILE (Reese et al., 2007). The use of each design tool is based on a number of assumptions, which differ from one design tool to another. A detailed study, however, has shown that results obtained by the three design tools are comparable for design purposes. The selection of the appropriate design tool for a specific project is based upon the discretion of the Structural Designer. The aim of the inelastic static analysis of the shaft-group foundation strip is to determine the parameters required to verify the performance and strength criteria of the shaft group, namely, Δ_Y, V_Y, P_{1Y}, ϕ_{Y1}, ϕ_{p1}, Δ_D, Δ_r, P_1, V_1, V_4, and L. Table 16.3-2 summarizes the value of the parameters obtained from *w*-FRAME, CSiBridge, and LPILE.

Table 16.3-2 Parameters Obtained from *w*-FRAME, CSiBridge, and LPILE

	w-FRAME	CSiBridge	LPILE
Δ_Y (ft)	0.15333	0.15763	0.14333
$V_Y/5$ (kip)	178.807	179.827	180.53
P_{1Y} (kip)	221.4	229.18	$\cong P_1 = 262.496$
$\phi_{Y1@P1Y}$ (rad/in.)	0.000218	0.000225	0.000219
$\phi_{p1@P1Y}$ (rad/in.)	0.001562	0.00149	0.001476
Δ_D (ft)	0.25443	0.27931	0.30083
Δ_r (ft)	0.23833	0.26233	0.28074
P_1 (kip)	245.9	255	262.496
V_1 (kip)	53.26	54.79	55.123
V_4 (kip)	36.18	36.37	39.429
L (in.)	192	192	201.6

The following subsections verify the shaft-group performance and strength criteria based on the parameters obtained from the *w*-FRAME program: see Table 16.3-2.

16.3.10.1 Shaft-Group Foundation Performance Criteria

16.3.10.1.1 Demand Ductility Criteria (SDC 4.1.2)

The global displacement ductility demand, μ_D, of the shaft group is given by:

$\mu_D = \Delta_D / \Delta_Y = 0.25443 / 0.15333 = 1.66 \leq 2.5$ **OK**

16.3.10.1.2 Capacity Ductility Criteria (SDC 4.1.3)

In order to calculate the local displacement ductility capacity of an isolated shaft within a shaft group, the location of the potential lower plastic hinge in this shaft needs to be determined. The location of the potential lower plastic hinge in a shaft can be assumed at the same location as the peak value of bending moment below the bottom of the pile cap, *i.e.*, at a depth of L = 192 in. The point of contra-flexure shown in Figure 16.3-2 can be assumed to be the midpoint between the two points of maximum moment along the shaft. The lengths L_1 and L_2 defining the distances from the points of maximum moment to the point of contra-flexure (*cf.* Figure 16.3-2) are given by:

$L_1 \cong L_2 \cong L / 2 = 192$ in. / 2 = 96 in.

The equivalent analytical length of the top and the lower plastic hinges, L_{p1} and L_{p2}, respectively (Figure 16.3-2), are given by:

$L_{p1} = 0.08\, L_1 + 0.15\, f_{ye}\, d_{bl} \geq 0.3\, f_{ye}\, d_{bl}$ (SDC 7.6.2.1-1 for columns)

$L_{p2} = D + 0.08\, L_2$ (Analogous to SDC Eq. 7.6.2.3-1 for non-cased type I shaft)

where f_{ye} (= 68 ksi) and d_{bl} (= 1 in. for #8) are the expected yield stress and the nominal diameter of the shaft's longitudinal reinforcement, and D (= 24 in.) is the shaft diameter,

$L_{p1} = 0.08 \times 96 + 0.15 \times 68 \times 1 = 17.88$ in. $\geq 0.3 \times 68 \times 1 = 20.40$ in.$\Rightarrow$ L_{p1} = 20.4 in.

$L_{p2} = 24 + 0.08 \times 96 \Rightarrow L_{p2} = 31.68$ in.

Neglecting the difference between the axial force at the top and the potential lower plastic hinges, both the yield and the plastic curvatures at the top and the potential lower plastic hinges can be assumed equal, *i.e.*, $\phi_{Y1} \cong \phi_{Y2}$ (= 0.000218 rad/in.) and $\phi_{p1} \cong \phi_{p2}$ (= 0.001562 rad/in.): see Table 16.3-2.

The plastic rotation capacities of the top and the potential lower plastic hinges, θ_{p1} and θ_{p2}, respectively, are given by:

$\theta_{p1} = \phi_{p1} \times L_{p1} = 0.001562 \times 20.40 \cong 0.0319$ rad (SDC 3.1.3-4)

$\theta_{p2} = \phi_{p2} \times L_{p2} = 0.001562 \times 31.68 \cong 0.0495$ rad (SDC 3.1.3-4)

The idealized plastic displacement capacities Δ_{p1} and Δ_{p2} due to the rotation of the of the top and the potential lower plastic hinges, respectively, are given by:

$\Delta_{p1} = \theta_{p1} \times (L_1 - L_{p1}/2) \cong 0.0319 \times (96 - 20.4/2) \cong 2.74$ in. (SDC 3.1.3-3)

$\Delta_{p2} = \theta_{p2} \times (L_2 - L_{p2}/2) = 0.0495 \times (96 - 31.68/2) \cong 3.97$ in. (SDC 3.1.3-3)

The idealized yield displacements Δ_{y1}^{shaft} and Δ_{y2}^{shaft} associated with the formation of the top and the potential lower plastic hinges, respectively, are given by:

$\Delta_{y1}^{shaft} = \phi_{Y1} \times L_1^2/3 = 0.000218 \times 96^2/3 = 0.67$ in. (SDC 3.1.3-2)

$\Delta_{y2}^{shaft} = \phi_{Y2} \times L_2^2/3 = 0.000218 \times 96^2/3 = 0.67$ in. (SDC 3.1.3-2)

The local displacement capacities of the leading shaft Δ_{c1} and Δ_{c2} are given by:

$\Delta_{c1} = \Delta_{y1}^{shaft} + \Delta_{p2} = 0.67 + 2.74 = 3.41$ in. (SDC 3.1.3-6)

$\Delta_{c2} = \Delta_{y2}^{shaft} + \Delta_{p2} = 0.67 + 3.97 = 4.64$ in. (SDC 3.1.3-6)

The local displacement ductility capacities of the leading shaft μ_{c1} and μ_{c1} are given by:

$\mu_{c1} = \Delta_{c1}/\Delta_{y1}^{shaft} = 3.41/0.67 = 5.09 \geq 3$ **OK** (SDC 3.1.4-2)

$\mu_{c2} = \Delta_{c2}/\Delta_{y2}^{shaft} = 4.64/0.67 = 6.93 \geq 3$ **OK** (SDC 3.1.4-2)

The local displacement ductility capacities of the other shafts can be calculated similarly. However, these shafts are subjected to smaller axial compressive forces (or to tension), therefore, they have larger values of μ_{c1} and μ_{c2} compared to those of the leading shaft.

16.3.10.1.3 Global Displacement Criteria (SDC 4.1.1)

The yield displacement associated with the formation of the *first* plastic hinge in the shaft-group strip (at the top of the leading shaft) is Δ_{Y1} = 0.15333 ft (= 1.84 in.). The global displacement demand of the shaft-group strip resulting from the application of the column's overstrength moment and its associated overstrength shear is Δ_D = 0.25443 ft (= 3.05 in.). The global plastic displacement of the shaft group due to the plastic rotation capacity of the first plastic hinge, Δ_{P1}, is given by:

$\Delta_{P1} = (1/2) \times \theta_{p1} \times (L - L_{p1}/2) = (1/2) \times 0.0319 \times (192 - 20.4/2) = 2.9$ in.

The global displacement capacity of the shaft group (measured at the bottom of the pile cap), Δ_C, is given by:

$\Delta_C = \Delta_{Y1} + \Delta_{P1} = 1.84$ in. + 2.9 in. = 4.74 in.

$\therefore (\Delta_C/\Delta_D) = 4.74/3.05 = 1.55 > 1$ **OK**

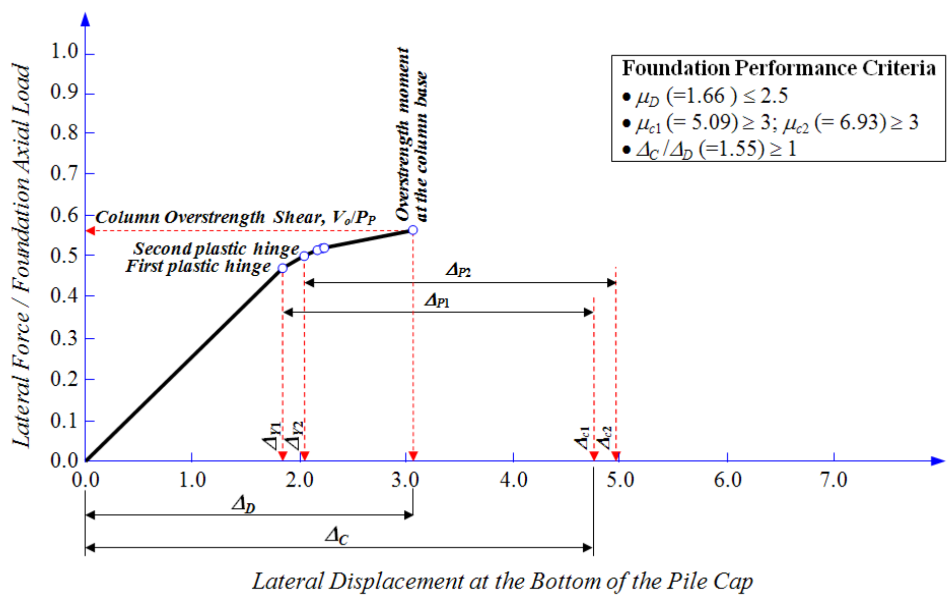


Figure 16.3-17 Force-Displacement Curve for the Shaft-Group Foundation Strip

16.3.10.2 Shaft-Group Foundation Strength Criteria

16.3.10.2.1 Minimum Lateral Strength (SDC 3.5)

The lateral pushover force associated with the formation of the first plastic hinge in the shaft-group strip is given by:

$(V_Y/5) / (P_P/5) = 178.807 / 382.148 = 0.468 \geq 0.1$ **OK**

16.3.10.2.2 P-Δ Effects (SDC 4.2)

For a shaft approximated as a guided-guided column, the P-Δ effects can typically be ignored if the following limit is satisfied (SDC 4.2).

$(P_{dl} \times \Delta_r /2)/(0.20 \times M_p^{shaft}) \leq 1.0$

For this practice bridge, P_{dl} (= 76.430 kip) is the axial force in the shaft attributed to tributary dead load with no overturning effect; M_p^{shaft} (= 313.2 kip-ft) is the idealized plastic moment capacity of the shaft corresponding to P_{dl} = 76.430 kip; and Δ_r is the relative lateral offset (of the displacement demand) between the top and the potential lower points of maximum moment: see Figure 16.3-1(b), given by:

$(P_{dl} \times \Delta_r/2)/(0.2 \times M_p^{shaft}) = (76.430 \times 0.2383/2)/(0.2\times 313.2) = 0.145 < 1.0$ **OK**

Therefore, the P-Δ effects can be ignored.

16.3.10.2.3 Structural Shear Capacity of Ductile Shafts (SDC 3.6)

For the shaft-group foundation strip of this practice bridge, only two shafts need to be checked for shear; namely, the (leading) row 1 shaft, which has the highest compression force, and the row 4 shaft, which has the lowest tension force.

The shear capacity of a shaft, ϕV_n, is based on the nominal material strengths (f'_c = 3.6 ksi, f_y = 60 ksi) and a strength reduction factor ϕ = 0.9, as follows:

$$\phi V_n = \phi V_c + \phi V_s, \qquad \text{(SDC 3.6.1-2)}$$

where V_c and V_s are the nominal shear strengths provided by the concrete and the shear-reinforcement, respectively. The concrete shear capacity of a member designed for ductility is defined by:

$$V_c = v_c \times A_e, \qquad \text{(SDC 3.6.2-1)}$$

where v_c is the permissible shear stress carried by concrete, and A_e is the effective shear area of the member's cross-section, which can be expressed in terms of the gross cross-sectional area, A_g (= π (24)^2/4= 452.39 in.^{2}), of the shaft as follows:

$$A_e = 0.8 \times A_g = 0.8 \times 452.39 \text{ in.}^2 = 361.91 \text{ in.}^2 \qquad \text{(SDC 3.6.2-2)}$$

For shafts whose net axial load is in tension, $v_c = 0$ (SDC 3.6.2). For shafts whose axial load is in compression, the value of v_c at a section depends on the section location:

- Inside the plastic hinge zone:

$$v_c = \text{Factor1} \times \text{Factor2} \times \sqrt{f'_c} \le 4\sqrt{f'_c} \text{ (psi)} \qquad \text{(SDC 3.6.2-3)}$$

- Outside the plastic hinge zone:

$$v_c = 3 \times \text{Factor2} \times \sqrt{f'_c} \le 4\sqrt{f'_c} \text{ (psi)} \qquad \text{(SDC 3.6.2-4)}$$

Factor 1 = $0.3 \le \rho_s\, f_{yh}$ (ksi) / 0.15 + 3.67 – $\mu_d \le 3$ (SDC 3.6.2-5)

Factor 2 = 1 + P_1 (lb.)/{2000 $\times$ A_g (in.^{2})} < 1.5 (SDC 3.6.2-6)

where f_{yh} (= 60 ksi) is the nominal yield stress of the shaft's spiral reinforcement; P_1 is the axial force in the row 1 shaft (=245900 lb.); and ρ_s is the ratio of the volume of spiral reinforcement to the core volume confined by the spiral, which can be expressed in terms of the area of the spiral reinforcement, A_b(#4) = 0.2 in.^{2}, the cross-sectional dimension of the confined concrete core measured between the centerline of the peripheral spiral, D' = 24 – 2 $\times$ (3 + 0.25) = 17.5 in., and the pitch of the spiral reinforcement, s = 6 in.

ρ_s = {4 $\times$ A_b}/{D' $\times$ s} = {4 $\times$ 0.2}/{17.5 $\times$ 6} = 0.0076 (SDC 3.8.1-1)

Factor 1 = 0.3 ≤ {0.0076 $\times$ 60/0.15 + 3.67–1.66} = 5.05 > 3 ⇨ Factor 1 = 3

Factor 2 = 1 + 245,900/{2000 $\times$ 452.39} = 1.272 < 1. ⇨ Factor 2 = 1.272

- Inside the plastic hinge zone:

  v_c = 3 $\times$ 1.272 $\times$ √3600 ≤ {4√3600 = 240 psi} ⇨ v_c = 228.96 psi

- Outside the plastic hinge zone:

  v_c = 3 $\times$ 1.272 $\times$ √3600 ≤ {4√3600 = 240 psi} ⇨ v_c = 228.96 psi

In calculating "Factor 1," the global displacement ductility demand, μ_D, is used in lieu of the local displacement ductility demand, μ_d, since a significant portion of the global displacement of the shaft is attributed to its local deformation (SDC 3.6.2).

The nominal shear strength provided by the concrete, V_c, is defined by:

$V_c = v_c \times A_e$ = 228.96 $\times$ 361.91 = 82,863 lb. = 82.863 kip

$\phi \times V_c$ = 0.90 $\times$ 82.863 = 74.577 kip

The shear reinforcement capacity of a confined circular section is defined by:

$V_s = (A_v\, f_{yh}\, D' / s)$, (SDC 3.6.3-1)

where $A_v = n \times \pi/2 \times A_b$ is the area of shear reinforcement, and n (= 1) is the number of individual interlocking spiral core-sections, and given by:

$A_v = 1 \times \pi/2 \times 0.2 = 0.314 \text{ in.}^2 \geq \{A_{v(min)} = 0.025D's/f_{yh} = 0.044 \text{ in.}^2\}$

(SDC 3.6.3-2 & 3.6.5.2-1)

$V_s = 0.314 \times 60 \times 17.5/6 = 54.95 \text{ kip} \leq \{V_{s(max)} = 8 \times \sqrt{f'_c} \times A_e = 173.717 \text{ kip}\}$

(SDC 3.6.5.1-1)

$\phi V_s = 0.90 \times 54.950 = 49.455$ kip

For the *row* 1 shaft (highest compression shaft), the shear demand is $V_1 = 53.62$ kip

$\phi V_n = \phi V_c + \phi V_s = 74.577 \text{ kip} + 49.455 \text{ kip} = 124.032$ kip

$\phi V_n / V_1 = 124.032/53.62 = 2.31 \geq 1$ **OK**

For the *row* 4 shaft (lowest tension shaft), the shear demand is V_4=36.18 kip

$\phi V_n = \phi V_c + \phi V_s = 0 \text{ kip} + 49.455 \text{ kip} = 49.455$ kip

$\phi V_n / V_4 = 49.455/36.18 = 1.37 \geq 1$ **OK**

Verifications of the performance and strength criteria of the shaft group based upon the parameters obtained from CSiBridge and LPILE are performed similar to the verifications performed using the parameters obtained from the *w*-FRAME program. A summary of the verifications performed using the three design tools is presented in Table 16.3-3.

Table 16.3-3 Summary of the Performance and Strength Criteria Verifications Performed Using the *w*-FRAME, the CSiBridge, and the LPILE Programs

			w-FRAME	CSiBridge	LPILE	Allowable Limits
Performance Criteria	I.	$(\mu_D = \Delta_D/\Delta_Y)$	1.66	1.77	2.10	≤ 2.5
	II.	$(\mu_{c1} = \Delta_{c1}/\Delta_{y1}^{shaft})$	5.09	4.78	4.69	≥ 3.0
		$(\mu_{c2} = \Delta_{c2}/\Delta_{y2}^{shaft})$	6.93	6.48	6.42	
	III.	(Δ_C/Δ_D)	1.55	1.39	1.27	≥ 1.0
Strength Criteria	I.	$(V_Y/5)/(P_P/5)$	0.468	0.471	0.472	≥ 0.1
	II.	$(P_{dl}\Delta_r/2)/(0.2\,M_p^{shaft})$	0.145	0.160	0.171	≤ 1.0
	III.	$(\phi V_n/V_1)$	2.31	2.27	2.27	≥ 1.0
		$(\phi V_n/V_4)$	1.37	1.36	1.25	

16.4 ANALYSIS AND DESIGN OF LARGE DIAMETER COLUMN-SHAFTS

16.4.1 Introduction

The design process of large diameter column-shafts (shafts) includes several steps. Some of the steps highly depend on the characteristics of the surrounding soil. The lateral response of the shaft is governed by the soil close to the ground surface, and soil-foundation-structure interaction (SFSI) analysis is required. However, the axial resistance of the shaft is controlled by the quality and the depth of the deeper soil layers. Shaft embedded in competent soil is capable of resisting ground shaking forces while experiencing small deformations, but for shaft embedded in soft or liquefiable soil, the majority of the soil resistance is lost, which results in large deformations and moment demands in the shaft.

The analysis and design of the large diameter shafts (Types I and II) based on the 6th Edition of AASHTO LRFD Bridge Design Specifications (BDS) together with California Amendments, and Caltrans' Seismic Design Criteria (SDC) Version 1.7 will be illustrated through an example.

