Parametric study of depth to maximum bending moment for prestressed concrete piles resisting lateral load

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- Seismic design of prestressed concrete piles requires transverse spiral reinforcement to be proportioned to confine and thus retain an intact concrete core during inelastic rotation cycles.
- This study used lateral analysis software for a parametric study to determine a practical maximum depth to maximum in-ground bending moment for 12, 14, and 16 in. (300, 360, and 410 mm) square and octagonal prestressed concrete piles subjected to displacement consistent with lateral seismic loads.
- Analysis of results indicates that prescriptive depth requirements may be overly conservative for 12 and 14 in. (300 and 360 mm) square piles in nonliquefiable soils, and a reduction could be made to the code-prescribed depth of ductile reinforcement for some pile types in nonliquefiable conditions.

The purpose of ductile confinement reinforcement in prestressed concrete piles is to contain the core concrete, thus maintaining pile strength during and after a seismic event. Confinement reinforcement is necessary only when required concrete strain exceeds unconfined strain limits, which otherwise lead to material loss and abruptly diminishing strength. Further, such strain is only necessary for portions of the pile undergoing extreme rotation coincident with maximum moment, which can be assumed to increase up to a point where stable plastic hinging occurs.[1,2] The manifestation of the plastic hinge in prestressed piles occurs over a length of the pile, described as a plastic hinge zone, rather than at a discrete point.[3,4] The *International Building Code*[5] (IBC) provides a simple approach to specifying ductile confinement reinforcing in the upper 35 ft (11 m) of piles in seismic design categories D, E, and F. In addition to prescriptive requirements, the IBC also allows for project-specific curvature analysis to determine a reduction in the requirements for ductile reinforcement. Prior to this research, the authors of this paper conducted many project-specific curvature-based pile designs in a wide variety of soil conditions. Although not comprehensive, the results of that work suggest that the minimum prescriptive in-ground depth for ductile reinforcement may be excessive and that a more exhaustive study may inform a reduced prescriptive depth requirement.

To investigate, a parametric study was conducted to explore all possible variables related to curvature in laterally loaded piles for all practical general soil conditions to determine the possible range of depth to in-ground maximum moment. Industry-standard modeling techniques were used to investigate 12, 14, and 16 in. (300, 360, and 410 mm) prestressed concrete piles subjected to displacement consistent with and in excess of design-level seismic loads. Parameters of the models are soil properties, prestressed concrete pile properties, pile stickup, and pile head boundary conditions. Resulting maximum depth to the maximum in-ground moment is reported for distinct conditions, such as pile size, liquefaction potential, and boundary condition at the top of the pile. Recommendations are directly related to the analyzed depths to maximum in-ground moment and length of plastic hinge zone.

Background

Seismic design of prestressed concrete piles requires transverse spiral reinforcement to be proportioned to confine and thus retain an intact concrete core during inelastic rotation cycles. Prescriptive provisions of the IBC require a minimum volumetric ratio of confinement reinforcement relative to the pile cross section and a minimum in-ground depth to which the confinement reinforcement must extend; 20 ft (6 m) for seismic design category C and 35 ft (11 m) for categories D, E, and F. In addition, half of the prescriptive volumetric ratio of transverse reinforcing is required to extend to the toe of the pile. The level of prescriptive detailing is based entirely on the seismic design category of the building without regard to specified pile loading or soil type and stratigraphy.

