Minimum confinement reinforcement for prestressed concrete piles and a rational seismic design framework

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- The existing design requirements of confinement reinforcement in potential plastic hinge regions in prestressed concrete piles in seismic regions vary significantly, often resulting in unconstructible details.
- This paper presents a rational approach for designing minimum confinement reinforcement for prestressed concrete piles in seismic regions.
- This paper also presents a new axial load limit for prestressed piles, an integrated framework for the seismic design of piles and superstructure, the dependency of pile displacement capacity on surrounding soils, and how further reduction to confinement reinforcement could be achieved.

Precast, prestressed concrete piles have been widely used in the design of foundations for bridges, buildings, and wharf structures. This is because they are relatively lightweight and fabricated in a controlled environment to maintain good construction quality. They are also less prone to cracking during driving and have good corrosion resistance due to concrete serving as an effective moisture barrier. A variety of prestressed concrete piles are standardized by the precast concrete industry. The cross sections of these piles may be square, octagonal, or circular and either solid or hollow. Solid square and solid octagonal cross sections with a circular strand pattern and spiral transverse reinforcement are the most commonly used types in design practice in seismic regions (**Fig. 1**). This is because square piles are easier to cast, and octagonal piles minimize the impact of spalling of cover concrete on the moment-curvature response. Given the typical length requirements, it is convenient to cast precast, prestressed concrete piles in a horizontal position rather than in a vertical position. With the piles cast horizontally, the square piles in particular provide ease to the casting process. The most common sizes used in current seismic design practice are 12, 14, and 16 in. (300, 350, 400 mm) square piles and 16 and 24 in. (610 mm) octagonal piles. Typically, pile cross sections tend to be smaller for building structures and larger for bridge and heavy marine structures.

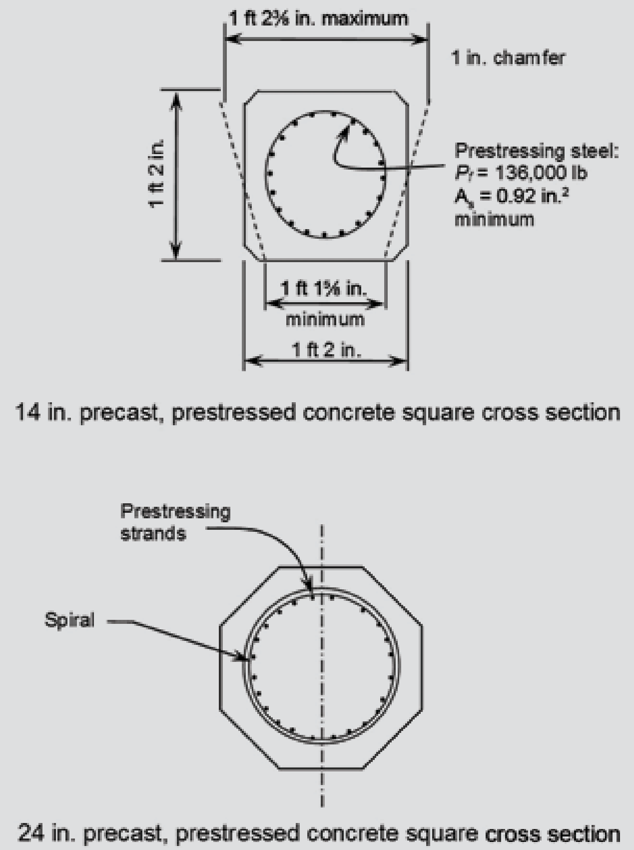

Figure 1. Typical details of the standard piles used for foundations in seismic regions. Note: P_f = prestressing force; A_s = area of prestressing steel. 1 in. =25.4 mm; 1 ft = 0.305 m; 1 lb = 4.448 N.

In the United States, high seismic regions, such as California, Washington, South Carolina, and Alaska, adopt their own design criteria in conjunction with the national codes and standards for the design of foundations. This is to ensure that satisfactory performance of structures can be achieved when they are subjected to earthquake motions. The seismic design philosophy adopted in these regions generally follows the capacity design philosophy, which, according to Paulay and Priestley[1] and Priestley et al.[2] may be summarized as follows:

- Under design-level earthquake loads, structures are designed to respond inelastically through flexural yielding.
- Locations of plastic hinges are selected and detailed carefully to ensure that structures can develop dependable ductile response.
- Using suitable strength margins, undesirable mechanisms of inelastic responses, such as shear failure and inadequate anchorage of reinforcing bars, are prevented from developing.