16.4.2 Design Practice

Column-shafts with two different types of geometry (Types I and II) are commonly used in Caltrans projects. Bridge foundations are designed to remain essentially elastic when resisting the column's overstrength moment, the associated overstrength shear, and the axial force at the base of the column (Caltrans, 2010). However, prismatic Type I column-shafts are designed to form the plastic hinge below the ground in the shaft and, therefore, are designed as ductile components.

The concrete cover and area of transverse and longitudinal reinforcement in Type I column-shafts may change from column to the shaft. However, the cross section of the confined core is the same for both the column and the shaft.

Type II column-shafts are designed with enlarged shaft so that the plastic hinge will form at or above the shaft/column interface, thereby containing the majority of inelastic action to the ductile column element. The diameter of the shaft is at least 24 in. larger than the diameter of the column, and therefore, two separate cages are used for the column and the shaft. The column cage is embedded in the shaft to allow the transfer of forces from column to the shaft. Refer to SDC for embedment length requirements. Use of a construction joint at the base of the column allows concrete of the lapped portion to be cast under dry conditions, therefore eliminating need of inspection in that area. However, a construction joint is not used for shafts smaller than 5 ft.

Type II shafts are designed to remain elastic, $\mu_D \leq 1$ and the global displacement ductility demand, μ_D for the shaft must be less than or equal to the μ_D for the column supported by the shaft. See SDC Section 7.7.3.2 for design requirements for Type II column-shafts.

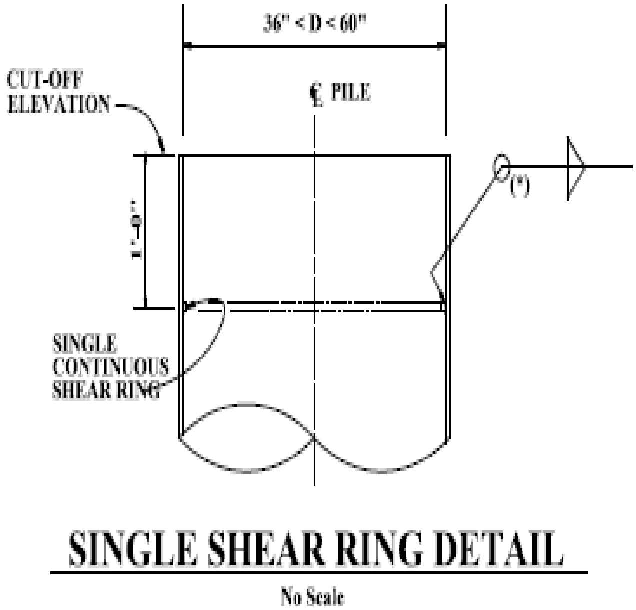
Type I column-shafts are appropriate for short columns, while a Type II column-shaft is commonly used in conjunction with taller columns. The use of Type II shafts will increase the foundation costs, compared to Type I shafts. However, there is an advantage of improved post earthquake inspection and repair. For short columns, designers have used isolation casing in Type-II column-shafts to increase ductility capacity and to reduce shear demand in the shaft.

CISS piles are pipe piles driven to the desired tip, followed by removal of the soil within the steel shell. The interior surface is cleaned to remove any soil or mud. Then, the rebar cage is placed, and concrete is cast inside of the steel shell. CISS piles provide excellent structural resistance against lateral loads and are a good option if any of the following conditions exist: poor soil, soft bay mud or loose sands, liquefaction, scour, and lateral spreading. To improve composite action, a shear transfer mechanism such as studs or shear rings can be welded to the steel shell interior face of large diameter piles. For a more in-depth discussion about the shear rings or welded studs, please refer to Caltrans' Research Project Report (Gebman and Restrepo, 2006).

Following is a summary of current Caltrans' policy on the use of shear rings in CISS piles and drilled shafts with permanent casing:

- Shear rings are not required for pile group foundations.
- Shear rings are not required on corrugated metal pipe casings.
- Shear rings are not required for piles ≤ 3 ft in diameter.
- For Type I and II shafts less than 5 ft but greater than 3 ft in diameter use one ring as shown in Figure 16.4-1a.
- For Type I and II shafts 5 ft or larger in diameter, use two rings as shown in Figure 16.4-1b.

The spacing and size of the shear rings is shown in Table 16.4-1.



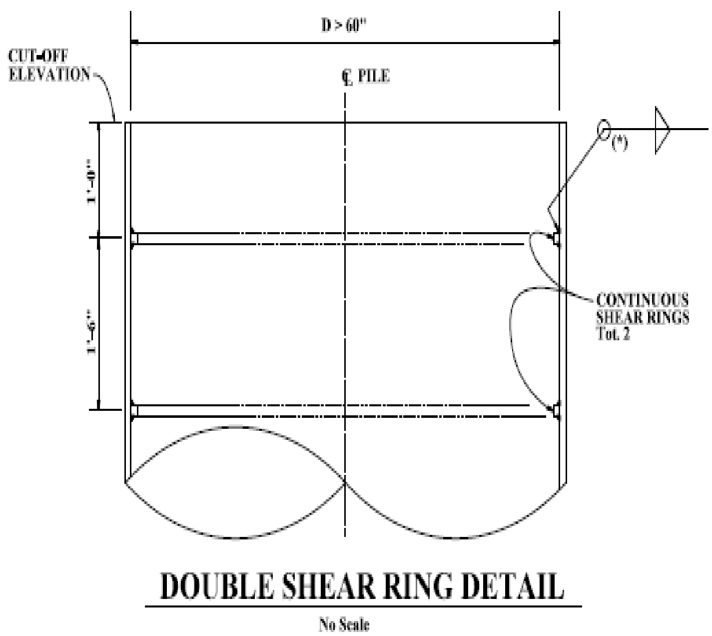


Figure 16.4-1 Single and Double Shear Ring Details

Table 16.4-1 Shear Ring Table

Pile Diameter	Shear Ring Options		Fillet Weld*
	Reinforcing Bar Size	Bar Stock Size	
< 6 ft	#4	1/2 in x 1/2 in	3/8 in.
6 ft to 10 ft	#6	5/8 in. x 5/8 in.	1/2 in.
> 10 ft	#8	1 in. x 1 in.	5/8 in.

16.4.3 Lateral Stability Check of Type I and II Shafts

The column-shaft is considered stable when substantial decrease in pile shaft length does not result in appreciable increase in deflection. Lateral stability analysis may be performed by gradually lowering the tip elevation and calculating corresponding deflection as shown in Figure16.4-2. The length (of the pile) at which lowering the tip elevation does not change deflection appreciably (say less than 5%) is called critical length of the shaft. The following guidance on lateral tip elevation calculations is applicable to both CIDH and CISS piles:

- For column-shafts (types I and II) in multi-column bents, lateral stability analysis must be performed to determine the critical length of the shaft. A factor of safety of 1.0 must be applied to determine the lateral tip elevation. If applicable, the combined effects of liquefaction and scour are to be considered in analysis.
- For column-shafts (types I and II) without rock sockets in single-column bents, lateral stability analysis is performed to determine the critical length of the shaft. A factor of safety of 1.2 must be applied to determine the lateral tip elevation. If applicable, the combined effects of liquefaction and scour are to be considered in analysis.
- For column-shafts (types I and II) with rock sockets in single-column bents, lateral stability analysis is performed to determine the critical length of the shaft. A factor of safety of 1.2 must be applied to the rock socket portion of the shaft in determining the lateral tip elevation. If applicable, the combined effects of liquefaction and scour are to be considered in analysis.

Critical Column Shaft Length (L_P)

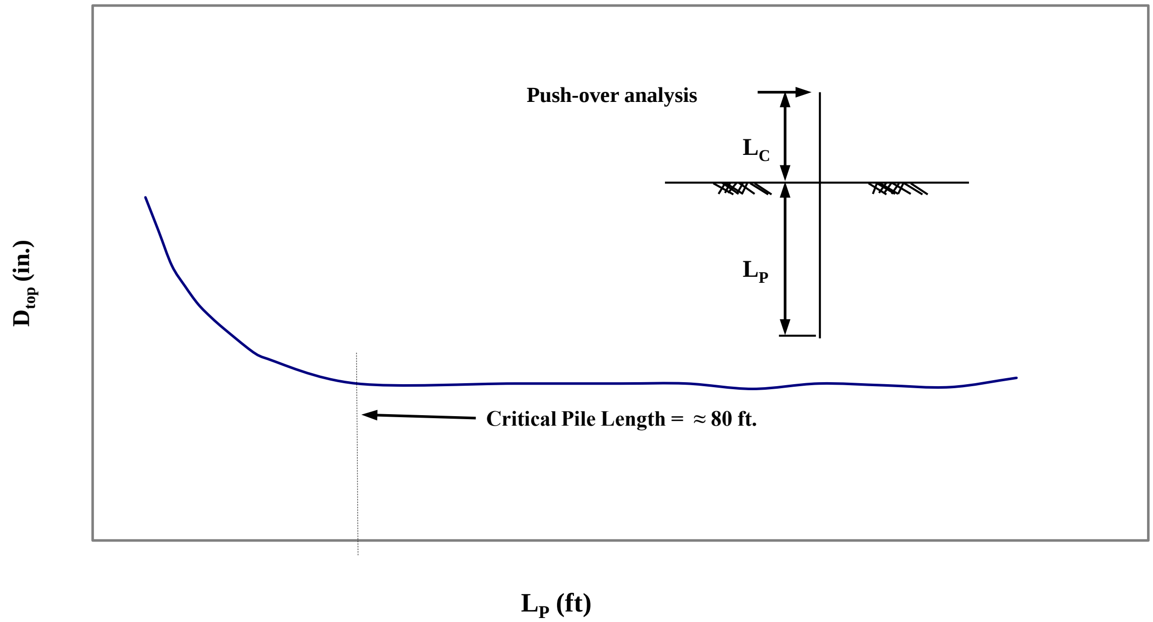


Figure 16.4-2 Lateral Stability Analysis of Column-Shafts

16.4.4 Reinforcement Spacing Requirements of Column-Shafts

16.4.4.1 Minimum Reinforcement Spacing in Column-Shafts

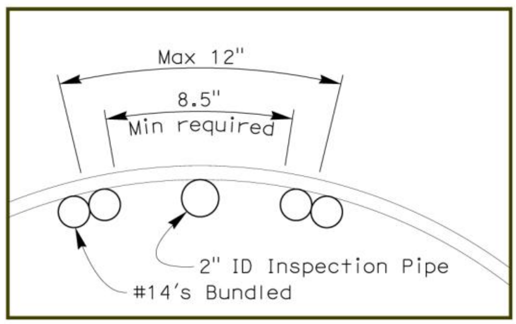
Construction of column-shafts does not require compaction or vibration of concrete except for the top 15 ft of a dry pour because the concrete slump is at least 7 in. Dry pour is defined when the drilled hole is dry or dewatered without the use of temporary casing to control water. Therefore, flow of concrete significantly affects integrity of the shaft and its resistance to applied loads. A clear window of 5 in. x 5 in. has been required for shaft reinforcement to minimize possibility of any anomalies. When the drilled hole cannot be dewatered and the concrete is poured in the wet using the slurry-displacement method of construction, inspection of the shaft is required. Clear spacing at inspection pipe locations may be reduced from 5 in. to 3 in. between the inspection pipe and adjacent longitudinal rebar or 8.5 in. between longitudinal rebars adjacent to inspection pipes as shown in Memo to Designers 3-1. For more details, see Table 16.4-10. Furthermore, if the spacing between the shaft and column reinforcing cages does not meet the minimum spacing requirement, the engineer must either increase the size of the shaft or require a mandatory construction joint (for wet pour).

16.4.4.2 Maximum Reinforcement Spacing in Column-Shafts

The main concern in detailing a shaft is how to accommodate the minimum 8.5 in. longitudinal reinforcing steel clear spacing at inspection pipe locations as required by MTD 3-1. The 8 in. maximum center-to-center spacing of longitudinal reinforcing steel has been revised by California Amendments (10.8.1.3) to 12 in. for shafts with diameters equal to and larger than 5 ft, and 10 in. for shafts with diameters smaller than 5 ft.

The rationale for the required spacings is based on a recent study at University of California-San Diego. Preliminary findings show that use of the proposed spacing given above will not affect the confinement effectiveness of the transverse steel. Furthermore:

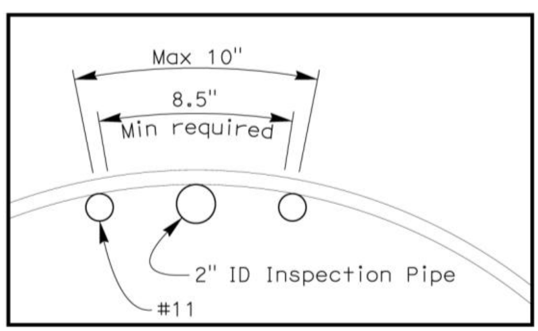
- Number 14 bars are generally the largest bars used for longitudinal steel in column-shafts of 5 ft diameter or larger. The 12 in. spacing was derived by assuming #14s bundled with 8.5 in. clear space at the inspection pipe locations as shown in Figure 16.4-3.



Partial Plan View

Figure 16.4-3 Typical Rebar Spacing in Shafts (Equal and Larger than 5 ft ϕ)

- Number 11 bars are generally the largest bars used for longitudinal steel in column-shafts less than 5 ft diameter. The 10 in. spacing was derived by assuming single #11s with 8.5 in. clear space at the inspection pipe as shown in Figure 16.4-4.



Partial Plan View

Figure 16.4-4 Typical Rebar Spacing in Shafts (Smaller than 5 ft ϕ)

16.4.4.3 Cage Offset Requirements for Type-II Shaft

Construction Specifications require a permanent steel casing to be placed in the area where the column and shaft cages of a Type-II column-shaft overlap and when a construction joint is required. This means the pour in that segment is in the dry condition and can be vibrated. The spacing of the column reinforcement in this area can follow the standard requirements for column steel reinforcement. In a Type-II column-shaft, the allowable offset between centerlines of the column and shaft reinforcement cages is limited by the required horizontal clearance between the two cages. In this case, the clear distance between the two cages must be at least 3.5 in. if a construction joint is used and 5 in. without a construction joint. Furthermore, the offset between centerlines of the shaft cage and the drilled hole (horizontal tolerance at cut-off point) must be limited to provide minimum concrete cover of 3 in. to the shaft outermost reinforcement at all locations. Figure 16.4-5 shows cage offset requirements for Type II Shafts:

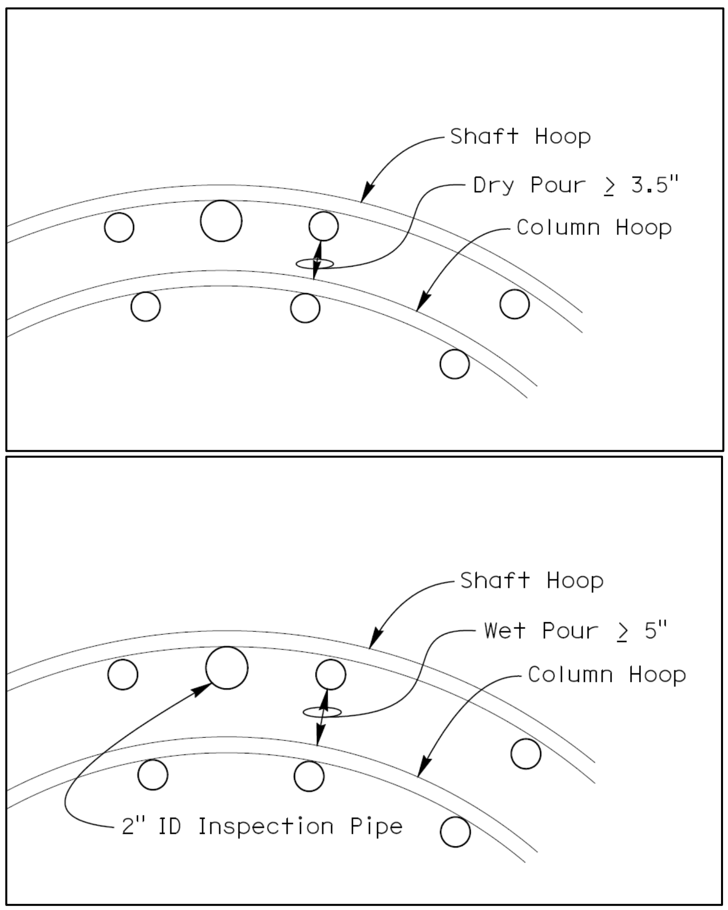


Figure 16.4-5 Cage Offset Requirements for Type II Shaft

16.4.4.4 Rock Socket Design Criteria

When CIDH piles tipped in rock are analyzed for lateral loads, the p-y method reports shear demand forces that exceed the seismic overstrength shear, V_o calculated demand in the column. The abrupt change to high-stiffness *p-y* springs may amplify shear force to more than $5V_o$ within the rock socket.

In current Caltrans practice, the designer must enlarge or reinforce the pile to resist the amplified shear force. However, there is ongoing debate over whether the design force is "real" and whether the discretization of distributed soil reaction to nodal springs is appropriate at the rock interface. Designing for the amplified shear force would increase rebar congestion for an uncertain benefit.

The Caltrans policy imposes an upper limit on the design shear force, recognizing the general problems of the discretization of distributed soil reaction to nodal springs application in rock. The design shear force demand in CIDH shafts and rock sockets need not be taken more than two and half times the seismic overstrength shear force of the column: $V_u \leq 2.5V_o$

The in-ground amplification of shear forces in rock sockets deserves special consideration. Shear demand values from analysis can be misleading, as the discrete spring is not capable of handling a sudden transition to hard rock.

16.4.4.5 Minimum Lateral Reinforcement in Column-Shafts

- If $V_u < \phi V_c$ the minimum lateral reinforcement of the shaft must be # 5 hoops at 12 in. center-to-center spacing.

- If $V_u \geq \phi V_c$ the minimum lateral reinforcement of the shaft must be the larger of $A_v \geq 0.0316\,(f'_c)^{0.5}(b_v)s/f_y$ (AASHTO 5.8.2.5), and # 5 hoops at 12 in. center-to-center spacing.

Radial bundling of longitudinal reinforcement is not allowed due to fabrication challenges.

16.4.5 Design Process

The design process of large diameter column-shafts is presented by the flow chart on the following page.