These requirements can be traced back to the PCI Prestressed Concrete Piling Committee "Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling."6 At the time of the development of the PCI recommended practice, the literature lacked substantial research regarding confinement reinforcement in piles. Consequently, the prescriptive requirements for the volumetric ratio of transverse reinforcing steel and in-ground depth to which prescribed steel must extend have been excessively but necessarily conservative. Since the publication of the PCI recommended practice, significant research has been conducted to better understand the volumetric ratio of reinforcement required for adequate confinement of the core of the pile where inelastic rotation is expected.[7,8] Current updated prescriptive requirements are found in the American Concrete Institute's *Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318-19R)*[9] and the IBC, where the volumetric ratio of steel has been updated based on Sritharan, Cox, Huang, Suleiman, and Arulmoli.[8] However, the authors are unaware of research conducted to establish the maximum in-ground depth at which inelastic rotation is possible for piles subjected to lateral load. The purpose of this research is to establish maximum in-ground depth of plastic hinging that is possible for 12, 14, and 16 in. (300, 360, and 410 mm) prestressed concrete piles subjected to lateral loading. Piles are investigated analytically in a substantially exhaustive set of soil conditions, representing the practical range of possible soil-pile interaction.

Parametric analytical study

A series of incremental nonlinear laterally loaded pile simulations were performed on 12, 14, and 16 in. (300, 360, and 410 mm) rectangular and octagonal prestressed concrete piles in a substantially varied array of soil profiles. These analyses were displacement controlled, simulating static lateral force applications at the top of the pile. Soil profiles were chosen to represent all possible soil conditions with respect to lateral soil stiffness, effectively enveloping all practical possibilities for depth to the location of maximum bending moment in the pile. Free-field soil interaction and cyclic response were not simulated. Elastic pile behavior was considered throughout all simulations. An initial set of variations of parameters were considered that would have required more than 37,632 simulations. In the interest of limiting the study to a more reasonable data set while maintaining a substantially representative set of results with respect to determining the depth to the maximum bending moment, the number of soil profiles and pile parameters considered was reduced based on redundancy or reliable incorporation within maximum and minimum results. The reduced data set resulted in 5544 simulations representing 77 soil profiles, 4 loading variations, and 72 pile variations. Output data for 8 variations of displacement-controlled lateral load and two pile head conditions were recorded per simulation. Parameters considered are listed with initial and reduced numbers of variations.

- pile size: six variations and no reduced variations
- concrete strength: 4 initial variations and 3 reduced variations
- bending stiffness percentage: 4 variations and no reduced variations
- pile stickup: 2 initial variations and 1 reduced variation
- soil profile: 196 initial variations and 77 reduced variations
- lateral load (multiple variables are incorporated into a single simulation): 8 variations and no reduced variations
- pile head condition (multiple variables are incorporated into a single simulation): 2 variations and no reduced variations

Analytical model

Figure 1 shows a schematic representation of the analytical model. A general-use pile lateral analysis software was used for all simulations. Input structural parameters of the

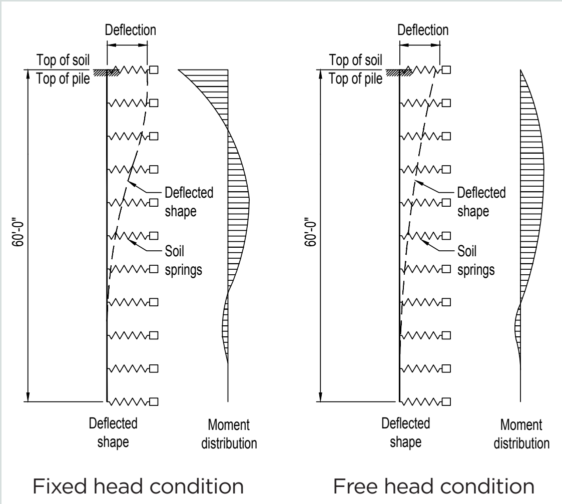

Figure 1. Analytical model with generalized moment distribution. Note: 1 ft = 0.305 m.

piles included concrete compressive stress, pile diameter, and elastic bending stiffness. Nonlinear lateral response of the soil was simulated using nonlinear p-y curves predefined within the lateral analysis software for general soil types, with minor modification as described herein. Displacement control was used for all simulations in which 0.5 in. (13 mm) displacement increments were used up to 4 in. (100 mm) maximum. No axial load was applied to the piles. Nonlinear structural response of the pile was not considered.