When implementing the capacity design philosophy, the locations of plastic hinges are typically chosen in the structure at or above the ground surface, preventing the foundation elements, including piles, from experiencing inelastic actions. This approach allows easy inspection of damage associated with formation of plastic hinges and avoids large inelastic rotations potentially developing at fewer locations. An exception is made when bridge columns are extended into the ground as drilled shafts, in which case in-ground plastic hinges are allowed to form in the foundation shafts or piles supporting wharfs. Despite attempts to avoid forming plastic hinges in the foundation elements, preventing inelastic actions in piles that support footings is not always practical because the moment gradient along the pile length is markedly influenced by the properties of the soil surrounding the pile.[2] Furthermore, the interaction gap between geotechnical and structural engineers during the foundation design process and ways that the foundation elements are modeled by the two disciplines (geotechnical models often use elastic piles, while structural models completely ignore the soils and sometimes the piles) can increase the potential for the piles to experience inelastic actions when the structure is subjected to an earthquake load. The extent of inelastic action that the piles may experience during an actual seismic event is not well understood because earthquake reconnaissance efforts typically do not investigate this issue unless pile failure is evident at a site. When plastic actions are developed in piles supporting a building or bridge columns, the seismic response of the structure is altered from that assumed in design. Hence, the validity of the current seismic design practice, which treats the pile foundation design independently of the superstructure design that is typically done with the assumption of a fixed column base, should be questioned.

Given the aforementioned challenges, a research investigation was undertaken to do the following:

- determine an appropriate seismic curvature demand for piles through a literature review of past research and field experiences
- establish a rational equation that will provide the minimum amount of transverse (that is, confinement) reinforcement for prestressed concrete piles while ensuring curvature capacity greater than that established as the potential maximum curvature demand
- embed a curvature ductility factor within the developed equation to help designers obtain the necessary confinement reinforcement more appropriately
- using the developed equation, determine permissible lateral displacements that the prestressed piles will be able to withstand in different soil conditions
- formulate recommendations suitable for the design of confinement reinforcement for precast, prestressed

piles in seismic regions

- propose an integrated framework for designing foundation and superstructure

The results of this research are presented in this paper.

Various confinement requirements

As the first step in this study, the spiral confinement reinforcement requirements of several codes and standards for prestressed concrete piles were examined. They included the PCI recommended practice,[3] Applied Technology Council (ATC-32),[4] Uniform Building Code (UBC),[5] International Building Code (IBC),[6] American Society of Civil Engineers (ASCE) *Minimum Design Loads for Buildings and Other Structures*,[7] American Concrete Institute (ACI) *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05)*,[8] and the New Zealand code of practice for concrete structures (NZS).[9,10] Although the confinement requirements specified in some of these documents are comparable, there exist significant differences among the requirements specified by several codes and standards for a given prestressed pile. Five distinct requirements for the volumetric ratio of spiral confinement reinforcement ρ_s are identified in Eq. (1) to (5).

PCI

$$\rho_s = 0.25\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}}-1\right)\left[0.5+\frac{1.4P}{f_c^{'}A_g}\right] \tag{1}$$

$$\text{but not less than } 0.12\left(\frac{f_c^{'}}{f_{yh}}\right)\left[0.5+\frac{1.4P}{f_c^{'}A_g}\right]$$

where

$f_c^{'}$ = compressive strength of unconfined concrete

f_{yh} = yield strength of transverse reinforcement

A_g = gross section area of the concrete pile section

A_{ch} = cross-sectional area of confined core concrete section, measured out-to-out of the spiral reinforcement as defined by ACI 318-05

P = design axial force (derived from overstrength consideration)