Type II Column-Shaft Design Process

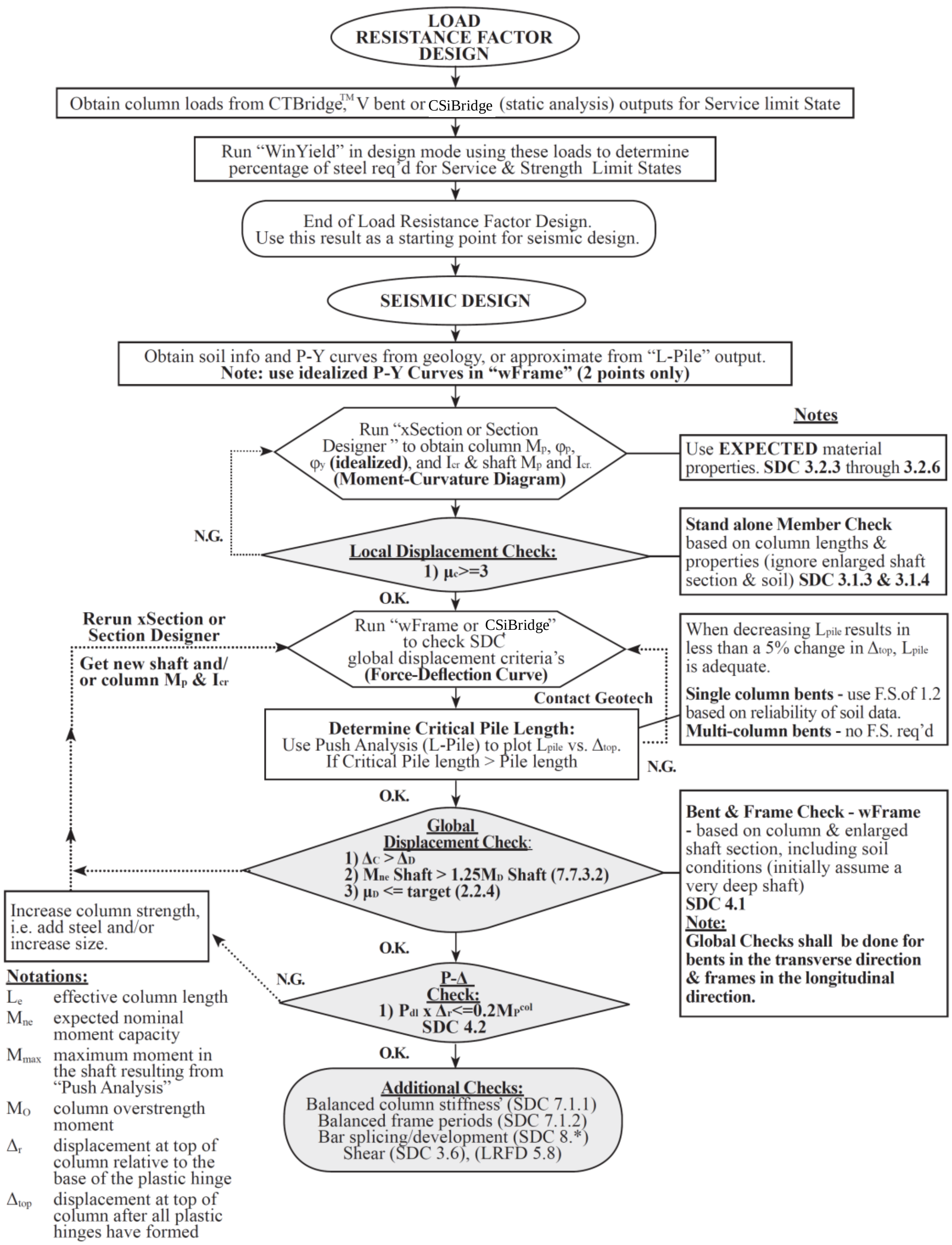


Figure 16.4-6 Typical Design Process for Type II Column-Shaft

16.4.6 Design Example

Design process for a Type II shaft foundation is illustrated through the following example. A fixed column-to-shaft connection has been assumed in this example. However, a pin column-to-shaft connection will reduce forces/moments transferred to the foundations.

Given:

The following example is a two span post-tensioned concrete box girder bridge with a 2-column bent. The superstructure is a post-tensioned concrete box girder and is supported by 2 Type-II shafts as shown in Figure16.4-7. The soil profile of the bent consists of loose to very loose sand with gravel for the top 34 ft, underlain by very dense sand (friable sandstone) to about 55 ft below ground. Bedrock consisting of very hard sandstone/siltstone was encountered below that depth and changed to fresh and hard, with Rock Quality Designation (RQD) of 60% up to 100% at lower depths. Ground water has been encountered near the surface of the streambed. There is a moderate to high potential for liquefaction. Scour potential is high with local scour at the supports estimated from top of streambed or pile cutoff with 15 ft of degradation and contraction.

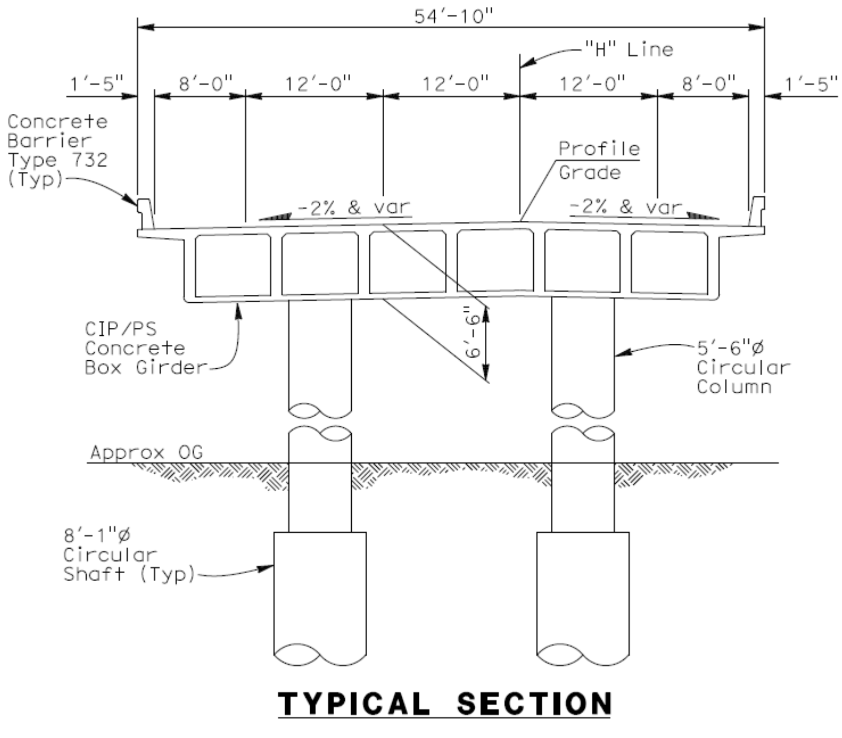


Figure 16.4-7 Transverse Elevation of Example Bridge

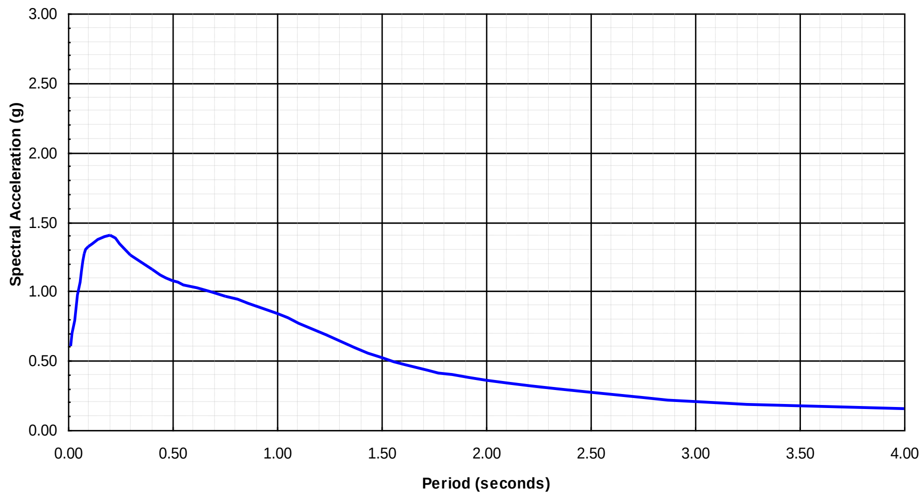


Figure 16.4-8 Recommended Seismic Design Criteria Curve Equivalent to V_{s30} = 270 m/s with PBA 0.6g

As shown in Figure 16.4-10, section A-A, the columns are 5.5 ft in diameter. The shafts are 97 inches in diameter with 60 #14 bundled reinforcing bars and #8 confining hoops at 7.5 in. and double #8 hoops at 7.0 in. spacing along the shaft and the rock socket, respectively. The concrete cover to shaft reinforcement outside and inside the rock socket region is 9 in. and 5 in., respectively (Figure 16.4-10, sections B-B, C-C, and D-D).

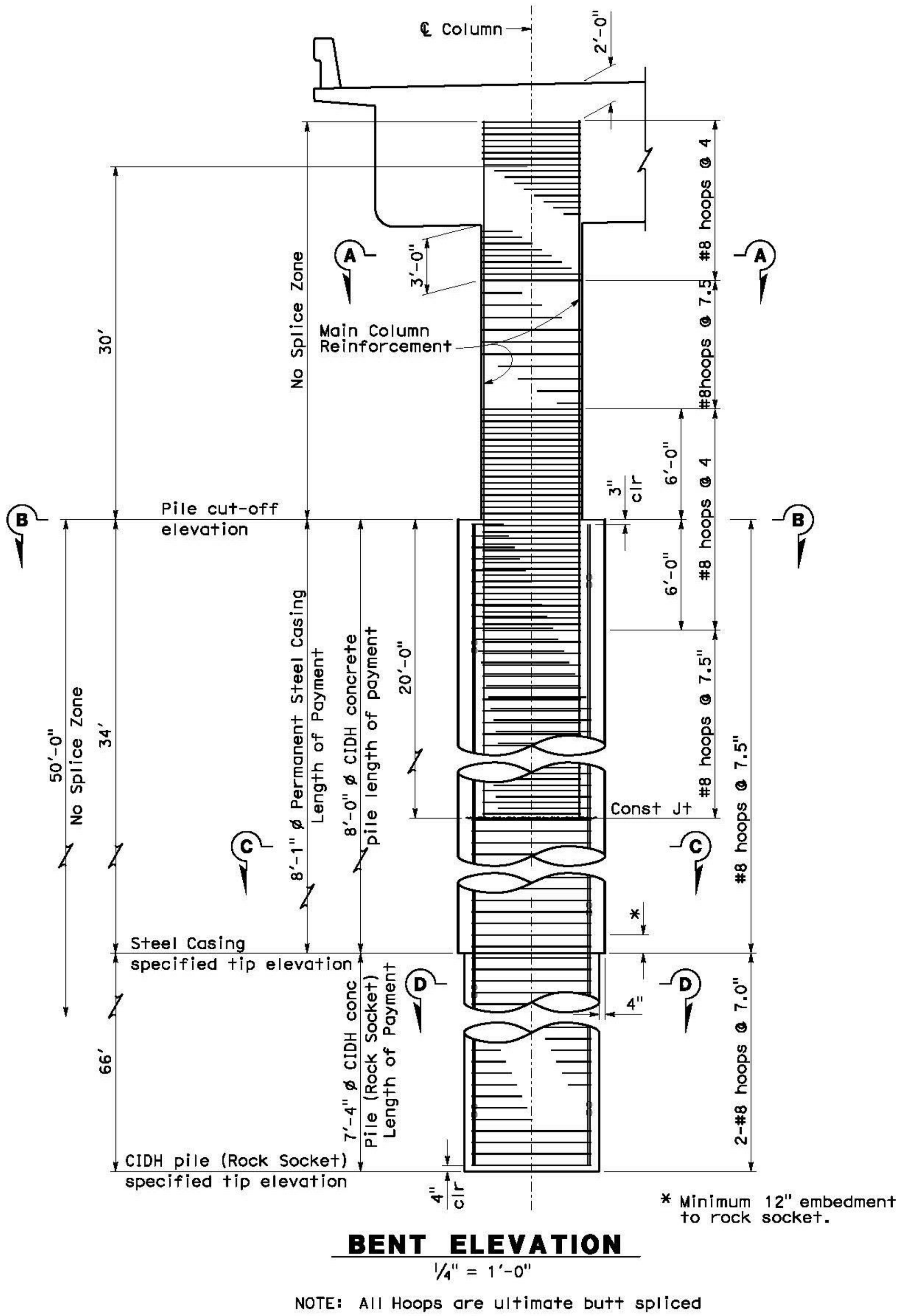
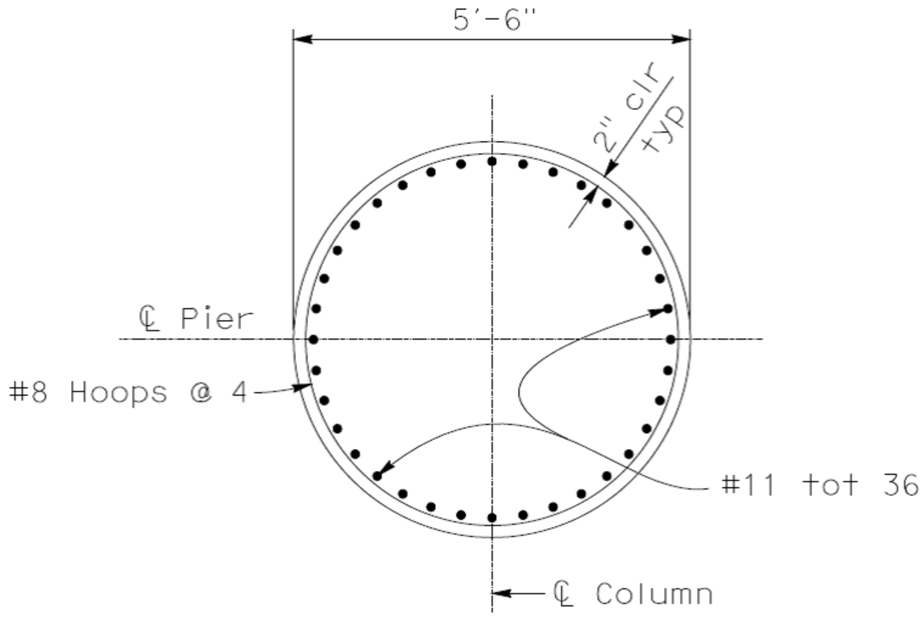
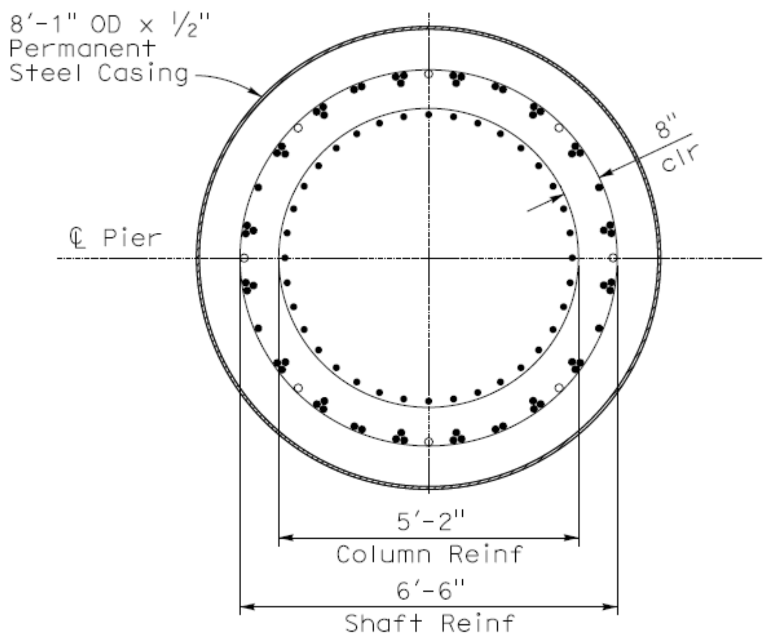


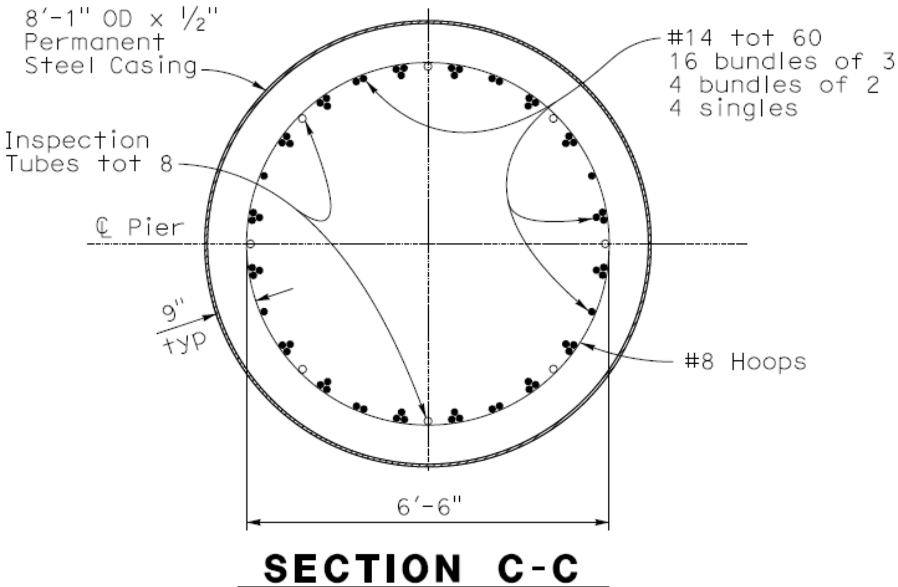
Figure 16.4-9 Column-Shaft Reinforcement Details (not to scale)



SECTION A-A



SECTION B-B



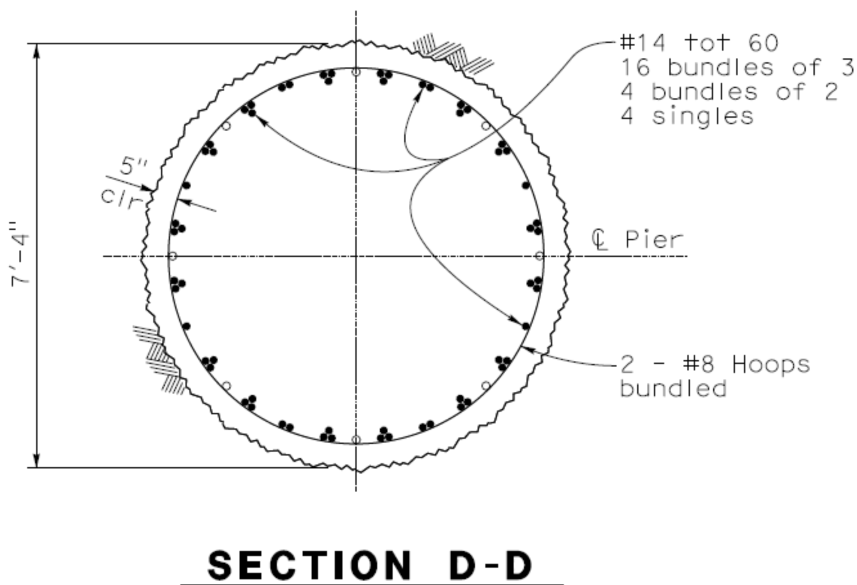


Figure 16.4-10 Typical Shaft and Column Section Details

Note: CIDH Concrete Piling (Rock Socket Section D-D) shown in this example is between the standard diameters of 7ft and 7.5 ft as stated in MTD 3-1 (Caltrans 2013). Contractor can use a special auger to handle a case like this. It is highly recommended to have the rock socket as a standard diameter per MTD 3-1.

Material properties are as follows:

- Concrete: f'_c = 4 ksi; f'_{ce} = 5 ksi
- Reinforcement: f_y = 60 ksi; f_{ye}= 68 ksi
- Unfactored live loads at the base of the Column are listed in Table 16.4-2
- Plastic moment and shear at the base of the column are calculated as M_p=12,309 k-ft, and V_p = 821 kips
- Un-factored dead load and seismic forces at the base of the column are listed in Table 16.4-3
- The nominal geotechnical resistance of the shaft in compression is 5,310 kips
- The nominal geotechnical resistance of the shaft in tension is 2,655 kips

Note: Concrete Strength of f'_c = 4 ksi was chosen in this example for a stronger concrete shear capacity and it is not far off from the default value of f'_c = 3.6 ksi.