Pile parameters

Six sizes of piles were chosen for the study based on their widespread use in low- to mid-rise buildings and other structures: 12, 14, and 16 in. (300, 360, and 410 mm) square and octagonal piles. **Tables 1** and **2** provide corresponding pile section properties. Piles were modeled as plain concrete without reinforcement. The 28-day compressive strengths of the concrete f_c' were taken as 5000, 6000, 8000, and 10,000 psi (34, 41, 55, and 69 MPa). To simulate a possible range of reduced bending stiffness due to cracking, the moment of inertia of the pile was defined as a percentage of gross moment of the pile section. The gross moment of inertia was factored by 0.25, 0.5, 0.75, and 1.0. Free- and fixed-head conditions were analyzed for all models.

Soil model

Due to the infinite number of possible permutations for specific soil stratigraphy, this study does not attempt to capture all possible soil profiles with specificity relative to all soil parameters. Rather, the selection of soil parameters and variation of soil stratigraphy for this study focuses solely on practical variations of lateral pile response. To this end, soil types were defined based on general soil properties per Table 3. Soil models were created by varying the soil type used in each layer, the number of layers, and the depth to the boundaries of layers such that depth to maximum in-ground moment would be enveloped for all practical soil profiles.

The soil parameters used were based on ASCE 7[10] soil property definitions (**Table 3**) for general soil types. The sands

Table 1. Square concrete pile properties

f_c', psi	Pile size, in.	Equivalent diameter, in.	1.0*I*, in.⁴	0.75*I*, in.⁴	0.5*I*, in.⁴	0.25*I*, in.⁴	1.0*IE*, lb-in.² × 10⁹	0.75*IE*, lb-in.² × 10⁹	0.5*IE*, lb-in.² × 10⁹	0.25*IE*, lb-in.² × 10⁹
5000	12	13.54	1728	1296	864	432	6.965	5.224	3.482	1.741
	14	15.80	3201	2401	1601	800	12.903	9.677	6.452	3.226
	16	18.05	5461	4096	2731	1365	22.012	16.509	11.006	5.503
6000	12	13.54	1728	1296	864	432	7.629	5.722	3.815	1.907
	14	15.80	3201	2401	1601	800	14.135	10.601	7.067	3.534
	16	18.05	5461	4096	2731	1365	24.113	18.085	12.056	6.028
8000	12	13.54	1728	1296	864	432	8.810	6.607	4.405	2.202
	14	15.80	3201	2401	1601	800	16.321	12.241	8.161	4.080
	16	18.05	5461	4096	2731	1365	27.843	20.882	13.922	6.961
10,000	12	13.54	1728	1296	864	432	9.850	7.387	4.925	2.462
	14	15.80	3201	2401	1601	800	18.248	13.686	9.124	4.562
	16	18.05	5461	4096	2731	1365	31.130	23.347	15.565	7.782

Note: *E* = modulus of elasticity; f_c' = 28-day concrete compressive strength; *I* = moment of inertia. 1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = 6.895 kPa.

Table 2. Octagonal concrete pile properties

f'_c, psi	Pile size, in.	Equivalent diameter, in.	1.0*I*, in.^{4}	0.75*I*, in.^{4}	0.5*I*, in.^{4}	0.25*I*, in.^{4}	1.0*IE*, lb-in.^{2} × 10^9	0.75*IE*, lb-in.^{2} × 10^9	0.5*IE*, lb-in.^{2} × 10^9	0.25*IE*, lb-in.^{2} × 10^9
5000	12	12.31	1134	851	567	284	4.571	3.428	2.285	1.143
	14	14.36	2105	1579	1053	526	8.484	6.363	4.242	2.121
	16	16.43	3592	2694	1796	898	14.478	10.858	7.239	3.619
6000	12	12.31	1134	851	567	284	5.007	3.755	2.503	1.252
	14	14.36	2105	1579	1053	526	9.294	6.970	4.647	2.323
	16	16.43	3592	2694	1796	898	15.859	11.895	7.930	3.965
8000	12	12.31	1134	851	567	284	5.781	4.336	2.891	1.445
	14	14.36	2105	1579	1053	526	10.732	8.049	5.366	2.683
	16	16.43	3592	2694	1796	898	18.313	13.735	9.156	4.578
10,000	12	12.31	1134	851	567	284	6.464	4.848	3.232	1.616
	14	14.36	2105	1579	1053	526	11.999	8.999	5.999	3.000
	16	16.43	3592	2694	1796	898	20.474	15.356	10.237	5.119