ATC-32

$$\rho_s = 0.16\left(\frac{f_c^{'}}{f_{yh}}\right)\left[0.5+1.25\frac{P}{f_c^{'}A_g}\right]+0.13\left(\rho_l-0.01\right) \tag{2}$$

where

ρ_l = ratio of nonprestressed longitudinal column reinforcement, which is = A_{st}/A_g

UBC

$\rho_s \geq 0.021$ for piles 14 in. (360 mm) and smaller; $\rho_s \geq 0.021$ for piles 24 in. (610 mm) and larger (3)

ACI 318-05

$$\rho_s = 0.45\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{A_g}{A_{ch}}-1\right) \tag{4}$$

$$\text{but not less than } 0.12\left(\frac{f_c^{'}}{f_{yh}}\right)$$

NZS

$$\rho_s = \frac{(1.3-\rho_l m)}{2.4}\left(\frac{A_g}{A_{ch}}\right)\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{P}{\phi f_c^{'}A_g}\right)-0.0084 \tag{5}$$

$$\text{but not less than } \frac{A_{st}}{110D^{'}}\frac{f_y}{f_{yh}}\frac{1}{d_b}$$

where

m = nondimensional ratio = $\dfrac{f_y}{0.85f_c^{'}}$

ϕ = curvature

A_{st} = total area of mild longitudinal steel reinforcement

D' = core concrete diameter measured to the center of the transverse reinforcement

f_y = yield strength of longitudinal reinforcement

d_b = diameter of the reinforcing bar

Figure 2 compares the required volumetric ratio of the transverse reinforcement for two prestressed piles using five different suggested methods as a function of axial load

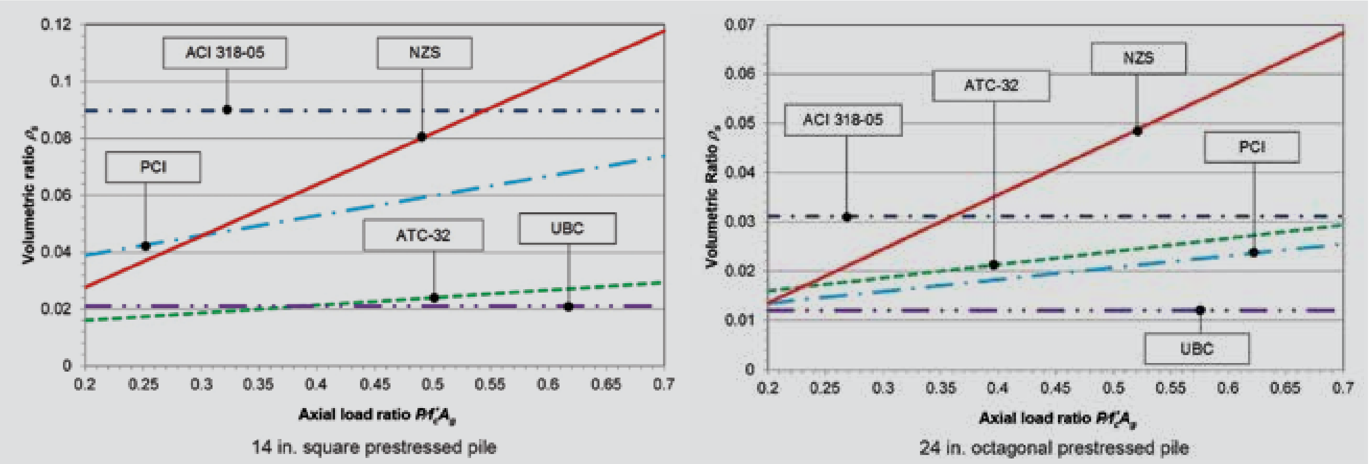

Figure 2. Confinement reinforcement per different design requirements as a function of axial load ratio. Note: A_g = gross section area of the concrete pile section; $f_c^{'}$ = compressive strength of unconfined concrete; P = design axial force (derived from overstrength consideration). 1 in. =25.4 mm.

ratio. In both cases, the compressive strength of unconfined concrete $f_c^{'}$ was 8.0 ksi (55 MPa), the yield strength of transverse reinforcement f_{yh} was 60 ksi (410 MPa), and a 2 in. (50 mm) concrete cover was used. According to Fig. 2:

- The required ρ_s for prestressed piles differs significantly among design codes and standards. At both low and high axial loads, the difference in ρ_s requirements is more than a factor of about three to five depending on the pile.