Table 16.4-2 Un-factored Live Load Forces at Column Base

	Design Truck			Permit Truck		
Case	I	II	III	I	II	III
M_T (kip-ft)	-189.2	-70.77	-109.7	-296.02	-28.21	-36.89
M_L (kip-ft)	59.25	752.94	123.57	98.91	1,239.31	199.08
P (kip)	295.13	397.07	615.51	458.23	705.35	922.37
V_T (kip)	-13.76	4.74	-5.2	-21.53	-2.57	2.4
V_L (kip)	1.39	17.18	2.9	2.31	128.26	4.65

Note:
Case I: Maximum Transverse Moment (M_T) and associated effects
Case II: Maximum Longitudinal Moment (M_L) and associated effects
Case III: Maximum Axial Force (P) and associated effects

Table 16.4-3 Un-factored Dead Load and Seismic Forces Applied at the Column Base

Un-factored, without impact Loads	M_T (k-ft)	M_L (k-ft)	P (kips)	V_T (kips)	V_L (kips)
DC	86.88	305.65	1,793.75	6.31	-19.8
DW	10.32	35.5	163.65	0.75	-2.30
PS	-193.6	-195.6	37.1	4.45	12.30
Seismic-I Transverse	14,771		1,926	985	
Seismic-II Longitudinal		14,771	1,926		985

Note: The maximum axial force of the column (DC + DW) without considering the overturning effect of seismic forces is 1,926 kips (SAP 2000 transverse analysis), and the plastic moment of the column under such load is 12,309 kip-ft. The corresponding overstrength moment and associated shear force are calculated as $M_o^{col} = 1.2M_p^{col}$ = 14,771 k-ft, and V_o^{col} = 985 kips assuming that L_c = 30 ft. However, in practice, the magnitudes of overstrength moment and associated shear depend on the applied axial force.

Requirements:

- Calculate LRFD factored loads for service, strength, and extreme event-I (seismic) limit states applicable to the shaft design.
- Check geotechnical capacity, assuming nominal geotechnical resistance of 5,310 kips in compression and 2,655 kips in tension.
- Check development length for column reinforcement extended into (Type-II) shaft (SDC 8.2.4).
- Check shaft structural resistance for non-seismic loads.
- Design shaft flexural reinforcement (non-seismic effects).
- Design shaft for shear reinforcement (non-seismic effects).
- Check shaft for seismic effects.

16.4.6.1 LRFD Factored Loads for Service, Strength, and Extreme Event Limit States

Considering effects of live load movements in the longitudinal and transverse directions, the following three live load cases are commonly considered in design:

Case I: Maximum Transverse Moment (M_T) and associated effects

Case II: Maximum Longitudinal Moment (M_L) and associated effects

Case III: Maximum Axial Force (P) and associated effects

Analysis results for other applicable un-factored loads acting on the shaft are given in Table 16.4-3, together with forces and moments resulting from seismic analysis in transverse and longitudinal directions.

The LRFD load combinations used in foundation design and corresponding load factors (AASHTO Table 3.4.1-1) are summarized in Table 16.4-4. The upper and lower limits of permanent load factors (γ_p) are shown as *U* and *L*, respectively.

Table 16.4-4 LRFD Load Factors

	DC	*DW*	*PS*	*EV*	*HL*-93	*P*-15	Seismic
Strength I-*U*	1.25	1.5	1	1.35	1.75	0	0
Strength I-*L*	0.9	0.65	1	0.9	1.75	0	0
Strength II-*U*	1.25	1.5	1	1.35	0	1.35	0
Strength II-*L*	0.9	0.65	1	0.9	0	1.35	0
Strength III-*U*	1.25	1.5	1	1.35	0	0	0
Strength III-*L*	0.9	0.65	1	0.9	0	0	0
Strength V-*U*	1.25	1.5	1	1.35	1.35	0	0
Strength V-*L*	0.9	0.65	1	0.9	1.35	0	0
Service I	1	1	1	1	1	0	0
Extreme Event I	1	1	1	1	0	0	1

The *PS* load factor of 1, as shown in Table 16.4-4, is recommended when a column's cracked moment of inertia is used in analysis. However, for load cases other than extreme event-I, a load factor of 0.5 may be used. See AASHTO LRFD Table 3.4.1-3.

In order to determine loads at the bottom of the shaft, the column diameter and length are needed. For this example, the column diameter is 5.5 ft with a length of 30 ft. The overall un-factored *DC* for column weight = 30 ft × π × $(5.5)^2/4$ × 0.15 kip/ft^3 = 106.92 kips.

The LRFD load factors are applied to axial force and moments in longitudinal and transverse directions to calculate factored loads for strength, service, and extreme event limit states, as summarized in Tables 16.4-5, 16.4-6, and 16.4-7. Loading shown in those tables are for live load cases I, II, and III.

Table 16.4-5 Case III: Maximum Axial Force P and Associated Effects

Factored Loads	M_T (k-ft)	M_L (k-ft)	*P* (kip)
Strength I-*U*	-261	456	3,602
Strength I-*L*	-301	319	2,835
Strength II-*U*	-119	508	3,770
Strength II-*L*	-159	371	3,003
Strength III-*U*	-70	240	2,525
Strength III-*L*	-109	103	1,758
Strength V-*U*	-218	407	3,356
Strength V-*L*	-257	269	2,589
Service I	-206	269	2,610
Extreme Event I, Seismic I	14,771		3,921
Extreme Event I, Seismic II		14,771	3,921

The following are a few example calculations for the factored loads shown in Table 16.4-5:

Axial force for Strength II-*U* limit state: P = 1.25(1793.75) + 1.5(163.65) + 1(37.1) + 1.35(922.37) = 3,770 kips

Longitudinal moment for Strength II-*L* limit state: M_L = 0.9(305.65) + 0.65(35.5) + 1(-195.6) + 1.35(199.08) = 371 k-ft

Transverse moment for Strength II-*L* limit state: M_T = 0.9(86.88) + 0.65(10.32) + 1(-193.6) + 1.35(-36.89) = -159 k-ft

Gross axial force for Service I limit state: P = 1(1793.75) + 1(163.65) + 1(37.10) + 1(615.51) = 2,610 kips

However, the net Service I loads need to be reported to Geotechnical Services for settlement calculations. For a large diameter shaft without any pile cap, the difference between P_{net} *and* P_{gross} is small. Therefore: $P_{net} = P_{gross}$ = 2,610 kips

Similarly, the net permanent loads are to be calculated and reported to Geotechnical Services.

P = 1(1793.75) + 1(163.65) + 1(37.10) = 1,995 kips

Table 16.4-6 Case I: Maximum Transverse Moment and Associated Effects

Factored loads	M_T (k-ft)	M_L (k-ft)	P (kip)
Strength I-*U*	-401	343	3,041
Strength I-*L*	-440	206	2,274
Strength II-*U*	-469	373	3,143
Strength II-*L*	-508	236	2,270
Strength III-*U*	-70	240	2,525
Strength III-*L*	-109	103	1,651
Strength V-*U*	-325	320	2,923
Strength V-*L*	-364	183	2,156
Service I	-286	205	2,290
Extreme Event I, Seismic I	14,771		3,921
Extreme Event I, Seismic II		14,771	3,921

Table 16.4-7 Case II: Maximum Longitudinal Moment and Associated Effects

Factored Loads	M_T (k-ft)	M_L (k-ft)	P (kips)
Strength I-*U*	-193	1,557	3,220
Strength I-*L*	-233	1,420	2,453
Strength II-*U*	-108	1,913	3,477
Strength II-*L*	-147	1,776	2,710
Strength III-*U*	-70	240	2,525
Strength III-*L*	-109	103	1,758
Strength V-*U*	-165	1,256	3,061
Strength V-*L*	-204	1,119	2,294
Service I	-167	898	2,392
Extreme Event I, Seismic (Case I)	14,771		3,921
Extreme Event I, Seismic (Case II)		14,771	3,921

16.4.6.2 Shaft Geotechnical Capacity Check

Strength and Service loads for Case II shown in Table 16.4.7 are the controlling load cases for shaft design in this example.

The CA Amendment articles 10.5.5.2.4 and 10.5.5.3.3 specify resistance reduction factors (ϕ) for strength and extreme event limit states as 0.7 and 1.0, respectively. Compare factored loads on piles/shafts to factored resistance for Strength II-*U* limit state:

Compression: 3,477 kips compression < (0.7) × 5,310 (Nominal) = 3,717 kips ***OK***

The tension demand in this example is zero, and the tension factored resistance is (0.7) x 2,655 (Nominal) = 1,859 kips

Therefore, it is acceptable.

Compare factored loads on piles/shafts to factored resistance for Extreme Event limit State:

Compression: 3,921 kips (compression) < (1) × 5,310 (Nominal) = 5,310 kips **OK**

The tension demand in this example is zero, and the tension factored resistance is (1) x 2,655 (Nominal) = 2,655 kips

Therefore, it is acceptable.

16.4.6.3 Check Development Length for Column Reinforcement into Type-II Shaft

AASHTO Eq. 5.11.2.2: Development of Deformed Shaft Bars in Compression:

Eq. 5.11.2.2.1-1: $l_{db} \geq 0.63(1.88)(60)/(4)^{0.5} = 35.53 \text{ x } 1.2 = 42.64$ in.

(AASHTO 5.11.2.3)

Eq. 5.11.2.2.1-2: $l_{db} \geq 0.3(1.88)(60) = 33.84 \text{ x } 1.2 = 40.61$ in.

(AASHTO 5.11.2.3)

AASHTO 5.11.2.2.2 states that the basic development length may be multiplied by applicable modification factors.

AASHTO 5.11.2.2.2: Reinforcement is enclosed within a spiral of not less than 0.25 in. in diameter and not more than a 4 in. pitch, modification factor = 0.75. (This reduction does not apply because we have the pile shaft hoops at 7.5 in.).

Development length of the bars in compression is 42.64 in.

Column longitudinal reinforcement shall be extended into Type II (enlarged) shafts in a staggered manner with the minimum recommended embedment lengths of ($D_{c,max} + L_d$) and ($D_{c,max} + 2 \times L_d$), where $D_{c,max}$ is the larger cross-section dimension of the column, and L_d is the development length in tension of the column longitudinal bars. This practice ensures adequate anchorage in case the plastic hinge damage penetrates into the shaft (SDC 8.2.4).

AASHTO 5.11.2.1: $l_{db} = 1.25(1.56)(68)/(5)^{0.5} = 59.3$ in.

not less than $l_{db} = 0.4(1.63)(68) = 44.33$ in.

Development length = $l_{db} = 0.6 \times 59.3 = 35.58$ in.

Development length = $(D_{c,max} + L_d) = 66 + 35.58 = 101.58$ in.

Development length = $(D_{c,max} + 2 \text{ x } L_d) = 66 + 2 \times 35.58 = 137.16$ in.

Development provided is 20 ft = 240 in. > 137.16 in. **OK**

16.4.6.4 Check Shaft Structural Resistance:

Check axial force for strength limit state loads:

Shaft Tension Capacity = $\phi P_n = \phi(A_{st} \times f_y)$; (AASHTO 5.7.6.1)
= 0.9 (60 × 2.25) (60) = 7,290 kips **OK**

Note: Tension in shaft for Strength limit state is negligible. Therefore, shaft is acceptable in tension.

Shaft Compression Capacity = $\phi P_n = 0.75\{0.85[0.85 \times f'_c\,(A_g - A_{st}) + f_y A_{st}]\}$;

(AASHTO 5.7.4.4)

$\phi P_n = 0.75\{0.85[0.85 \times 4(7238.3 - 135) + 60(135)]\} = 20{,}560$ kips > 3,770 kips OK

Check axial force for Seismic Extreme Event Loading:

$\phi = 1$ for seismic resistance factors (CA Amendment article 5.5.5).

$\phi = 0.9$ shear resistance factor (SDC 1.7 and 3.6.7) for shear during seismic and other extreme events.

Shaft Tension Capacity:

$\phi P_n = 1\ (60 \times 2.25)\ (60) = 8{,}100$ kips > 0 kips **OK**

Shaft Compression Capacity:

$\phi P_n = 1\{0.85[0.85 \times 4(7{,}238.3 - 135) + 60(135)]\} = 27{,}414$ kips > 3,921 kips

OK

Note: In this example, the factored axial resistance is significantly higher than factored loads. Therefore, the interaction of axial force and bending moment is not checked. However, in general such interaction needs to be considered in design. Shafts are also to be checked for maximum shear for service and strength limit states.

Assuming, the maximum shear will occur at the top of the shaft, the maximum factored strength II limit state shear force is as follows:

Strength II: $V_u = 1.35(128.26^2 + 2.57^2)^{0.5} = 173.2$ kips

The shear capacity of a reinforced concrete pile/shaft can be calculated as follows:

$V_n = V_c + V_s + V_p \leq 0.25\, f'_c\, b_v\, d_v + V_p$ (AASHTO 5.8.3.3)

$V_c = 0.0316\beta(f'_c)^{0.5}(b_v)(d_v)$

$V_s = [A_v\,(f_y)(d_v)\,(\cot\theta + \cot\alpha)\sin\alpha]/\,s$

$V_p = 0$ (no pre-stressing in pile/shaft)

$b_v = D = 96$ in. (AASHTO 5.8.3.1)

$d_v = 0.9\, d_e = 0.9\,(70.91) = 63.82$ in. (AASHTO 5.8.2.9)

$D_r = D - 2\,(clr + \text{hoop}\ d_{db} + \text{long}\ d_{db}/2) = 96 - 2(9 + 1.13 + 1.88) = 71.98$ in.

$d_e = D/2 + D_r/\pi = 96/2 + 71.98/\pi = 70.91$ in. (AASHTO C5.8.2.9-2)

$\alpha = 90$

$A_v = 0.79$ in.^{2}, $S = 7.5$ in. down to the bottom of casing then $A_v = 1.58$ in.^{2} $S = 7$ in., at rock socket section

Check for minimum transverse reinforcement:

$A_v \geq 0.0316\,(f'_c)^{0.5}(b_v)s/f_y = 0.0316(4)^{0.5}(96)(7.5)/60 = 0.76$ in.^{2} **OK** (AASHTO 5.8.2.5)

Determine shear capacity for Strength II:

β = 2, θ = 45°, α = 90° (For Strength II piles in compression, the simplified method, AASHTO 5.8.3.4.1, may be used.)

$V_c = 0.0316(2)(4)^{0.5}(96)(63.82) = 774.42$ kips

V_s = 0.79(60)(63.82)[(cot(45°)+cot(90°)) sin(90°)]/7.5 = 403.34 kips

V_n = 774.42 + 403.34 = 1,177.76 kips < 0.25(4)(96)(63.82) = 6,126.72 kips **OK**

$V_r = \phi V_n$ = 0.9(1,177.76) = 1,060 kips > 550 kips (See Figure 16.4-11) **OK**

Note: AASHTO Eq. 5.8.3.4.2-4 or *B*5.2 may have been used instead of the simplified method to calculate a higher shear resistance, as follows:

$\varepsilon_x = [(|M_u|/d_v) + 0.5N_u + 0.5|V_u - V_p|\cot(\theta) - A_{ps} f_{po}]/[2(E_s A_s + E_p A_{ps})]$

(AASHTO B5.2-1)

N_u = 0 kips (no tensile load)

M_u = 1916 kip-ft (moment demand assumed at top of pile/shaft)

V_u = 173 kips (shear demand assumed at top of pile/shaft)

A_{ps} = 0 (no pre-stressing steel in pile/shaft)

$\varepsilon_x = [(|M_u|/d_v) + 0.5N_u + 0.5|V_u - V_p|\cot(\theta) - A_{ps}f_{po}]/[2(E_s A_s + E_p A_{ps})]$

(AASHTO B5.2-1)

$\varepsilon_x = [(1916 \times 12/63.82) + 0.5|173|\cot(45)]/[2(29{,}000)(60 \times 2.25)] = 0.000057$

$v_u = |V_u - \phi V_p|/(\phi b_v d_v) = |173|/[(0.9)(96)(63.82)] = 0.0313$ (AASHTO 5.8.2.9)

$v_u/f'_c = 0.0313/4 = 0.0078$

β = 3.24, θ = 24.3, (AASHTO Table B5.2-1)

Using θ = 24.3, resolving for ε_x = 0.000057; β = 3.24

$V_c = 0.0316(3.24)(4)^{0.5}(96)(63.82) = 1{,}254.55$ kips

V_s = 0.79(60)(63.82)[(cot(24.3°) + cot(90°))sin(90°)]/7.5 = 893.30 kips

V_n = 1,254.55 + 893.30 = 2,147.85 kips < 0.25(4)(96)(63.82) = 6,126.72 kips **OK**

$V_r = \phi V_n$ = 0.9 (2,147.85 kips) = 1,933.07 kips > 550 kips **OK**

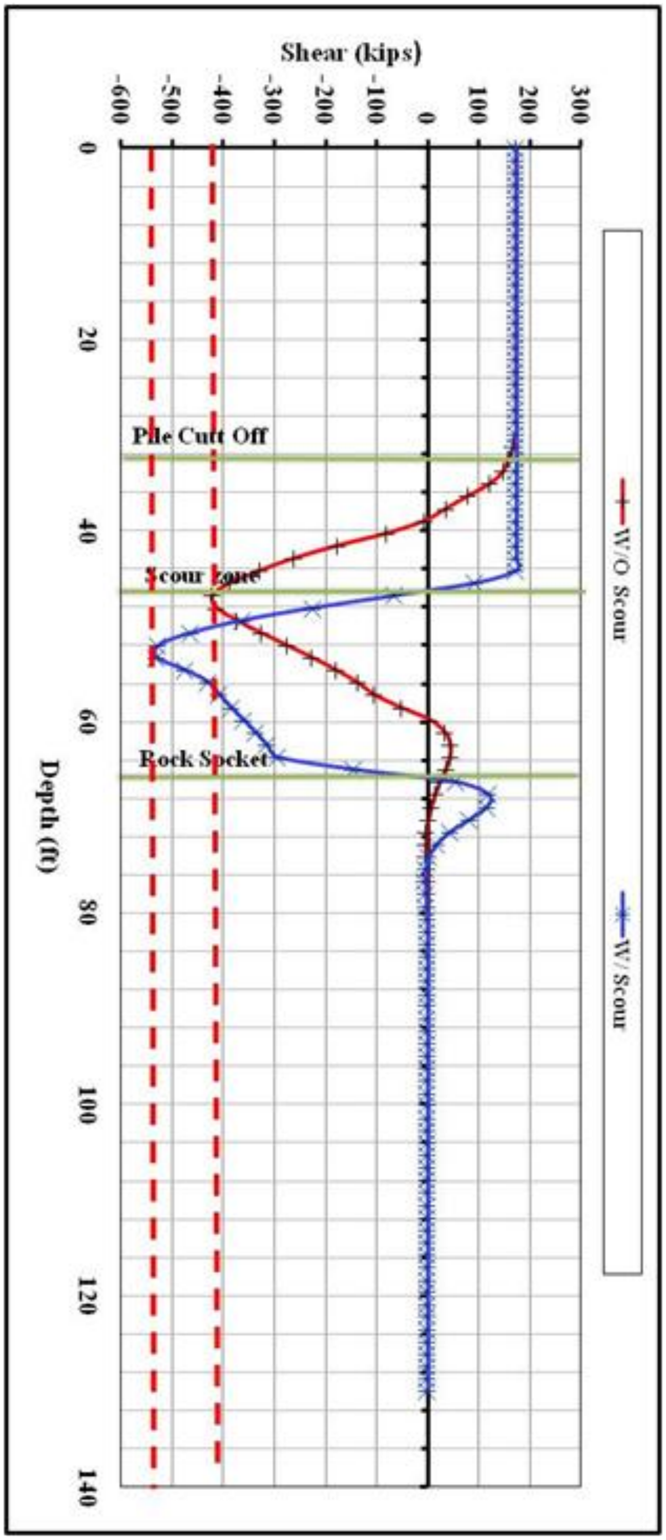


Figure 16.4-11 Shear Diagram at Strength State for Single Shaft

Determine the shear capacity with 0.5 in. casing using the following (AASHTO 5.8.3.4.1) simplified method:

Determine shear capacity for Strength II:

$\beta = 2$, $\theta = 45°$, $\alpha = 90°$ (for Strength II piles in compression, the simplified method, AASHTO 5.8.3.4.1, may be used.)