Note: *E* = modulus of elasticity; f'_c = 28-day concrete compressive strength; *I* = moment of inertia. 1 in. = 25.4 mm; 1 lb = 4.448 N; 1 psi = 6.895 kPa.

were classified as loose, medium, or dense by varying the unit density γ and the friction angle θ. The clays were organized as soft, medium, stiff, very stiff, or hard by varying the unit density γ, the undrained shear strength S_u (cohesion), and the small strain factor E_{50}. Predefined values of the lateral analysis software were used for the small strain factor of the soft and medium clays. All values were kept constant through each soil layer except for the undrained shear strength of clays. For soft and medium clays, the shear strength varied with depth using S_u equal to $0.22\gamma'$ (OCR^m) (where *OCR* is overconsolidated ratio and $m = 1 - \sin\theta$), with a limiting maximum value equal to 2000 lb/ft^2 (96 kPa). This was modeled by entering a linearly varying shear strength for soft or medium clay layers based on values calculated using undrained shear strength at the top and bottom of the layer.

Initially, the most widely varying set of soil type and layering combinations were considered to a depth below grade equal to 60 ft (18 m), referred to as set 1. Between one and four layers were used for each soil profile, such that the profiles ranged from a 60 ft deep monolithic profile to four layers with a depth totaling 60 ft. Relative layer depths were varied in increments of 10 ft (3 m), and soils were varied in various combinations, resulting in eight general soil layering groups (**Tables 4** and **5**). Each soil model group consists of 98 subgroups accounting for variations of soil type, soil strength, layer arrangement, and layer thickness. For each subgroup, both saturated and unsaturated conditions were considered. The resulting total number of soil profiles was 196, which would require 784 total lateral analysis software simulations per pile type, considering two pile stickup and two head fixity variations, free and fixed. The resulting total number of simulations was considered impractical and unnecessary. To investigate soil subgroups that could be eliminated from the study due to redundancy, piles with maximum and minimum bending stiffness were simulated in each of the 784 configurations described. This included simulations of the 12 in. (300 mm), octagonal, 5000 psi (34 MPa) and 16 in. (410 mm) square, 10,000 psi (69 MPa) piles. All soil models were run with both fixed-head and free-head conditions, with no stickup and with 5 ft (1.5 m) of stickup. Soil profiles with consistently redundant results as related to the primary response parameter of interest, depth to maximum in-ground moment, were identified, and the number of soil profiles was reduced from 196 to 77. Soil profiles retained for the broader study of all pile types considered are referred to as set 2 (Table 5).

Findings

Results of simulations of all piles on soil profile set 2 were analyzed to determine trends related to pile and soil parameters. **Table 6** reports the maximum depth to maximum in-ground moment for each pile size and each cross-sectional type. Results in the table are separated into soil profiles with liquefied layers and soil profiles without liquefied layers. A comprehensive table of values for depth to in-ground maximum moment for all set 2 simulations, along with lateral analysis software output for the pile types analyzed with soil set 1, is available in Pinto et al.[11]

The following general observations were made:

- Depth to maximum in-ground moment increases with increased bending stiffness of piles across all soil profiles. Therefore, depth to maximum in-ground moment increased as the moment of inertia, pile strength, and percentage of the effective section were increased.