- Except for ACI 318-05 and UBC, the required ρ_s increases with an increase in the external compressive axial load ratio. The ρ_s value becomes independent of the axial load when confinement is provided such that the pile's full axial load capacity will not be compromised due to spalling of the cover concrete.

- NZS[9,10] requires the largest amount of confinement for high external axial loads, whereas ACI 318-05[8] requires the largest amount of confinement for low external axial loads, which is primarily because this equation is independent of the axial load.

- The ACI 318-05 requirement for both piles at small axial loads translates to no. 3 (10M) spiral reinforcement at a spacing of less than 0.7 in. (18 m), which does not meet the minimum spacing requirement, demanding a larger-diameter spiral. Such requirements are difficult to meet in practice because they cause construction challenges due to congestion of reinforcement.

Critical information

Design parameters

Several parameters influence the required amount of confinement reinforcement in the potential plastic hinge region of prestressed concrete piles. These variables, which were identified in Eq. (1) to (5), are based on the existing design methods and theoretical confinement models.

Although the ultimate curvature of the section (or the section curvature ductility capacity) should be a design variable when quantifying the required confinement reinforcement, this is not generally included in confinement equations because the target curvature value is typically considered an unknown. The same applies to confinement equations available for columns, beams, and walls. While this shortcoming may be considered a disadvantage of prevalent methods used in current design practice, it is acknowledged that determining a target curvature demand for a given problem is not straightforward because this variable depends on earthquake demand and other factors.

In addition to addressing the target curvature demand or the required ductility capacity, two other challenges need to be resolved prior to establishing a confinement equation for prestressed concrete piles. First, a consistent, simple approach to idealize the moment-curvature response of prestressed concrete pile sections does not exist and must be established. Second, there was no rationale found for the axial load limits suggested in codes for prestressed concrete piles. Hence, a more suitable axial load limit should be established for piles subjected to both flexural and axial loads.

Target curvature demand

To establish a possible upper-bound curvature demand for precast concrete piles, a review of published literature on prestressed concrete piles was completed and the following information was gathered:

- measured curvature capacity during large-scale testing of precast, prestressed concrete piles that were assumed to have sufficient confinement reinforcement in the plastic hinge region

- back calculated curvature demands on piles that experienced damage during past earthquakes as well as expected demands on piles subjected to earthquake loading[11]

Figure 3 summarizes the results of this study, which include data from the testing of 12 to 18 in. (300 to 460 mm) square and 14 in. (360 mm) octagonal precast, prestressed concrete piles. The curvature demand reported in Fig. 3 come from cast-in drill-hole shafts as well as prestressed concrete and steel piles, with the maximum value being reported for a prestressed concrete pile subjected to the 2003 Tokachi-oki earthquake by Koyamada et al.[12] The reported curvature capacities in Fig. 3 range from 0.0002 to 0.00107 in.$^{-1}$ (0.008 to 0.0421 m^{-1}) while the curvature demand varies from 0.0002 to 0.00152 in.$^{-1}$ (0.0598 m^{-1}). Furthermore, the maximum reported curvature capacity of 0.00107 in.$^{-1}$ is about 70% of the maximum reported demand of 0.00152 in.$^{-1}$, indicating that piles in some cases have probably been designed with insufficient curvature capacity and are susceptible to earthquake failure. Despite the limited data, it appears that the development of a design equation to quantify the confinement reinforcement for prestressed concrete piles should provide a curvature capacity of at least 0.00152 in.$^{-1}$ for prestressed pile sections expected to form plastic hinges in high seismic regions.