$V_c = 0.0316(2)(4)^{0.5}(96)(63.82) = 774.42$ kips
$V_s = 0.79(60)(63.82)[(\cot(45°) + \cot(90°))\sin(90°)]/7.5 = 403.34$ kips

$V_{np} = 0.5F_{cr}A_g$ with $F_{cr} = 0.78E_s/(D/t)^{3/2} \leq 0.58\,F_y$ (AASHTO 6.12.1.2.3c)

$F_{cr} = 0.78E_s/(D/t)^{3/2} = 0.78 \times 29000/(97/0.5)^{3/2} = 8.37$ ksi $< 0.58 \times 45 = 26.1$ ksi

$V_{np} = 0.5F_{cr}A_g$ Nominal shear resistance (pipe or casing)

$V_{np} = 0.5 \times 8.37 \times \pi \times 97 \times 0.5 = 637.65$ kips

$V_n = 774.42 + 637.65 + 403.34 = 1{,}815.41$ kips $< 6{,}126.72$ kips **OK**

$V_r = \phi V_n = 0.9$ (1,815.41 kips) = 1,633.87 kips > 550 kips

(See figure 16.4-11) **OK**

If the pile shear demand exceeds the capacity, a more refined analysis may be performed accounting for the passive resistance of the soil against the pile. Maximum moments acting on the shaft at the bottom of the column for extreme event I, strength and service are shown in Tables 16.4-5, 16.4-6 and 16.4-7 as:

LRFD Limit State	M_T (kip-ft)	M_L (kip-ft)	$\sqrt{M_T^2 + M_L^2}$ (kip-ft)
Strength Limit: Strength II Case II	-108	1,913	1,916
Strength Limit: Strength II Case I	-508	236	560
Extreme Event I Limit State: Seismic I/II	14,771	14,771	20,889
Service Limit State Case II	-167	898	913
Service Limit State Case I	-286	205	352

The maximum shaft bending moment demand for non-seismic loading (6,500 kip-ft) is extracted from Figure 16.4-12. The maximum shaft bending moment capacity for non-seismic loading is determined using the Xsection program (Caltrans, 2006):

$M_r = \phi M_{ne@0.003} = (0.9)26{,}928 = 24{,}235.2$ k-ft > 6,500 k-ft **OK**
(AASHTO 5.5.4.2)

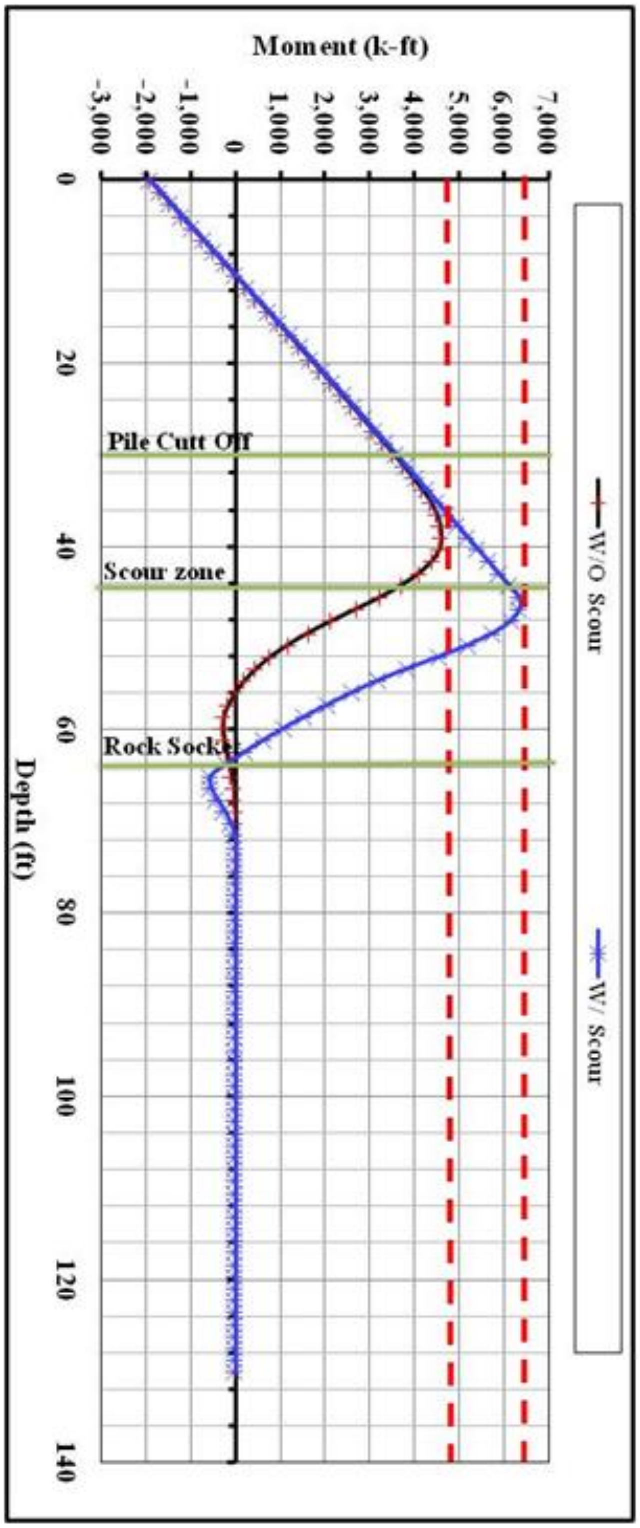


Figure 16.4-12 Moment Diagram at Strength State for Single Shaft

Communications of Structural Designer with Geotechnical Services (Refer to MTD 3-1 and MTD 1-35)

The following table will be sent from Structure Design to Geotechnical Services:

Table 16.4-8 Foundation Design Loads

Foundation Design Loads											
	Service-I Limit State (kips)			Strength Limit State (Controlling Group, kips)				Extreme Event Limit State (Controlling Group, kips)			
	Total Load		Permanent Loads	Compression		Tension		Compression		Tension	
Support No.	Per Support	Max. Per Pile	Per Support	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile	Per Support	Max. Per Pile
Bent 2	2,610	2,610	1,995	3,770	3,770	0	0	3,921	3,921	0	0

Notes:

- Support loads shown are per column
- Service I loads are reported as net loads
- Strength and Extreme Event loads are reported as gross loads
- Load tables may be modified to submit multiple lines of critical load combinations for each limit state, if necessary

16.4.6.5 Shaft Seismic Analysis and Design Procedures

16.4.6.5.1 Design Approach

The design of CIDH with permanent steel casing would be analyzed and designed similarly to a CISS.

16.4.6.5.2 Preliminary Substructure/Foundation Design

Based on the geotechnical and hydraulics engineer requirements, the typical bent was designed as follows. The clear column height is 30 ft supported by an 8 ft diameter CIDH with 0.5 in. thick permanent casing for the top 34 ft with a 66 ft rock socket at diameter of 7 ft - 4 in. The selected pile system serves well in a 2-column bent arrangement to overcome the potential channel bed scour and soil liquefaction.

16.4.6.5.3 Preliminary Demand Assessment

It is a common practice to design bridges for *service and strength limits states* and then refines the bridge system for seismic design requirements. Furthermore, the seismic design is a non-linear process and the bridge design may have to be iterated several times to reach a desirable solution. Finally, the designer may have to take the seismic changes to the bridge back to the service load design and go through the entire design procedure until service, strength, and seismic requirements are all met.

The engineer's experience and judgment is a key factor in the amount of time required to perform these tasks.

One possible method in the seismic design is the creation of a linear elastic model of the structure and performing of a modal dynamic analysis to obtain an estimate of the displacement demands. An alternative method, as used in this chapter, is to use the initial slope of the force-deflection curve from the push analyses of bents and frames and estimate the displacement demands from such analyses (SDC 5.2.1). Details of the displacement demand computation will be shown later.

16.4.6.5.4 Material Properties

The material properties used in the seismic analysis are as follows:

- Concrete strength, f'_{ce} = 5000 psi
- Concrete specific weight for calculation of modulus of elasticity = 145 lb./ft^3
- Other concrete properties are based on SDC sections 3.2.5 and 3.2.6.
- Reinforcement (A706 steel) properties are based on SDC sections 3.2.2 and 3.2.3.

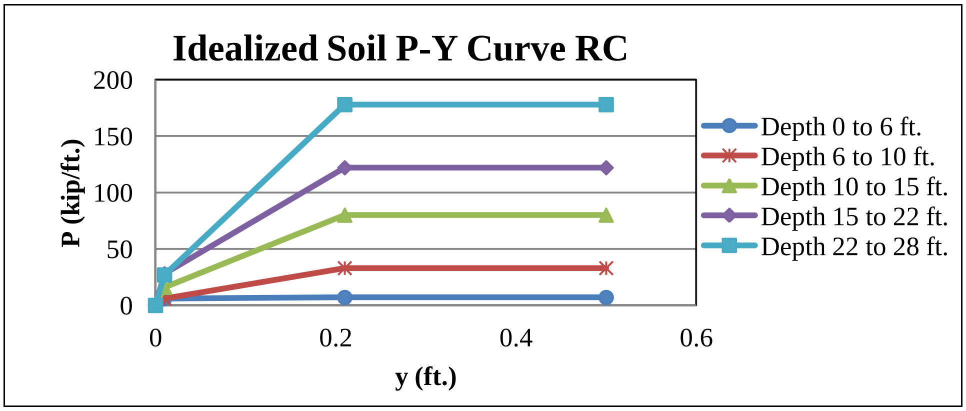
16.4.6.5.5 Soil Springs for CIDH Piles with Permanent Steel Casing

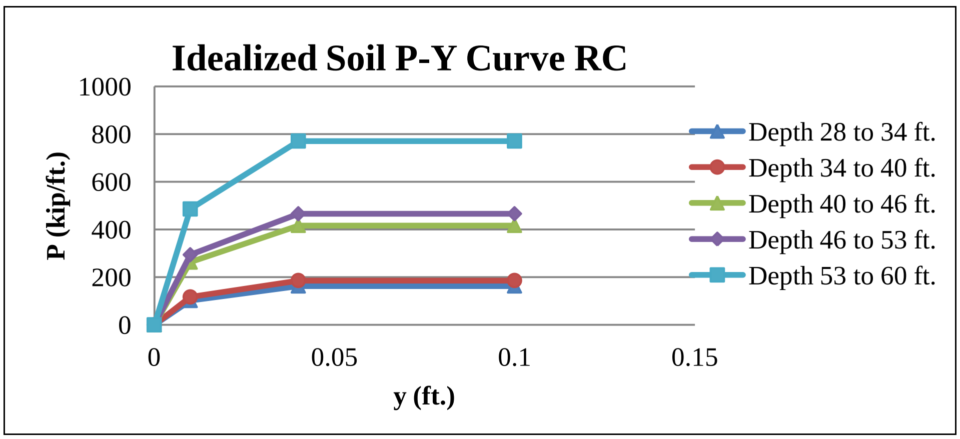
The shaft is embedded in two soil layers. The topsoil layer with depth of 34 ft is sand with gravel and the bottom soil layer is sandstone rock material. The *p-y* and *t-z* curves reflect these characteristics.

The *p-y* curves are used in the lateral modeling of soil as they interact with the large diameter shafts. The geotechnical engineer generally produces these curves and the values are converted to proper soil springs within the push analysis. The spacing of the nodes selected on the pile members would naturally change the values of spring stiffness. However, a minimum of 10 elements per pile is advised (recommended optimum is 20 elements or 2 ft to 5 ft pile segments).

The *t-z* curves are used in the modeling of skin friction along the length of piles. Vertical springs are attached to the nodes to support the dead load of the bridge system and to resist overturning effects caused by lateral bridge movement. The bearing resistance at the tip of the pile is usually modeled as a *q-z* spring. This spring may be idealized as a bilinear spring placed in the boundary condition section of the push analysis input file.

Figures 16-4.13 and 16-4.14 show Idealized Soil Springs for the right (RC) and left (LC) columns, respectively:





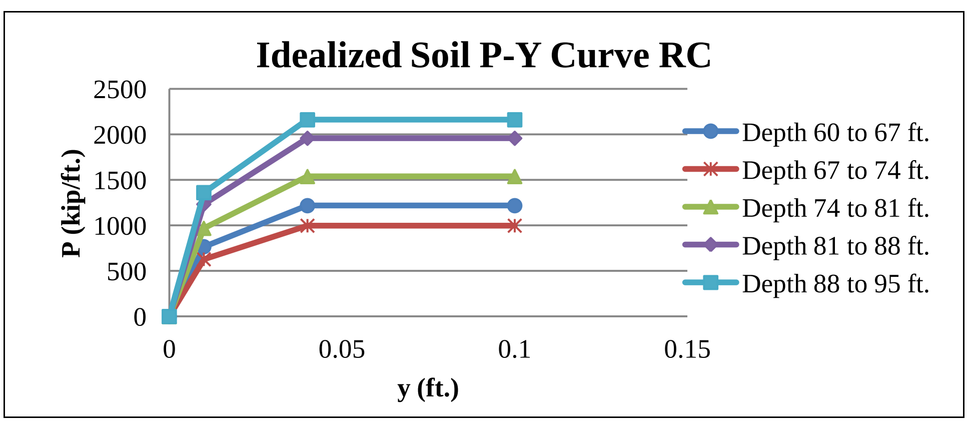
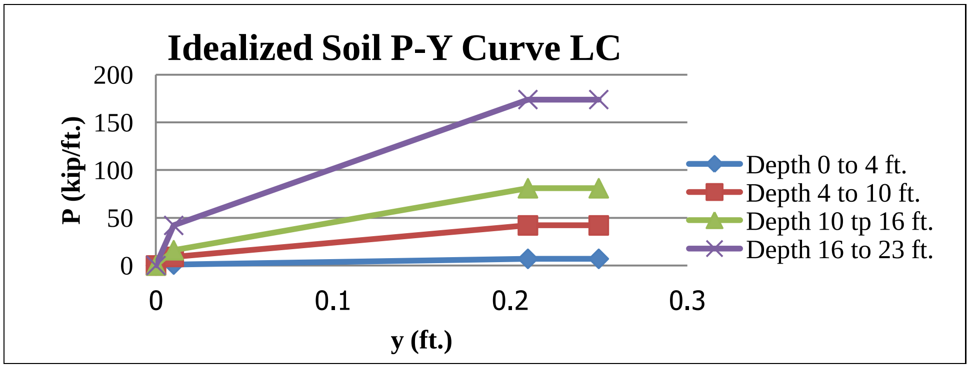
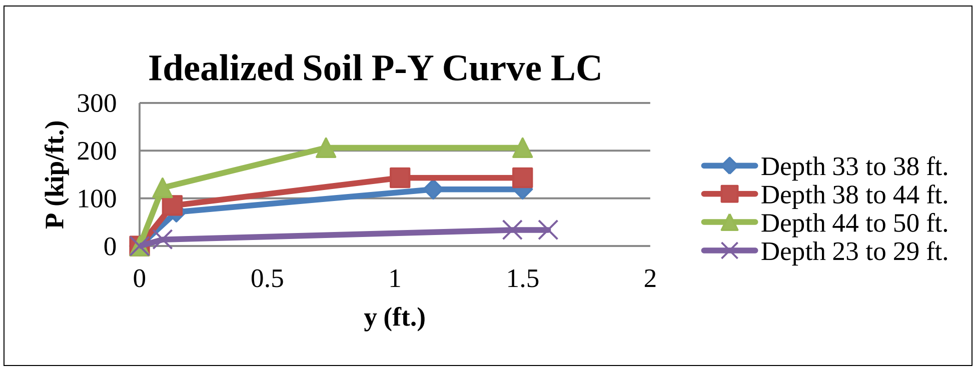
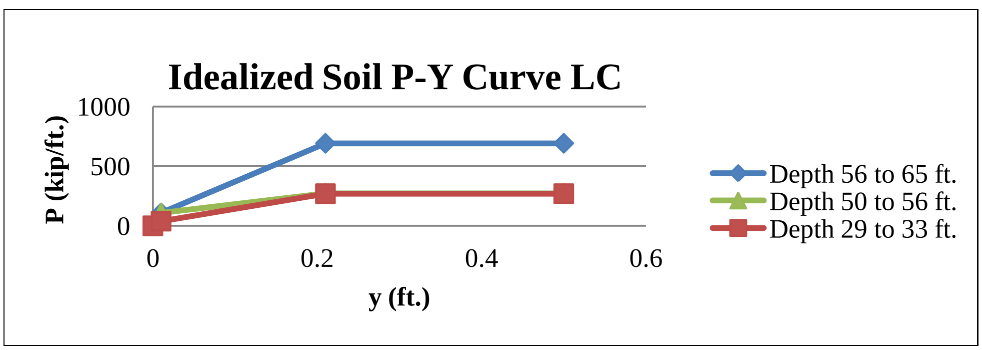


Figure 16.4-13 Idealized Soil Springs for Right Column







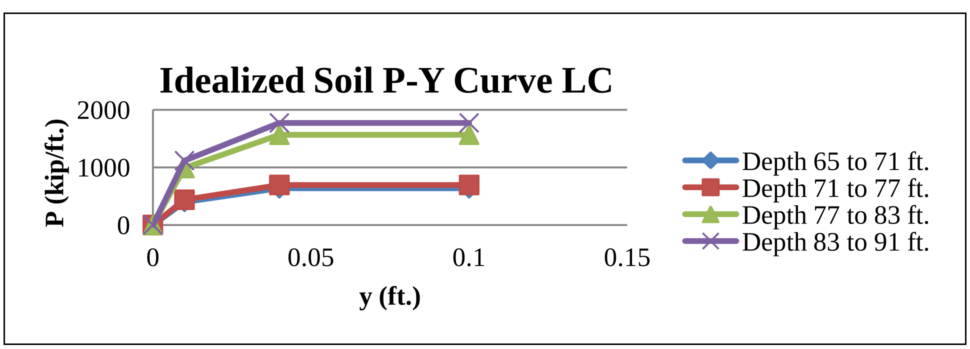


Figure 16.4-14 Idealized Soil Springs for Left Column

16.4.6.6 Ductility and Transverse Push-Over Analysis and Design

16.4.6.6.1 Displacement capacity versus demand ($\Delta_C > \Delta_D$)

CIDH Piles with permanent steel casing, column-shaft design: The concept of placing a reinforced concrete flexural element within a pipe/casing element is used in this example. The pipe/casing is installed to the desired tip elevation and cleaned out, leaving a soil plug or a rock socket in place at the tip of the pile. The column-shaft rebar cage is then placed within the pipe, followed by the concrete pour.