- Octagonal piles exhibited relatively greater depth to maximum in-ground moment compared with rectangular piles of the same dimension.

- Depth to maximum in-ground moment increases as pile head deflection increases for all free-head piles and most fixed-head piles. A general outlier to this trend occurs in soil profiles with stiffer soils at the top of the profile overlaying weaker soils, such as profile groups 2, 5, and 8.

- Fixed-head boundary conditions result in greater depth to maximum in-ground moment than free-head boundary conditions.

- Free-head piles result in a greater magnitude of in-ground

Table 3. Soil properties

	ASCE 7 site class	Average shear wave velocity V_s, ft/sec	Average field standard penetrations resistance for 100 ft	Undrained shear strength S_u, lb/ft^2	Soil type	Soil type	Soil properties used for analyses		
							θ, deg	γ, lb/ft^3	
Sand	C: very dense soil and soft rock	1200 to 2500	>50	>2000	Dense sand	Sand (Reese)	40	125	
Sand	D: stiff soil	600 to 1200	15 to 50	1000 to 2000	Medium sand	Sand (Reese)	35	118	
Sand	E: soft clay soil	<600	<15	<1000	Loose sand	Sand (Reese)	30	110	
	ASCE 7 site class	**Average shear wave velocity V_s, ft/sec**	**Average field standard penetrations resistance for 100 ft**	**Undrained shear strength S_u, lb/ft^2**	**Soil type**	**Soil type**	**Soil properties used for analyses**		
							C, lb/ft^2	**γ, lb/ft^3**	**E_{50}**
Clay	C: very dense soil and soft rock	1200 to 2500	>50	>2000	Hard clay	Stiff clay without free water (Reese)	4000	120	0.004
Clay					Very stiff clay	Stiff clay without free water (Reese)	2000	120	0.005
Clay	D: stiff soil	600 to 1200	15 to 50	1200 to 2000	Stiff clay	Moderately stiff clay without free water	1000	110	0.007
Clay	E: soft clay soil	<600	<15	<1000	Medium clay	Moderately stiff clay without free water	500	100	Default
Clay					Soft clay	Soft clay (Matlock)	250	90	Default

Note: Some soil types are based on soil resistance versus pile deflection (*p-y*) profiles developed by Matlock and Reese, as indicated. C = cohesion; E_{50} = small strain factor; γ = unit density of soil; θ = friction angle. 1 ft = 0.305 m; 1 lb = 4.448 N.

Table 4. Soil profiles for set 1

Profile	Layer 1		Layer 2		Layer 3	
	Thickness, ft	Soil type	Thickness, ft	Soil type	Thickness, ft	Soil type
1a	60	Liquid sand				
1d	60	Dense sand				
1g	60	Stiff clay				
1h	60	Very stiff clay				
1i	60	Hard clay				
2c	50	Medium sand	10	Very stiff clay		
2e	50	Medium clay	10	Very stiff clay		
2f	40	Liquid sand	20	Very stiff clay		
2g	40	Loose sand	20	Very stiff clay		
2h	40	Medium sand	20	Very stiff clay		
2i	40	Soft clay	20	Very stiff clay		
2j	40	Medium clay	20	Very stiff clay		
2l	30	Loose sand	30	Very stiff clay		
2m	30	Medium sand	30	Very stiff clay		
2n	30	Soft clay	30	Very stiff clay		
2o	30	Medium clay	30	Very stiff clay		
3g	40	Very stiff clay	20	Loose sand		
3h	40	Very stiff clay	20	Medium sand		
3i	40	Very stiff clay	20	Soft clay		
3j	40	Very stiff clay	20	Medium clay		
3k	30	Very stiff clay	30	Liquid sand		
3l	30	Very stiff clay	30	Loose sand		
3m	30	Very stiff clay	30	Medium sand		
3n	30	Very stiff clay	30	Soft clay		
3o	30	Very stiff clay	30	Medium clay		
4n	10	Loose sand	40	Medium clay	10	Very stiff clay
4m	10	Liquid sand	40	Medium clay	10	Very stiff clay
4o	10	Soft clay	40	Medium clay	10	Very stiff clay
4p	10	Liquid sand	40	Medium sand	10	Very stiff clay
4q	10	Loose sand	40	Medium sand	10	Very stiff clay
4r	10	Soft clay	40	Medium sand	10	Very stiff clay
4s	20	Liquid sand	20	Medium clay	20	Very stiff clay
4t	20	Loose sand	20	Medium clay	20	Very stiff clay
4u	20	Soft clay	20	Medium clay	20	Very stiff clay
4v	20	Liquid sand	20	Medium sand	20	Very stiff clay