Moment-curvature idealization

To define curvature ductility capacity of a concrete section, first the actual moment-curvature response needs to be idealized, preferably with a bilinear curve, and the curvature ductility can then be defined as a ratio between the ultimate curvature and the idealized yield curvature. The moment-curvature response of prestressed concrete pile sections has unique characteristics and is difficult to idealize due to these sections' use of high-strength prestressing strands, the significantly large thickness of cover concrete, and a lack of mild steel reinforcement. Therefore, the idealization typically used for the moment-curvature response of reinforced concrete sections[2] in which the inelastic action is initiated by yielding of the mild steel reinforcement was found to be inappropriate.[11] In the absence of an easily applicable bilinear idealization approach in the literature, several different moment-curvature idealizations were examined. Following are the methods chosen to define the first yield condition as well as the nominal and ultimate moment resistance of a prestressed concrete pile section. As with the reinforced concrete sections, the elastic curvature corresponding to the nominal moment, which is found from the moment and curvature defined at the first yield limit state, defines the idealized yield curvature.

First yield condition In typical prestressed concrete pile sections with no mild steel reinforcement, nonlinear response begins when concrete enters the nonlinear stress-strain region. Consequently, the first yield moment for prestressed concrete pile sections is defined using a concrete strain of 0.002 in./in. (0.002 mm/mm), at which point the stress-strain behavior of concrete is assumed to begin responding in a nonlinear manner. The first yield curvature $\phi_y^{'}$ is thus equal to the curvature corresponding to a concrete strain of 0.002 in./in. in the extreme compression fiber; the corresponding flexural resistance of the pile section defines the first yield moment $M_y^{'}$.

Nominal (or yield) moment In consideration of the unique moment-curvature response of prestressed concrete pile sections, it was found that defining the nominal moment capacity M_n (equal to yield moment M_y) using a concrete strain of 0.004 in./in. (0.004 mm/mm) or a strain value in the extreme prestressing strand was not satisfactory through comparisons of idealized and actual moment-curvature responses. Consequently, the nominal moment capacity is defined as the average of the minimum moment and the maximum moment that occurs between the first yield moment and the ultimate moment. Through analysis of numerous prestressed pile sections, this approach was found to be not only simple but also fairly consistent in providing satisfactory idealized responses. The minimum moment typically occurs when the cover concrete of the pile section is completely crushed, whereas the maximum moment may be equal to the ultimate moment capacity of the pile section. Hence, the idealized yield curvature ϕ_y is obtained from Eq. (6).

$$\phi_y = \frac{M_n}{M_y^{'}} \phi_y^{'} \qquad (6)$$

Ultimate moment Using the information found in the literature, the ultimate moment of prestressed concrete piles is defined by one of the following three conditions, whichever occurs first:

- 80% of the peak moment resistance of the section
- the moment corresponding to the first occurrence of a strain of 0.04 in./in. (0.04 mm/mm) in a prestressing strand
- the moment associated with a strain in the extreme compression fiber of the core concrete equal to the ultimate strain capacity of the confined concrete ε_{cu}

The first condition has traditionally been used in seismic practice to minimize drastic increases in displacement of a laterally loaded flexural member due to reduction in its moment capacity. However, in typical prestressed pile sections, the ultimate moment is expected to be controlled by the third condition, in which the strain is limited according to the recommendation of Mander et al. (Eq. [7]):[13]