The pipe/casing and the reinforced concrete element are considered two parallel elements where strength or stiffness of elements can be added. Since composite action is not guaranteed, it is conservative to use the non-composite properties of the system.

The pipe/casing piles are ASTM A252, Grade 3 as required by Caltrans' Standard Specifications section 49-2.02B(3) 2010 with a minimum yield strength of 45 ksi. The yield moment of the pipe/casing is estimated based on the yield stress of 45 ksi and then is increased by 25% to represent the plastic moment of the pipe/casing. The compression portion of the pipe/casing is continuously supported against buckling.

The elongation required for ASTM A252, for Grade 3 pipe/casing material is estimated at 12% and a factor of safety of 2 is used, based on engineering judgment, to represent the minimum strain at peak stress of the pipe/casing steel at 6%.

Properties for 97 in. diameter by 0.5 in. thick pipe:

$$I_s = \frac{\pi\left(D_o^4 - D_i^4\right)}{64} = \frac{\pi\left[\left(\frac{97}{12}\right)^4 - \left(\frac{96}{12}\right)^4\right]}{64} = 8.5\ \text{ft}^4$$

$$S = \frac{I_s}{R} = \frac{8.5}{4.04} = 2.1\ \text{ft}^3$$

where R = outer radius of the pipe

D_o= outer diameter of the pipe

D_i= inner diameter of the pipe

$M_y = S\sigma_y = 2.1 \times 12^3 \times 45 = 163{,}296$ kip-in. = 13,608 kip-ft

where σ_y= (45 ksi) yield stress of the pipe

$M_p = 1.25M_y = 17{,}010$ kip-ft

$(E_sI_s)_{pipe} = (29{,}000 \times 144)8.5 = 3.55 \times 10^7$ kip-ft^2

$$\text{Design Steel Strain} = \frac{\text{Ultimate Steel Strain}}{\text{Factor of Safety}} = \varepsilon_{cu} = \frac{0.12}{2} = 0.06\ \text{rad}$$

$$\text{Design Curvature} = \phi_u = \frac{0.06}{\text{PipeRadius}} = \frac{0.06}{48.5 \text{ in.}} = 0.00123 \text{ rad/in.}$$

The concrete core and the rebar cage combine to produce a well-confined concrete element that is modeled within the xSECTION program (Caltrans, 2006) to generate section properties. The design requirements are met since the plastic hinges form in the column. Several cross sections along the column/shaft are analyzed. The cross section properties for various locations are tabulated in Table 16.4-9.

Since the steel casing of the shaft is fully extended from the top of rock to the cut-off, the confinement of shaft cross sections could be modeled with the steel casing as the lateral confining element. The hoop reinforcement, #8 at 7.5 in., is included in the confinement computation. The axial forces of 1,926 and 2,004 kips are used for the column and shaft cross-sections respectively.

60-#14 bars, A706 steel, #8-hoops at 7.5 in. spacing

Axial load = 2,004 kips

M_p = 27, 220 kip-ft

ϕ_p = 0.000465 rad/in.

I_{cr} = 82.42 ft^4

E_c = 4,032 ksi ≈ 580,000 ksf

$(E_c I_{cr})_{cage}$ = 580,000 × 82.42 = 4.78 × 10^7 kip-ft^2

For the cross section at the mid-height of the shaft casing, the following data is obtained by using xSECTION software. (See Typical xSECTION Figures 16.4-15 and 16.4-16).

The combined effects of the 2 elements, steel pipe/casing, and concrete core are as follows:

M_p = 17,010 + 27,220 = 44,230 kip-ft

$$I_{combined} = \frac{(E_s I_s)_{pipe} + (E_c I_{cr})_{cage}}{E_{concrete}} = \frac{3.55(10)^7 + 4.78(10)^7}{580{,}000} = 143.64 \text{ ft}^4$$

E_c = 4,032 ksi = 580,000 ksf

ϕ_p = 0.000465 rad/in. based on minimum of the two elements

The $I_{combined}$ is computed for converting the combined properties of steel pipe/casing and concrete core into an equivalent concrete property. These parameters are to be used in the pushover analysis later. The smaller of the curvature of steel pipe and concrete core is selected as the ductility capacity limit.

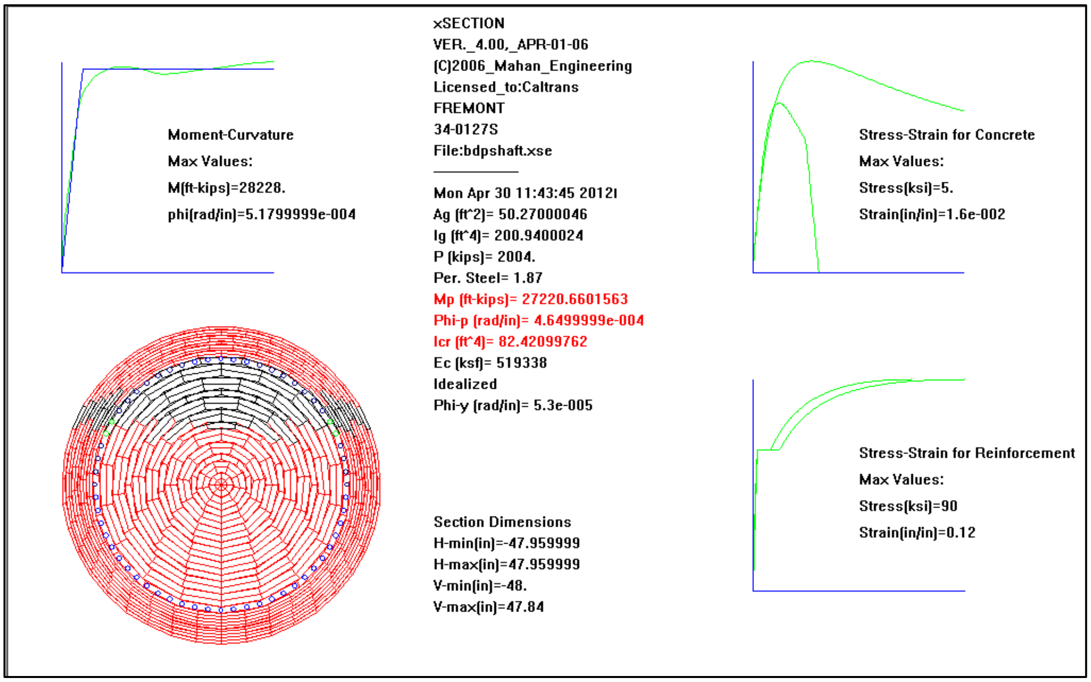


Figure 16.4-15 Shaft Cross Section with Moment Curvature with 1.87% Longitudinal Steel

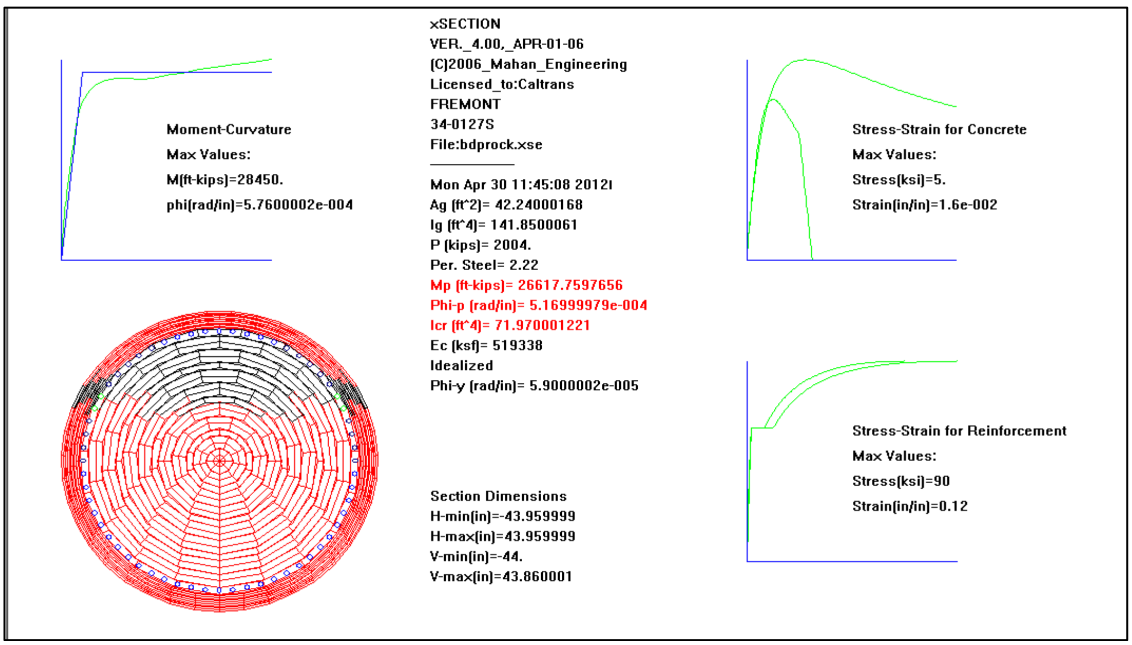


Figure 16.4-16 Shaft Cross Section with Moment Curvature in Rock Region with 2.22% Longitudinal Steel

Table 16.4-9 Cross-Section Properties for the Column and Shaft

Location	Section Composition	Concrete Core			Pipe/Casing			Combination			
		M_p kip-ft	ϕ_p rad/in.	I_{cr} ft^4	M_p kip-ft	ϕ_p rad/in.	I_{cr} ft^4	M_p kip-ft	ϕ_p rad/in.	I_{cr} ft^4	E_c ksf
Column	Core	12,309	0.000898	19.78				12,309	0.000898	19.78	580,000
Shaft	Core	27,220	0.000465	82.42	17,010	0.00123	8.5	44,230	0.000465	143.62	580,000
Shaft+Col	Shaft+Col	34,157	0.000317	95.07	17,010	0.00123	8.5	51,167	0.000317	143.62	580,000
Shaft @rock	Core only	28,508	0.000864	71.97				28,508	0.000864	71.97	580,000
Cap beam +	Unconfined conc	27,111	0.000587	86.93				27,111	0.000587	86.93	580,000
Cap beam -	Unconfined conc	25,602	0.000422	83.1				25,602	0.000422	83.1	580,000

A typical bent is then modeled within the wFRAME program (Caltrans, 1995) using the above properties and foundation springs to perform a non-linear pushover analysis.

The force-deflection curve shown in section 16.4.6.6.4 indicates that 2 plastic hinges form at the top of columns followed by 2 more plastic hinges developing at the bottom of columns as designed for a Type II shaft The period of this bent is around 1.17 seconds and the initial stiffness and the displacement demand (Δ_D) are 263 kips/in. and 9.42 in., respectively. Detailed computations are shown below:

k_i = Initial Slope of Force – Deflection Curve = Lateral Force/Yield Displacement

$k_i = (0.39 \times W)/\ \Delta_{yi} = (0.39 \times 3{,}536 \text{ kips})/5.25 \text{ in} = 263 \text{kip/in.}$

where W = total of dead load plus added dead load (kips)

$$T = \text{period} = 0.32\sqrt{\frac{W}{k_i}} = 0.32\sqrt{\frac{3{,}536}{263}} = 1.17 \quad \text{seconds}$$

Displacement demand:

$\Delta_D = (ARS \text{ x } W)/K_i = (0.7 \text{ x } 3536)/263 = 9.42$ in.

The plastic hinge length at top of column is calculated from SDC Section 7.6.2. The standard plastic hinge length (SDC 7.6.2.1) is used with the moment diagram. The rotational capacity (θ_{p}) of the plastic hinge is similar to the equations shown in SDC section 3.1.3. The plastic component of the displacement capacity, however, is based on an effective length of column defined from the center of the lower plastic hinge to the center of the upper plastic hinge as follows (Figure 16.4-17):

For the plastic hinge at top of column:

L_p = length of plastic hinge = $0.08L_1 + 0.15f_{ye}d_{bl} \geq 0.3\, f_{ye}d_{bl}$ (SDC 7.6.2.1-1)

L_1 = portion of column from plastic hinge to contraflexure (zero moment)

L_1 = 16 ft

d_{bl} = diameter of main bar = 1.63 in. for #11 bar

$L_p = 0.08 \times 16 \times 12 + 0.15 \times 68 \times 1.63 = 32 \text{ in.} \geq 0.3 \times 68 \times 1.63 = 33.25$ in.

$\theta_p = L_p \phi_p$ (SDC 3.1.3-4)

Where ϕ_p = plastic curvature capacity at top of column from cross section analysis

$\theta_p = 33.25 \times 0.000898 = 0.0298$ rad

$\Delta_p = \theta_p \times L = 0.0298 \times 30$ ft $\times$ (12 in./ft) = 10.73 in.

L = portion of column from center of top plastic hinge to center of bottom plastic hinge

$\Delta_c = \Delta_p + \Delta_y$ for each and every plastic hinge

$\Delta_c = 10.73 + 5.25 = 15.98$ in. $> \Delta_D = 9.42$ in. **OK**

The overturning effects increased the axial load from 1,926 kips to 2,695 kips in the compression column. However, the increase in plastic moment was less than 20%. There is additional reserve capacity in the cap beam to easily meet the higher column moment demand.

Comparison of displacement capacity (Δ_C) and demand (Δ_D) shows adequate ductility in the transverse direction for this particular bent ($\Delta_C > \Delta_D$).

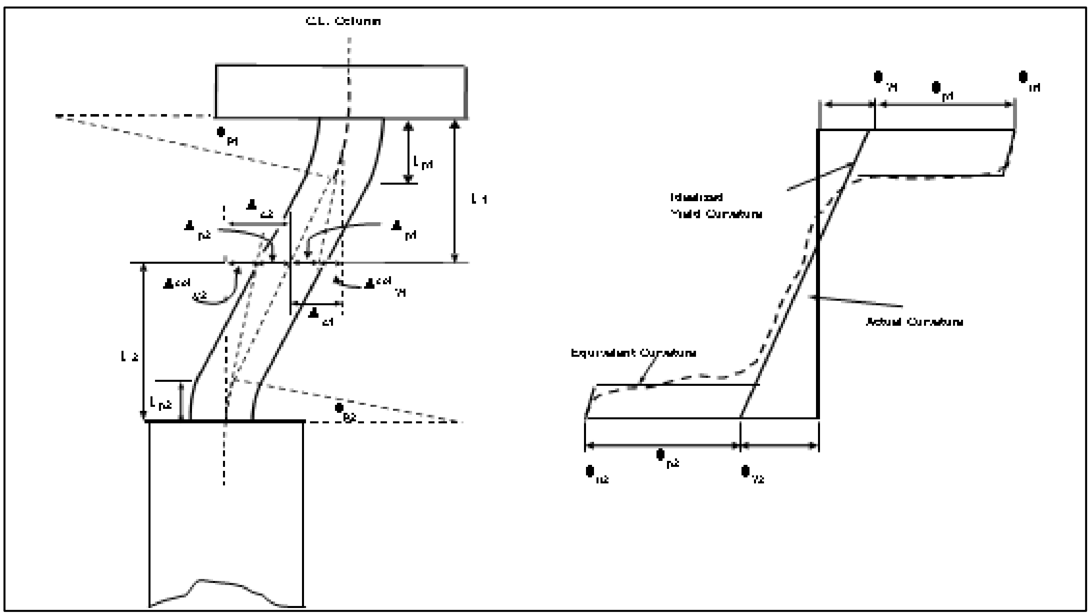


Figure 16.4-17 Plastic Hinges and Deformation

16.4.6.6.2 P-Δ Moment Check:

The P-Δ moment check is required per SDC section 4.2.

(P - Δ) Check: $M_{P-\Delta} = P(\Delta_D/2)$ = 1,926 kips (9.42 in./12)/2 = 756 kip-ft

$$\frac{M_{P-\Delta}}{M_P} = \frac{756}{12{,}309} = 0.061 < 0.2 \therefore \text{OK} \qquad \text{(SDC 4.2-1)}$$

16.4.6.6.3 Minimum Ductility Requirements

The minimum ductility requirement (SDC section 3.1.4.1) is based on the following values: The distances of L_1 and L_2 are 16 ft and 14 ft, respectively, from the moment diagram of the push over analysis per definition in SDC Figure 3.1.3-2. The yield curvature at the top and bottom of column is 0.000088 rad/in. The plastic curvature at the top and bottom is 0.000898 rad/in. (from column xSection analysis).

From SDC Eq. 7.6.2.1-1:

$L_{P1} = 0.08L_1 + 0.15f_{ye}d_{bl} \geq 0.3\, f_{ye}d_{bl} = 0.08 \times 16 \times 12 + 0.15 \times 68 \times 1.63 = 32$ in.
$< 0.3 \times 68 \times 1.63 = 33.25$ for the top plastic hinge (SDC 7.6.2.1-1)

$L_{p2} = 0.08L_2 + 0.15f_{ye}d_{bl} \geq 0.3\, f_{ye}d_{bl} = 0.08 \times 14 \times 12 + 0.15 \times 68 \times 1.63 = 30$ in.
$< 0.3 \times 68 \times 1.63 = 33.25$ for the bottom plastic hinge (SDC 7.6.2.1-1)

$\theta_{p1} = L_{p1}\phi_{p1} = 33.25 \times 0.000898 = 0.0298$ rad. (SDC 3.1.3-9)

$\theta_{p2} = L_{p2}\phi_{p2} = 33.25 \times 0.000898 = 0.0298$ rad. (SDC 3.1.3-9)

$$\Delta_{p1} = \theta_{p1} \times \left(L_1 - \frac{L_{p1}}{2} \right) = 0.0298 \times \left(16 \times 12 - 33.25/2\right) = 5.22 \text{ in.} \qquad \text{(SDC3.1.3-8)}$$

$$\Delta_{p2} = \theta_{p2} \times \left(L_2 - \frac{L_{p2}}{2} \right) = 0.0298 \times \left(14 \times 12 - 33.25/2\right) = 4.51 \text{ in.} \qquad \text{(SDC 3.1.3-8)}$$

$$\Delta_{y1}^{col} = \frac{L_1^2}{3}\phi_{y1} = \frac{(16 \times 12)^2}{3}(0.000088) = 1.08 \text{in.} \qquad \text{(SDC 3.1.3-7)}$$

$$\Delta_{y2}^{col} = \frac{L_2^2}{3}\phi_{y2} = \frac{(14 \times 12)^2}{3}(0.000088) = 0.83 \text{in.} \qquad \text{(SDC 3.1.3-7)}$$

$$\Delta_{c1} = \Delta_{y1}^{col} + \Delta_{p1} = 1.08 + 5.22 = 6.30 \text{in.} \qquad \text{(SDC 3.1.3-6)}$$

$$\Delta_{c2} = \Delta_{y2}^{col} + \Delta_{p2} = 0.83 + 4.51 = 5.34 \text{in.} \qquad \text{(SDC 3.1.3-6)}$$

$$\mu_{c1} = \frac{\Delta_{c1}}{\Delta_{y1}^{col}} = \frac{6.30}{1.08} = 5.83 > 3 \qquad \text{(SDC 3.1.4-2)}$$

$$\mu_{c2} = \frac{\Delta_{c2}}{\Delta_{y2}^{col}} = \frac{5.34}{0.83} = 6.43 > 3 \qquad \text{(SDC 3.1.4-2)}$$

16.4.6.6.4 Minimum Lateral Strength

The lateral strength of the bent is 0.39g, as calculated from the force-deflection curve of the push over analysis as shown in Figure 16.4-18. This meets the requirement of SDC section 3.5, which requires minimum lateral strength of 0.1 g.