Table 4. Soil profiles for set 1 (cont.)

4w	20	Loose sand	20	Medium sand	20	Very stiff clay
4x	20	Soft clay	20	Medium sand	20	Very stiff clay
5b	10	Very stiff clay	30	Medium clay	20	Loose sand
5c	10	Very stiff clay	30	Medium clay	20	Soft clay
5e	10	Very stiff clay	30	Medium sand	20	Loose sand
5f	10	Very stiff clay	30	Medium sand	20	Soft clay
5h	20	Very stiff clay	30	Medium clay	10	Loose sand
5i	20	Very stiff clay	30	Medium clay	10	Soft clay
5j	20	Very stiff clay	30	Medium sand	10	Liquid sand
5k	20	Very stiff clay	30	Medium sand	10	Loose sand
5l	20	Very stiff clay	30	Medium sand	10	Soft clay
5n	10	Very stiff clay	40	Medium clay	10	Loose sand
5o	10	Very stiff clay	40	Medium clay	10	Soft clay
5q	10	Very stiff clay	40	Medium sand	10	Loose sand
5r	10	Very stiff clay	40	Medium sand	10	Soft clay
5t	20	Very stiff clay	20	Medium clay	20	Loose sand
5u	20	Very stiff clay	20	Medium clay	20	Soft clay
5v	20	Very stiff clay	20	Medium sand	20	Liquid sand
5w	20	Very stiff clay	20	Medium sand	20	Loose sand
5x	20	Very stiff clay	20	Medium sand	20	Soft clay
7a	10	Loose sand	40	Liquid sand	10	Loose sand
7b	10	Soft clay	40	Liquid sand	10	Soft clay
8b	50	Liquid sand	10	Soft clay		
8c	50	Liquid sand	10	Medium sand		
8d	50	Liquid sand	10	Medium clay		

Note: 1 ft = 0.305 m.

moment than fixed-head piles for the same deflection at the top of the pile.

- Softer soils result in a greater magnitude of in-ground moments than stiffer soils.
- Changes in soil density to account for buoyancy in the lateral analysis software did not significantly affect the results.
- The soil profiles with relatively deeper layers of very weak soil overlaying a very stiff soil resulted in the greatest depth to maximum in-ground moment.

All values provided in Table 6 represent results of simulations with uncracked sections where the bending stiffness is equal to 1.0EI (where E is modulus of elasticity and I is moment of inertia) and maximized concrete compressive strength f_c' is equal to 10,000 psi (69 MPa). Therefore, it is reasoned that the maximum depths represented in Table 6 would provide an upper-bound limit for piles of like geometry with equal or lesser effective bending stiffness for all conventional square and octagonal prestressed concrete piles.