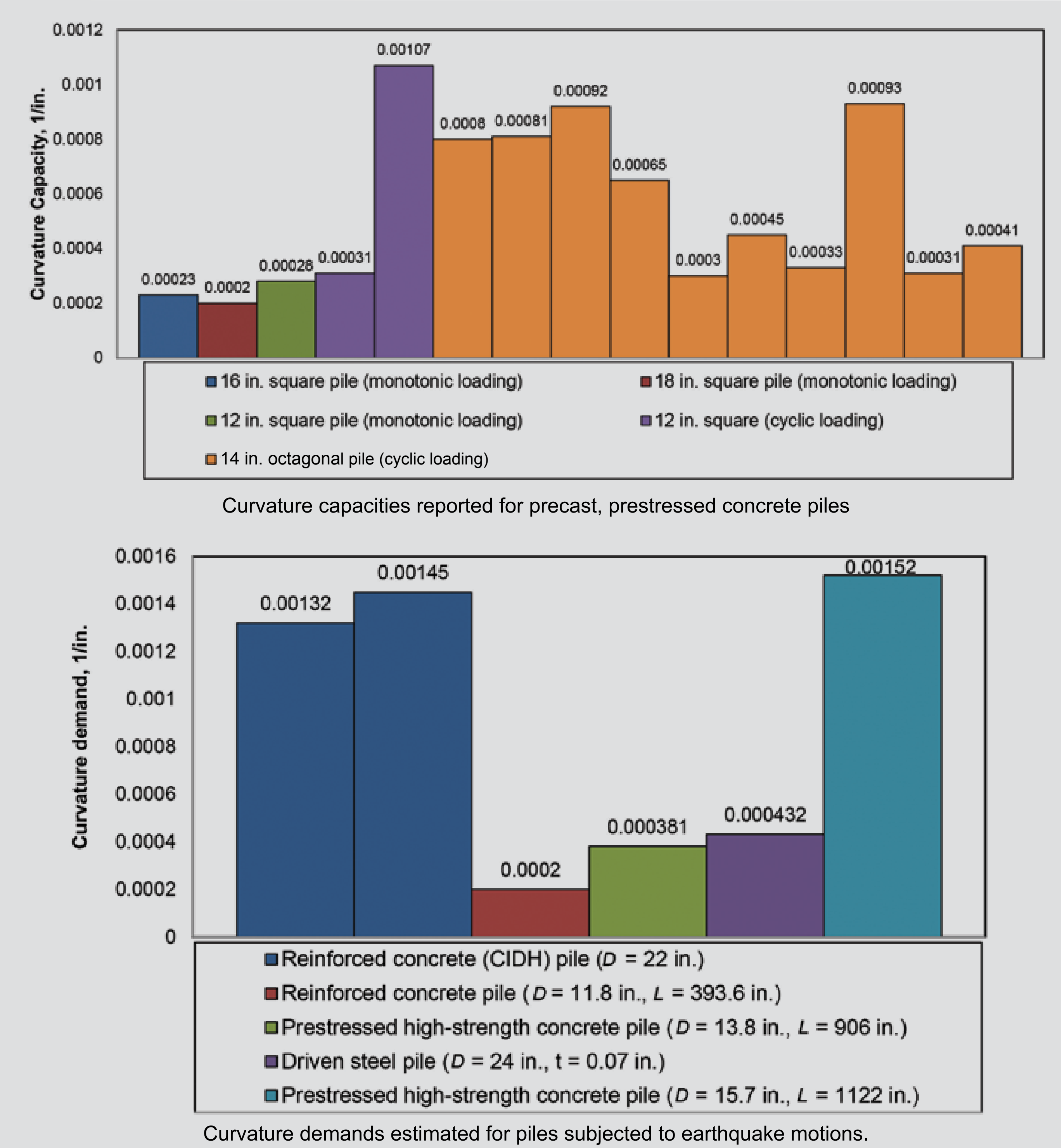

Figure 3. Summary of curvature reported for seismic piles in the literature. Note: *D* = diamenter; *L* = length. 1 in. = 25.4 mm.

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f'_{cc}} \qquad (7)$$

where

ε_{su} = strain corresponding to the ultimate strength of confinement reinforcement = 0.12 for Grade 60 (414 MPa) steel

f'_{cc} = compressive strength of confined concrete

Figure 4 illustrates an idealized and theoretically established moment-curvature response of a prestressed concrete pile section, which shows a satisfactory correlation between the two responses.

Limit on axial load ratio

A commonly used limit on axial load ratio for prestressed concrete piles is given in Eq. (3) as obtained from the *PCI Design Handbook: Precast/Prestressed Concrete*.[14]

$$N = \left(0.33 f_c^{'} - 0.27 f_{pc}\right) A_g \qquad (8)$$

where

N = allowable external axial load

f_{pc} = compressive stress in the concrete at the centroid of the pile section due to prestress after losses

Through rearrangements of the variables in Eq. (8), the axial load ratio limit can be expressed as Eq. (9).

$$\frac{N}{f_c^{'} A_g} = \left(0.33 - 0.27 \frac{f_{pc}}{f_c^{'}}\right) \qquad (9)$$