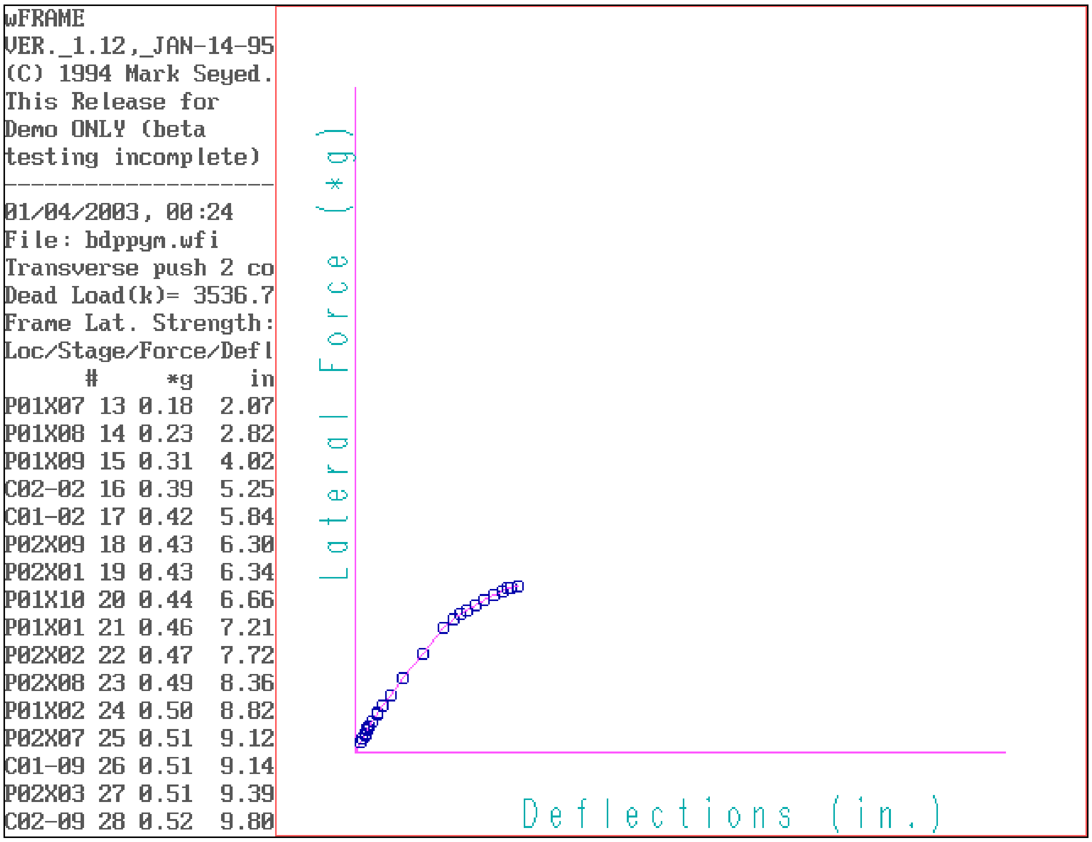


Figure 16.4-18 Lateral Load – Deflection Curve of Bent

16.4.6.7 Shear and Flexural Capacity

To meet the over-strength requirements of SDC section 4.3.1, the plastic moment of the column is increased by 20% in the push analysis to determine the larger flexural demand in the cap beam. The push analysis including over-strength of the column in addition to overturning indicated a maximum negative moment demand of 18,146 kip-ft in the cap beam, while the capacity of the cap beam in negative moment was 25,602 kip-ft

16.4.6.7.1 Shaft Shear Capacity with 0.5 in. Casing Excluding Scour and Liquefaction

The plastic shear demand in the column is limited to:

$$V_p = \frac{(M_p)_{TOP} + (M_p)_{BOT}}{(L)_{Plastic\ Himge\ to\ Plastic\ Hinge}} = \frac{(2)(12{,}309)}{30} = 820.6\ \text{kips}$$

The following cases affecting a shear increase should be considered: Increase the shear demand by an additional 20% due to overturning, V_O = 985 kips. The nominal shear capacity of the casing/pipe alone is calculated per AASHTO 6.12.1.2.3c and is less than the maximum shear demand as shown in Figure 16.4-19.

$V_{np} = 0.5F_{cr}\,A_g$ with $F_{cr} = 0.78E_s/(D/t)^{3/2} \le 0.58\ F_y$

(AASHTO 6.12.1.2.3c)

$F_{cr} = 0.78E_s/(D/t)^{3/2} = 0.78 \times 29000/(97/0.5)3/2 = 8.37$ ksi < $0.58 \times 45 = 26.1$ ksi

$V_{np} = 0.5F_{cr}\,A_g$ Nominal shear resistance (pipe or casing)

$V_{np} = 0.5 \times 8.37 \times \pi \times 97 \times 0.5 = 637.65$ kips

The shear capacity of the shaft reinforced concrete outside the plastic hinge zone must be added. For this case the traditional reinforced concrete column shear capacity equations of SDC 3.6.2 are used. In this particular case SDC Eqs. 3.6.3-1 and 3.6.3-2 yield the following for the pipe and an axial load at the top of shaft of 2,031 kips:

$$V_s = \frac{\pi A_v f_{yh} D'}{2s} = \frac{\pi(0.79)(60)(77\ \text{in.})}{(2)(7.5\,\text{in.})} = 764\ \text{kips} \qquad \text{(SDC 3.6.3-1)}$$

$$0.3 \le \text{Factor1} = \frac{\rho_s f_{yh}}{0.15(\text{ksi})} + 3.67 - \mu_d \le 3 \qquad \text{(SDC 3.6.2-5)}$$

Factor 1 = 3 outside the plastic hinge zone (SDC 3.6.2-5)

$$\text{Factor2} = 1 + \frac{P_c(\text{lb.})}{2{,}000(A_g\ \text{in.}^2)} < 1.5 \qquad \text{(SDC 3.6.2-6)}$$

$$\text{Factor2} = 1 + \frac{(2{,}031)(1{,}000)}{2{,}000(51.32)(144)} = 1.14 \qquad \text{(SDC 3.6.2-6)}$$

$$v_c\ (\text{psi}) = 3(\text{Factor2})\sqrt{f'_c}(\text{psi}) \le 4\sqrt{f'_c}(\text{psi}) \qquad \text{(SDC 3.6.2-4)}$$

$$v_c\ (\text{psi}) = 3(1.14)\sqrt{4{,}000} = 216.3\ \text{psi} < 4\sqrt{4{,}000} = 252.98\,\text{psi}$$

(SDC 3.6.2-4)

$V_c = v_c \times A_c$ where $A_c = 0.8A_g$

$V_c = 216.3(0.8A_g) = 216.3(0.8)(51.32 \times 144)/1.000 = 1{,}279$ kips

(SDC 3.6.2-1) and (SDC 3.6.2-2)

$$\phi V_n = \phi\left(V_c + V_s + V_{np}\right) = 0.9(1{,}279 + 764 + 637.65) = 2{,}413\ \text{kips} > 2{,}265\ \text{kips}$$

(SDC 3.6.1-1 and 3.6.1-2)

The shear capacity of the shaft casing and the reinforced concrete section are all added. The total shear capacity of 2,413 kips is larger than the shear demand without scour and liquefaction. It is slightly less than the maximum shear demand of **2.5*V_o*** (2,463 kips) required for the case with scour and liquefaction. To increase capacity, the designer may reduce the confinement spacing or increase the steel shell thickness of the shaft. The capacity calculated is acceptable for this example.

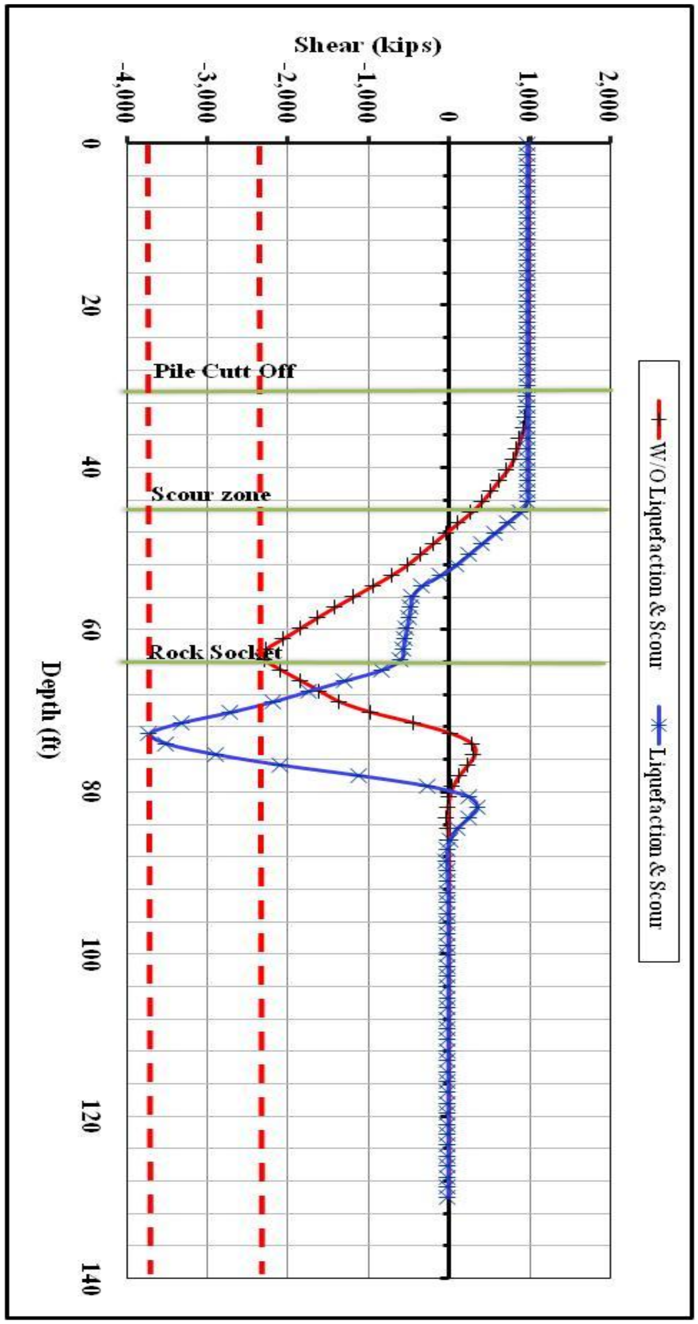


Figure 16.4-19 Shear Diagram at Potential Collapse State for Single Shaft

16.4.6.7.2 Shaft Shear Capacity at Rock Socket:

The shear capacity of the shaft without casing at the rock interface outside the plastic hinge zone must also be checked. For this case, the traditional reinforced concrete column shear capacity equations of SDC 3.6.2 are used. In this particular case SDC Eqn. 3.6.3-1 yields the following for the pipe with an increase in confinement spacing from a single #8 confinement at 7.5 in. to a doubled #8 confinement at 7 in. and an axial load at the top of rock socket section of 2,260 kips. For the case without scour or liquefaction, see L-Pile or CSiBridge Demand see Figure 16.4-19.

$$V_s = \frac{\pi A_v f_{yh} D'}{2s} = \frac{\pi(2)(0.79)(60)(77)}{(2)(7)} = 1{,}638 \text{ kips} < 2{,}265 \text{ kips} \quad \text{(SDC 3.6.3-1)}$$

$$0.3 \le \text{Factor1} = \frac{\rho_s f_{yh}}{0.15(\text{ksi})} + 3.67 - \mu_d \le 3 \quad \text{(SDC 3.6.2-5)}$$

Factor 1 = 3 outside the plastic hinge zone (SDC 3.6.2-5)

$$\text{Factor2} = 1 + \frac{P_c(\text{lb.})}{2{,}000(A_g \text{ in.}^2)} < 1.5 \quad \text{(SDC 3.6.2-6)}$$

$$\text{Factor2} = 1 + \frac{(2{,}260)(1{,}000)}{2{,}000(42.24)(144)} = 1.19 \quad \text{(SDC 3.6.2-6)}$$

$$v_c\ (\text{psi}) = 3(\text{Factor2})\sqrt{f'_c}(\text{psi}) \le 4\sqrt{f'_c}(\text{psi}) \quad \text{(SDC 3.6.2-4)}$$

$$v_c\ (\text{psi}) = 3(1.19)\sqrt{4{,}000} = 226 \text{ psi} < 4\sqrt{4{,}000} = 252.98 \text{psi} \quad \text{(SDC 3.6.2-4)}$$

$V_c = v_c \times A_c$ where $A_c = 0.8A_g$

$$V_c = 226(0.8A_g) = 226(0.8)(42.24 \times 144)/1.000 = 1{,}100 \text{ kips}$$

(SDC 3.6.2-1) and (SDC 3.6.2-2)

$$\phi V_n = \phi(V_c + V_s) = 0.9(1{,}100 + 1{,}638) = 2{,}464 \text{ kips} > 2{,}265 \text{ kips}$$

(SDC 3.6.1-1 and 3.6.1-2)

The shear demand for the case with scour and liquefaction effect as shown in Figure 16.4-19 is about 50% more than the shear capacity of the rock socket section as calculated above. Therefore, we limit the demand to be no more than **2.5V_o** (2,463 kips), which happened to be almost equal to the calculated capacity of 2,464 kips in this example.

$$v_c\ (\text{psi}) = 3(1.19)\sqrt{4{,}000} = 226 \text{ psi} < 4\sqrt{4{,}000} = 252.98 \text{psi} \quad \text{(SDC 3.6.2-4)}$$

$V_c = v_c \times A_c$ where $A_c = 0.8A_g$

$V_c = 226(0.8A_g) = 226(0.8)(42.24 \times 144)/1.000 = 1{,}100$ kips

(SDC 3.6.2-1) and (SDC 3.6.2-2)

$\phi V_n = \phi(V_c + V_s) = 0.9(1{,}100 + 1{,}638) = 2{,}464 \text{ kips} > 2{,}463 \text{ kips}$ **OK**

(2.5V_o where V_o = 985 kips)

(See L-Pile or CSiBridge Demand, see Figure 16.4-19)

16.4.6.7.3 Shaft Seismic Flexural Capacity:

The type II shaft flexural capacity under seismic loading outside the plastic hinge zone and away from the column inserted inside the type II shaft is determined from expected material properties. (Capacity protected component, per SDC 3.4)

where expected f'_{ce} = 5.0 ksi, f_{ye} = 68 ksi Using (Xsection)

The type II shaft flexural demands under seismic loading outside plastic hinge zone and away from the column inserted inside type II shaft are determined from (L-Pile) output as shown below in Figure 16.4-20 for both cases.

$M_r = \phi M_{n@0.003}$ = (1)(26,928(shaft) + 13,608(casing)) = 40,536 kip-ft > 1.25 (29,167) kip-ft (L-Pile, CSiBridge, or Wframe Demand) without liquefaction and scour demand (SDC 7.7.3.2) **OK**

$M_r = \phi M_{n@0.003}$ = (1) (26,928(shaft) + 13,608(casing) = 40,536 kip-ft < 1.25 (36,667) kip-ft (L-Pile, CSiBridge, or Wframe Demand) with liquefaction and scour demand (SDC 7.7.3.2) **NG**

The moment capacity is less than the moment demand of (1.25Md) per SDC by approximately 13% is acceptable for this example. Designers can add more reinforcement to increase the moment capacity by 13%. Designers need to be aware that the moment demand used in the example was extracted from L-Pile software and is based on a single pile, disregarding the framing action of a two-column system that can reduce the moment demand by at least by 15%.

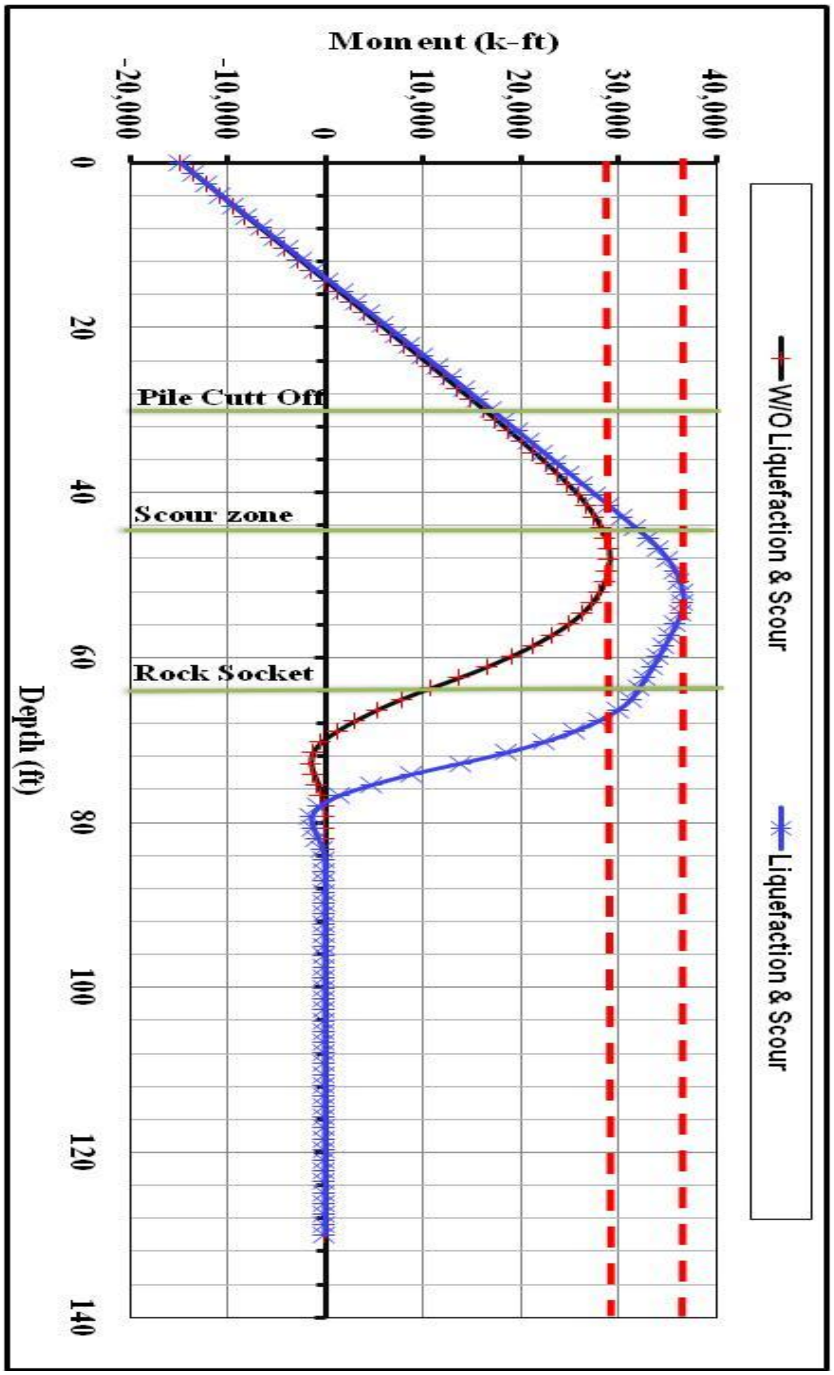


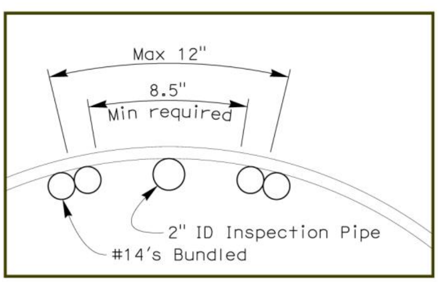
Figure 16.4-20 Moment Diagram at Potential Collapse State for Single Shaft

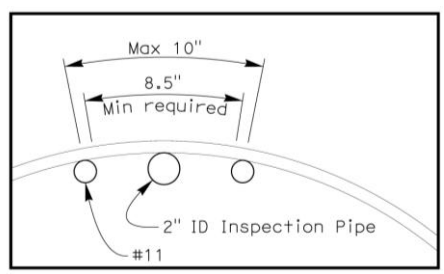
Table 16.4-10 Reinforcement Spacing Requirements in CIDH Piles with Inspection Tubes

Component	Location	Clear Spacing Requirements for Longitudinal Bars		Clear Spacing Requirements for Transverse Bars	
		Max[2] (in.)	Min (in.)	Max[3] (in.)	Min[5] (in.)
Type-I Column-Shaft with Diameter < 5ft	Column and Shaft	10-nd_b	Note 4	S_{max}-md_b	5
Type-I Column-Shaft with Diameter ≥ 5ft	Column and Shaft	12-nd_b	Note 4	S_{max}-md_b	5
Type-II Column-Shaft with Shaft Diameter < 5ft	Shaft	10-nd_b	Note 4	12	5
Type-II Column-Shaft with Shaft Diameter ≥ 5ft	Shaft	12-nd_b	Note 4	12	5

Note:

1. For Type II Shaft ≥ 5ft ϕ construction joint and permanent steel casing are mandatory, unless the drilled hole is determined by Geotechnical Services and Structure Representative to be dry.
2. n=1 is for single bars and n =2 for circumferential bundled bars. Refer to California Amendments to AASHTO LRFD Sixth Edition 10.8.1.3 and 5.13.4.5.
3. m is number of hoops in a bundle, otherwise m =1. S_{max} is maximum center-to-center (C-C) spacing of single hoops per SDC 8.2.5 Version 1.7.
4. Per MTD3-1, minimum longitudinal clear spacing of rebars is 5 in. except at locations of inspection pipes where it is 8.5 in. to accommodate the pipes





5. Minimum Clear Distance between parallel longitudinal and transverse bars is 5 in. per California Amendments to AASHTO LRFD Sixth Edition 5.13.4.5.2.