Boundary condition assumptions are also a crucial parameter for determining the depth to maximum in-ground moment, with the maximum and minimum limits corresponding to fixed- and free-head conditions, respectively. Practical rotational restraint at the pile head occurs somewhere between these simplified boundary assumptions. Similarly, depth to maximum in-ground moment would fall between simulated values using the models described herein. Therefore,

Table 5. Soil profiles for set 2

Profile	Layer 1		Layer 2		Layer 3		Layer 4	
	Thickness, ft	Soil type	Thickness, ft	Soil type	Thickness, ft	Soil type	Thickness, ft	Soil type
1b	60	Loose sand						
1c	60	Medium sand						
1e	60	Soft clay						
1f	60	Medium clay						
2a	50	Liquid sand	10	Very stiff clay				
2b	50	Loose sand	10	Very stiff clay				
2d	50	Soft clay	10	Very stiff clay				
2k	30	Liquid sand	30	Very stiff clay				
3a	50	Very stiff clay	10	Liquid sand				
3b	50	Very stiff clay	10	Loose sand				
3c	50	Very stiff clay	10	Medium sand				
3d	50	Very stiff clay	10	Soft clay				
3e	50	Very stiff clay	10	Medium clay				
3f	40	Very stiff clay	20	Liquid sand				
4a	10	Liquid sand	30	Medium clay	20	Very stiff clay		
4b	10	Loose sand	30	Medium clay	20	Very stiff clay		
4c	10	Soft clay	30	Medium clay	20	Very stiff clay		
4d	10	Liquid sand	30	Medium sand	20	Very stiff clay		
4e	10	Loose sand	30	Medium sand	20	Very stiff clay		
4f	10	Soft clay	30	Medium sand	20	Very stiff clay		
4g	20	Liquid sand	30	Medium clay	10	Very stiff clay		
4h	20	Loose sand	30	Medium clay	10	Very stiff clay		
4i	20	Soft clay	30	Medium clay	10	Very stiff clay		
4j	20	Liquid sand	30	Medium sand	10	Very stiff clay		
4k	20	Loose sand	30	Medium sand	10	Very stiff clay		
4l	20	Soft clay	30	Medium sand	10	Very stiff clay		
5a	10	Very stiff clay	30	Medium clay	20	Liquid sand		
5d	10	Very stiff clay	30	Medium sand	20	Liquid sand		
5g	20	Very stiff clay	30	Medium clay	10	Liquid sand		
5m	10	Very stiff clay	40	Medium clay	10	Liquid sand		
5p	10	Very stiff clay	40	Medium sand	10	Liquid sand		
5s	20	Very stiff clay	20	Medium clay	20	Liquid sand		
6a	10	Loose sand	30	Liquid sand	10	Loose sand	10	Very stiff clay
6b	10	Soft clay	30	Liquid sand	10	Soft clay	10	Very stiff clay
6c	10	Loose sand	30	Liquid sand	10	Soft clay	10	Very stiff clay
6d	10	Soft clay	30	Liquid sand	10	Loose sand	10	Very stiff clay
7c	10	Loose sand	40	Liquid sand	10	Loose sand		
7d	10	Soft clay	40	Liquid sand	10	Loose sand		
8a	50	Liquid sand	10	Loose sand				

Note: 1 ft = 0.305 m.

Table 6. Maximum depth to in-ground maximum moment

Cross section	Pile size, in.	Fixed head				Free head			
		Liquefiable		Nonliquefiable		Liquefiable		Nonliquefiable	
		Depth, ft	Profile	Depth, ft	Profile	Depth, ft	Profile	Depth, ft	Profile
Square	12	30.0	2k	20.5	4i, 4l	22.8	2k	12.5	4c
	14	34.5	6d	22.8	1e, 2d	23.8	2k	14.3	4l
	16	40.0	2k	25.3	1e	22.3	2k	15.5	4i
Octagonal	12	30.0	2k	23.5	1e, 2d	23.0	2k	17.0	1e
	14	36.0	6d	26.3	1e, 2d	24.8	2k	18.8	4i
	16	38.3	6d	29.3	1e, 2)	26.0	2k	21.0	4i

Note: 1 in. = 25.4 mm; 1 ft = 0.305 m.