Assuming $f_c^{'}$ equals 10,000 psi (69 MPa) to estimate an upper-bound value for an axial load ratio and taking f_{pc} to be in the range between 700 psi (4.8 MPa) and $0.2 f_c^{'}$,[14] the resulting limit on the external axial load ratio for prestressed piles is between 0.28 and 0.31. However, the authors considered this limitation on the axial load ratio to be irrelevant because no rationale for deriving Eq. (8) could be found. The new limit is intended for piles subjected to both axial and flexural actions and is defined using two key curvature values: the curvature that initiates crushing and spalling of unconfined cover concrete ϕ_{sp} and the curvature corresponding to the flexural cracking moment ϕ_{cr}. The moment at which crushing of the unconfined concrete begins is defined using a concrete strain of 0.004 in./in. (0.004 mm/mm). With this definition, the axial load in prestressed concrete piles is limited such that ϕ_{cr} should not exceed ϕ_{sp}. The reason for imposing this condition is that the magnitude of moment drop due to spalling of cover concrete is significant when $\phi_{cr} > \phi_{sp}$ (**Fig. 5**).

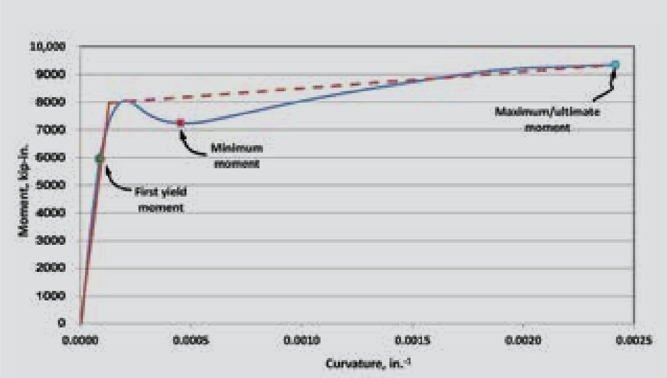

Figure 4. Moment-curvature idealization of a 24 in. octagonal prestressed concrete pile section. Note: 1 in. = 25.4 mm; 1 kip = 4.448 kN.

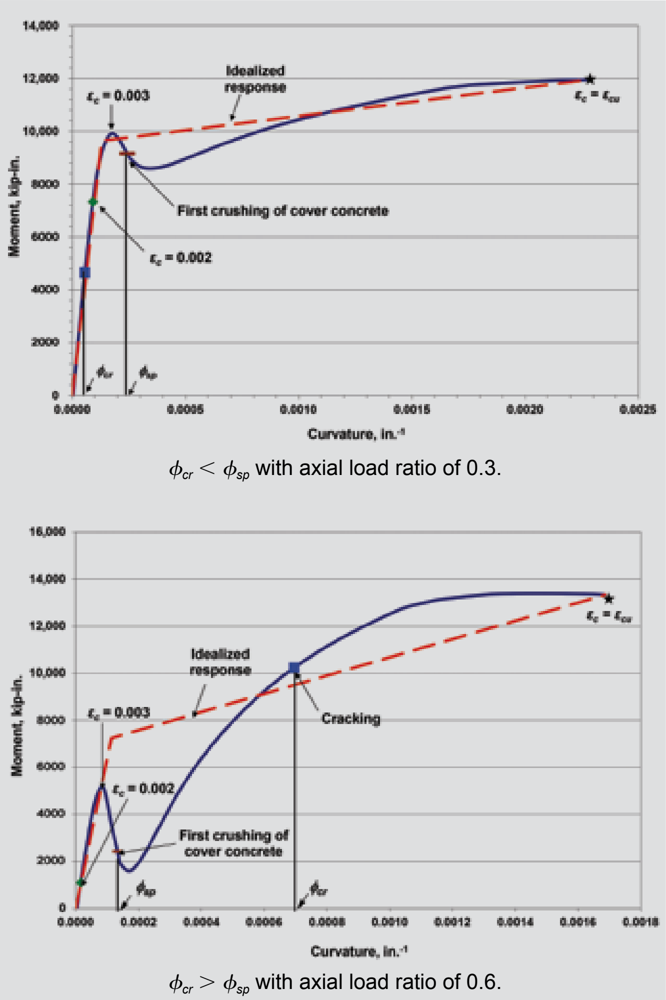

Figure 5. Examples of moment-curvature responses showing the impact of high axial load ratio using a 24 in. octagonal prestressed pile section. Note: ε_c = concrete compressive strain; ε_{cu} = ultimate strain capacity of the confined concrete; ϕ_{cr} = curvature corresponding to the flexural cracking moment; ϕ_{sp} = curvature that initiates crushing and spalling of unconfined cover concrete. 1 in. = 25.4 mm; 1 kip = 4.448 kN.