NOTATION

A = area of cross-section of a member (in.^{2}) (16.3.8)

A_b = area of individual reinforcing steel bar (in.^{2}) (16.3.10)

A_e = effective shear area of a cross-section (in.^{2}) (16.3.10)

A_g = gross cross-sectional area (in.^{2}) (16.2.3)

A_{jh}^{ftg} = effective horizontal area for a moment-resisting pile cap joint (in.^{2}) (16.2.6)

A_{ps} = area of prestressing steel (in.^{2}) (16.2.3)

ARS = 5% damped elastic Acceleration Response Spectrum, expressed in terms of g (SDC 2.1) (16.4.6.6.1)

A_s = total area of non prestressed tension reinforcement (in.^{2}) (16.2.3)

A_s' = total area of compression reinforcement (in.^{2}) (16.2.4)

A_{st} = total area of longitudinal steel (in.^{2}) (16.2.3)

A_v = area of shear reinforcement normal to flexural tension reinforcement (in.^{2}) (16.2.3)

a = depth of equivalent rectangular stress block (in.) (16.2.4)

B = coefficient used to solve a quadratic equation (16.2.4)

B_{eff}^{ftg} = effective width of the pile cap for calculating average normal stress in the horizontal direction within a pile cap moment-resisting joint (in.) (16.2.6)

b_v = effective width of a member for shear stress calculations (in.) (16.2.3)

C = coefficient used to solve a quadratic equation (16.2.4)

$C_{(i)}^{pile}$ = axial compression demand on a pile/shaft (kip) (16.2.3)

clr = minimum clearance for bottom-mat reinforcement (16.3.6.2)

D = shaft diameter (in.) (16.1.2)

D_c = column diameter (in.) (16.2.1)

D_{ftg} = depth of pile cap (in.) (16.2.1)

$D_{ftg,min}$ = minimum depth of pile cap (in.) (16.2.1)

D_{Rs} = depth of resultant soil-resistance measured from the top of pile cap (ft) (16.2.3)

D_r = diameter of the circle passing through the longitudinal reinforcement (in.) (16.2.3)

D' = cross-sectional dimension of a confined concrete core measured between the centerline of the peripheral hoop or spiral (in.) (16.3.10.2.3)

d = distance from the compression flange to the centroid of the compression steel (in.); depth below original ground (in.) (16.2.4)

d_b = nominal bar diameter (in.) (16.2.1)

d_{bd} = deformed bar diameter (in.) (16.2.1)

d_{bl} = nominal bar diameter of the longitudinal reinforcement of a shaft (in.) (16.3.2)

d_c = thickness of concrete cover measured from extreme tension fiber to center of closest bar (in.) (16.2.1)

d_e = effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement (in.) (16.2.4)

d_v = effective shear depth (in.) (16.2.3)

E_c = modulus of elasticity of concrete (ksi) (16.2.4)

E_s = modulus of elasticity of reinforcing steel (ksi) (16.3.5)

f'_c = specified 28-day compressive strength of unconfined concrete (ksi) (16.3.5)

f'_{ce} = expected compressive strength of unconfined concrete (ksi) (16.3.5)

f_{po} = average stress in prestressing steel (ksi)

f_r = modulus of rupture of concrete (ksi) (16.2.4)

f_{ss} = tensile stress in mild steel at the service limit state (ksi) (16.2.4)

f_{ue} = expected minimum tensile strength for A706 reinforcing steel (ksi) (16.3.5)

f_y = nominal yield stress for A706 reinforcing steel (ksi) (16.2)

f_{ye} = expected yield stress for A706 reinforcing steel (ksi) (16.3.2)

f_{yh} = nominal yield stress of transverse (spiral) shaft reinforcement (ksi) (16.3.10)

f_u = specified minimum tensile strength for A706 reinforcing steel (ksi) (16.3.5)

H = height of column measured from the top of the pile cap to the center of gravity of the superstructure (in.) (16.3.8)

H_c = clear height of a column measured from the top of the pile cap to the soffit of the superstructure (in.) (16.3.8)

I_{cr} = moment of inertia of the cracked cross-section of a member about its centroidal axis (in.^{4}) (16.2.4)

I_y = moment of inertia of a pile/shaft group about the y-axis (in.^{4}) (16.1.3)

k = a soil modulus parameter for sand (lb./in.^{3}) (16.3.4)

L = distance between the top and the lower points of maximum moment along a shaft: see Figure 16.3-2 (in.) (16.3.4)

L_1, L_2 = distances between the top and the lower points of maximum moment, respectively, and the point of contraflexure defined in Figure 16.3-2 (in.) (16.3.2)

L^* = effective length of the shaft segment between the top and the lower points of maximum moment along a shaft defined in Figure 16.3-3 (in.) (16.3.1)

L_{ftg} = cantilever length of the pile cap measured from the face of the column to the edge of the pile cap along the principal axis of the pile cap (in.) (16.2.1)

L_p = equivalent analytical plastic hinge length (in.) (16.3.2)

l'_d = development length of longitudinal column reinforcement (in.) (16.3.6)

l_d = development length of deformed bars in compression (in.) (16.3.6)

l_{db} = basic development length of deformed bars in compression (in.) (16.2.1)

l_{dh} = development length of deformed bars in tension (in.) (16.2.1)

l_{hb} = basic development length of hooked bars in tension (in.) (16.2.1)

M_{cap} = moment demand in the pile cap (kip-ft) (16.2.4)

M_{cr} = cracking moment of a member's cross-section (kip-ft) (16.2.4)

M_i = moment demand at the top of a *row i* shaft (kip-ft): see Figure 16.3-1 (16.3.2)

M_{iy} = moment at the top of a *row i* shaft at the formation of the *first* plastic hinge (kip-ft): see Figure 16.3-1 (16.3.2)

M_L = maximum longitudinal moment (kip-ft) (16.2)

M_n = nominal flexural resistance of a member's cross-section (kip-ft) (16.2.4)

M_{ne} = nominal moment capacity of a member's cross-section based on the *expected* material properties and concrete strain $\varepsilon_c = 0.003$ (kip-ft) (16.2.4)

M_o^{col} = overstrength moment capacity of column's cross-section (kip-ft) (16.2.3)

M_p = idealized plastic moment capacity of a member's cross-section (kip-ft) (16.1.3)

M_p^{col} = idealized plastic moment capacity of a column's cross-section (kip-ft) (16.3.8)

M_p^{shaft} = idealized plastic moment capacity of a shaft's cross-section (kip-ft) (16.3.2)

M_r = factored flexural resistance of a section in bending (kip-ft) (16.2.4)

M_T = maximum transverse moment (kip-ft) (16.2)

M_u = factored moment at a section (kip-ft) (16.2.3)

M_Y = moment at the column's base ($=\eta M_o^{col}$) associated with the formation of the *first* plastic hinge in the shaft group (kip-ft): see Figure 16.3-1 (16.1.3)

N = total number of piles/shafts in a pile/shaft group; standard blow count per foot for the California Standard Penetration Test (16.1.3)

N_u = applied factored axial force taken as positive if tensile (kip) (16.2.3)

n = modular ratio, E_s/E_c or E_p/E_c; number of individual interlocking spiral core-sections (16.3.10)

P = maximum axial force (kip) (16.1.3)

P_c = the column axial load including the effects of overturning (kip) (16.2.6)

P_{dl} = axial load attributed to dead load (kip) (16.3.2)

P_i = axial force demand on a *row i* shaft (kip): see Figure 16.3-1

P_{iy} = axial force at the top of a *row i* shaft at the formation of the *first* plastic hinge (kip): see Figure 16.3-1

P_n = nominal axial resistance of the pile/shaft (kip) (16.2.2)

P_{net} = net effective load acting on the bottom of the pile cap (kip) (16.2.2)

P_P = total axial load on a *shaft-group foundation* (kip) (16.3.2)

p_c = principal compression stress (ksi) (16.2.3)

p_t = principal tension stress (ksi) (16.2.6)

R_s = total resultant soil-resistance along the end and sides of pile cap (kip) (16.2.3)

Q = axial soil compressive resistance at the tip of a given diameter shaft (kip) (16.3.4)

s = pitch of spiral reinforcement measured along the length of the shaft (in.); spacing of reinforcing bars (in.) (16.2.4)

T_c = total tension force in the longitudinal reinforcement of a column associated with M_o (kip) (16.2.3)

T_{jv} = net tension force in a moment-resisting pile cap joint (kip) (16.2.6)

t = axial load transfer per unit length of a given diameter shaft (kip/ft) (16.3.4)

V_c = nominal shear strength provided by concrete (kip) (16.2.3)

V_i = shear demand at the top of a *row i* shaft (kip): see Figure 16.3-1

V_{iy} = shear force at the top of a *row i* shaft at the formation of the *first* plastic hinge (kip): see Figure 16.3-1

V_L = maximum longitudinal shear (kip) (16.2)

V_n = nominal shear resistance of a section (kip) (16.2.3)

V_{np} = nominal shear resistance of a pipe or casing (kip) (16.4.6)

V_p = component of the pre-stressing force in the direction of applied shear (kip) (16.2)

V_o^{col} = overstrength shear capacity of the column (kip) (16.3.2)

V_r = nominal shear resistance of cross-section (kip) (16.2.3)

V_s = nominal shear strength provided by shear reinforcement (kip) (16.2.3)

V_T = maximum transverse shear (kip) (16.2)

V_u = factored shear force at a section (kip) (16.2.3)

V_Y	=	shear force at the column's base ($=\eta V_o^{col}$) associated with the formation of the *first* plastic hinge in the shaft group (kip-ft): see Figure 16.3-1 (16.3.2)
x	=	distance from the compression flange to the neutral axis (in.) (16.1.3)
y	=	lateral deflection of a shaft at a specific depth (in.) (16.1.3)
z	=	vertical deflection of a shaft at a specific depth (in.) (16.2.3)
α	=	angle of inclination of transverse reinforcement to longitudinal axis (16.2.3)
β	=	factor relating effect of longitudinal strain on the shear capacity of concrete; ratio of long side to short side of footing (16.2.3)
β_s	=	ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face (16.2.4.2)
v_c	=	permissible shear stress carried by concrete (psi) (16.3.10)
v_{jv}	=	nominal shear stress in a moment-resisting joint (ksi)
v_u	=	average factored shear stress on concrete (ksi) (16.2.2)
ε_{cc}	=	confined concrete compressive strain at maximum compressive stress (16.3.5)
ε_{co}	=	unconfined concrete compressive strain at maximum compressive stress (16.3.5)
ε_{cu}	=	confined concrete ultimate compressive strain (16.3.5)
ε_{sh}	=	tensile strain at the onset of strain hardening for A706 reinforcement (16.3.5)
ε_{sp}	=	unconfined concrete ultimate compressive strain (spalling strain) (16.3.5)
ε_{su}	=	ultimate tensile strain of A706 reinforcement (16.3.5)
ε^R_{su}	=	reduced ultimate tensile strain of A706 reinforcement (16.3.5)
ε_{ye}	=	expected yield tensile strain of A706 reinforcement (16.3.5)
Δ_c	=	local member displacement capacity (in.) (16.3.2)
Δ_C	=	global displacement capacity of a shaft group (in.) (16.3.2)
Δ_D	=	global displacement demand of a shaft group (in.) (16.3.2)
Δ_{pi}	=	idealized local plastic displacement capacity of a shaft due to the rotation of the *i*th plastic hinge (in.): see Figure 16.3-2 (16.3.2.1.3)
Δ_{Pi}	=	global plastic displacement capacity of a shaft group due to the plastic rotation capacity of the *i*th plastic hinge (in.): see Figure 16.3-3
Δ_r	=	relative lateral offset between the top of the shaft and the lower point of maximum moment in a shaft (in.) (16.3.2)
Δ_{Yi}	=	yield displacement of a shaft group at the formation of the i^{th} plastic hinge in the shafts (in.): see Figure 16.3-2 (16.3.2.1.3)

Δ_{yi}^{shaft} = idealized local yield displacement of the i^{th} shaft (in.) (16.3.10)

Δ_{y2}^{shaft} = (16.3.10.1.2)

ϕ = angle of internal friction (16.2.3)

ϕ = strength reduction factor

ϕ_p = idealized plastic curvature of a cross-section (rad/in.) (16.2.3)

ϕ_Y = idealized yield curvature of a cross-section (rad/in.) (16.3.2)

γ_e = crack control exposure condition factor (16.2.4)

γ_p = load factor for permanent loads (16.2.2)

γ_t = unit weight of soil (lb./ft^3) (16.3.4)

η = non-dimensional constant defined in Figure 16.3-1(16.3.2)

μ_c = local displacement ductility capacity (16.3.2)

μ_d = local displacement ductility demand (16.3.10)

μ_D = global displacement ductility demand (16.3.2)

θ = angle of inclination of diagonal compressive stresses (rad) (16.2.3)

= angle defined in Figure 16.3-3 (rad)

θ_p = plastic rotation capacity (rad) (16.3.2)

ρ_s = ratio of volume of spiral reinforcement to the core volume confined by the spiral reinforcement (measured out-to-out) for circular cross-sections (16.3.10)

ψ = angle defined in Figure 16.3-3 (rad) (16.3.2)

REFERENCES

1. AASHTO, (2012). *AASHTO LRFD Bridge Design Specifications*, Customary U.S. Units (6th Edition), American Association of State Highway and Transportation Officials, Washington, DC.
2. AISC, (2010). *Specification for Structural Steel Buildings Manual*, American Institute of Steel Construction Inc., Chicago, IL
3. Caltrans, (1984). *Bridge Design Details 13-10: Reinforcing Bar Data – Grade 60* (*ACI 318-83*), California Department of Transportation, Sacramento, CA, July 1984.
4. Caltrans, (1990). *Bridge Design Details 6-71: Pile Footings*, California Department of Transportation, Sacramento, CA, June 1990.
5. Caltrans, (1995). *wFRAME, Push Analysis of Bridge Bents and Bridge Frames – Version 1.12*, California Department of Transportation, Sacramento, CA.
6. Caltrans, (2006). *xSECTION, Moment-Curvature Curves for Reinforced Concrete Structural Elements – Version 4.00,*, California Department of Transportation, Sacramento, CA.
7. Caltrans, (2014 a). *California Amendments to AASHTO LRFD Bridge Design Specifications – Sixth Edition*, California Department of Transportation, Sacramento, CA.
8. Caltrans, (2014b). *Bridge Memo to Designers 3-1: Deep Foundations*, California Department of Transportation, Sacramento, CA, June 2014.
9. Caltrans, (2008). *Bridge Memo to Designers 1-35: Foundation Recommendation and Reports*, California Department of Transportation, Sacramento, CA, June 2008.
10. Caltrans, (2010). *Bridge Memo to Designers 20-1: Seismic Design Methodology*, California Department of Transportation, Sacramento, CA, July 2010.
11. Caltrans, (2013). *Caltrans Seismic Design Criteria*, Version 1.7, California Department of Transportation, Sacramento, CA, April 2013.
12. Chen, W. F. and Duan, L. (2000). *Bridge Engineering Handbook*, Chapter 9, 15-16, CRC Press, Boca Raton, FL.
13. CSI, (2015). *CSiBridge 2015, v. 17.0.0*, Computers and Structures, Inc., Walnut Creek, CA.
14. Gebman, M. J. and Restrepo, A. J. (2006). "Axial Force Transfer Mechanisms within Cast-In Steel Shell Piles." *Report: SSRP-06/16*. University of California, San Diego, La Jolla, CA.
15. Parker, F., Jr. and Reese, L. C. (1971). "Lateral Pile-Soil Interaction Curves for Sand." *Proceedings of the International Symposium on Engineering Properties of Sea-Floor Soils and their Geophysical Identification*, UNESCO-University of Washington, Seattle, WA, July 1971, 212-223.
16. Reese, L. C. Wang, S. T., Isenhower, W., and Arrellega, J. A. (2007). "Computer Program LPILE Plus Version 5.0.38." *A Program for the Analysis of Piles and Drilled Shafts under Lateral Loads, Technical Manual*, Ensoft, Inc., Austin, TX.

17. Seyed, M. (1995). Computer Programs *w*-FPREP and *w*-FRAME Version 1.12 — *A Program for Two Dimensional Push Analysis of Bridge Bents and Frames*, Sacramento, CA.

18. Yashinsky, M. et al. (2007). *Pile Group Foundation Study*, California Department of Transportation, Sacramento, CA, November 2007.

19. Malek, A. M., and Islam M. S. (2010), "Seismic Analysis and Design of Pile Group Bridge Foundations in Soft and Liquefied Soil." *Journal of the Transportation Research Board*, No. 2202, 183-191.

This page is intentionally left blank.