Table 6 values are upper-bound values for fixed-head cases and lower-bound for free-head cases. It would therefore be conservative to use the results to limit the assumed depth to a below-grade hinge location for the fixed-head cases. However, it would be unconservative to do so for a pile assumed to be free or pinned.

Finally, depth to maximum moment varied significantly for liquefiable versus nonliquefiable soil profiles. For nonliquefiable soil profiles, maximum depth–to–peak moment results in Table 6 represent either 60 ft (18 m) deep soft clays, in the case of profile 1e, or profiles approaching the monolithic soft clays, such as profiles 2d, 4i, and 4l. For liquefiable cases, maximum values for all pile types occurred in 50 ft (15 m) liquefiable sand underlaid by a very stiff clay, a 30 ft (9 m) liquefiable sand with 10 ft (3 m) of soft clay, or sand above and below underlaid by a very stiff clay. As such, these results focus on the extreme depth of very weak or incompetent soils, whereas the depth to maximum moment was significantly less in the extreme cases for the majority of soil profiles simulated.

Recommendations

Based on the findings of this parametric study and an assumed hinge length of twice the pile dimension, it is recommended that the length of the pile for which prescriptive transverse reinforcement is required be reduced from 35 to 25 ft (11 to 7.6 m) for 12 and 14 in. (300 and 360 mm) square piles to be installed in nonliquefiable soils.

No change is recommended to the prescriptive depth requirement for prescriptive transverse reinforcement for 16 in. (410 mm) piles or octagonal piles.

The analyzed depth to maximum moment for a significant number of practical soil conditions was shown to be substantially less than 35 ft (11 m). Therefore, this study provides justification for the use of the performance-based design provisions of the PCI recommended practice[6] as a reliable means for reducing the depth required for ductile reinforcement.

In this study, all maximum analyzed depths to maximum in-ground moments were limited to maximum elastic bending stiffness analyzed in six soil profiles for the fixed-head cases and six soil profiles for the free-head cases. To analyze the number of soil and pile parameters of interest, this study considered elastic pile properties only. This is believed to be a limitation of this report, resulting in somewhat conservative results. With a smaller subset of soil profiles, including the 12 referenced in Table 6, consideration of nonlinear structural pile properties is feasible and would likely justify improved recommendations. Recommended improvement to prescriptive requirements for 16 in. (410 mm) and octagonal piles may also result.

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Notation

C = cohesion

E = modulus of elasticity

E_{50} = small strain factor

f_c' = 28-day concrete compressive strength

I = moment of inertia

m = $1 - \sin\theta$

S_u = undrained shear strength

V_s = average shear wave velocity

γ = unit density of soil

γ' = effective unit weight of soil

θ = friction angle

About the authors

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Abstract

Seismic design of prestressed concrete piles requires transverse spiral reinforcement to be proportioned to confine and thus retain an intact concrete core during inelastic rotation cycles. Prescriptive provisions of the *International Building Code* require a prescriptive minimum volumetric ratio of confinement reinforcement relative to the pile cross section and a minimum in-ground depth to which the confinement reinforcement must extend. This study seeks to determine a practical maximum depth to maximum in-ground bending moment for 12, 14, and 16 in. (300, 360, and 410 mm) prestressed concrete piles subjected to displacement consistent with lateral seismic loads. A commercially available lateral analysis software was used to conduct a parametric analytical study of square and octagonal piles embedded in substantially varied soil conditions. Piles were modeled to remain elastic. Depth to maximum in-ground moment was recorded. Maximum depth values for each pile type were reported. Analysis of results indicates that prescriptive depth requirements may be overly conservative for 12 and 14 in. square piles in nonliquefiable soils. It was concluded that a reduction could be made to the code-prescribed depth of ductile reinforcement for some pile types in nonliquefiable conditions.

Keywords

Confinement, design, ductile zone, in-ground moment, pile, plastic hinge, seismic.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process. The Precast/Prestressed Concrete Institute is not responsible for statements made by authors of papers in *PCI Journal*. No payment is offered.

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