Additional examples may be found in Fanous et al.[11] In such cases, the difference between the idealized moment and the actual resistance at curvatures close to ϕ_{cr} was found to be as large as 80%, and defining the ultimate moment of the pile section and the corresponding curvature was challenging. Also, the stability of the pile experiencing significant moment drop may not be dependable, and thus the curvature capacity of these piles should be limited to a value less than ϕ_{cr}.

Development of a new equation

Realizing the limitations of code requirements, Budek-Schmeisser and Benzoni suggested a seismic design procedure for precast concrete piles that quantifies the transverse reinforcement.[15] This procedure has not been widely adopted or verified, which is believed to be due to its complexity and the requirement that an equivalent column model, which itself introduces additional approximations, for the piles must be developed. Therefore, it is important to maintain the simplicity of the requirement as used in the current codes and standards. In consideration of the existing equations for ρ_s, the ATC-32 equation provides one of the most efficient amounts of confinement reinforcement. Other advantages of this equation are that the currently adopted equations for plastic bridge columns follow a similar format and that this equation targets a curvature ductility of 13 with an anticipation of 50% more reserve capacity beyond the target value. Hence a suitable equation was developed using ATC-32 as the basis but recognizing the following features:

- $0.13(\rho_l - 0.01)$ is not significant because ρ_l in prestressed piles is small and often leads to a negative value for this term. Therefore, this term was ignored.

- It was assumed that the new equation should ensure that a minimum reinforcement limit suggested in ACI 318-05 should be met, which is

  $$\rho_s \geq 0.12\left(\frac{f_c^{'}}{f_{yh}}\right)$$

  This limitation was suggested to ensure adequate flexural curvature capacity of concrete sections subjected to bending and axial load.[8]

- Using a preliminary version of Eq. (10) and a set of prestressed piles, the necessity of including the $f_{pc}A_g$ term in the axial load parameter P was examined. Including the initial prestressing had some influence on the curvature ductility capacity of prestressed pile sections, particularly at large axial load ratios. This was due to the influence of f_{pc} on the yield curvature ϕ_y rather than on the ultimate curvature ϕ_u. An attempt to include f_{pc} in the confinement equation led to unnecessarily conservative amounts of confinement reinforcement for piles with lower axial load ratios. Therefore, the f_{pc} term was eliminated in the axial load term of the confinement equation.

- The dependence of the preliminary confinement equation on the axial load ratio was found to be too large. With this and the ACI 318-05 minimum reinforcement requirements in mind, the constant terms of the equation, specifically 0.16 and 0.5 (Eq. [2]), were investigated to lessen the dependence of the equation on the axial load ratio. Through small sets of analyses of pile sections, it was determined that these two constants needed to be replaced by 0.06 and 2.8, respectively.

- As in ATC-32, the A_g term is typically used in confinement equations, though the transverse reinforcement is intended to confine the core area and not the gross concrete area. Therefore, the definition of the axial load ratio in the confinement equation should be based on A_{ch}, whereas the axial load ratio in general is defined using A_g. Unlike the reinforced concrete sections, the ratio between A_g and A_{ch} is significant (greater than 1.5) for prestressed concrete piles due to the large thickness of cover concrete, and the ratio also changes noticeably among standard precast, prestressed concrete piles. To overcome this challenge, the axial load ratio in the confinement equation was replaced with A_{ch}, and then this ratio was normalized with respect to the A_g/A_{ch} ratio of a prestressed pile section. For this purpose, the 16 in. (400 mm) octagonal pile was chosen, which led to a multiplier of 1.87 (A_{ch}/A_g) for the axial load ratio term.

With these changes, Eq. (10) for confinement was established.

$$\rho_s = 0.06\left(\frac{f_c^{'}}{f_{yh}}\right)\left[2.8 + \left(\frac{1.25P}{f_c^{'}A_{ch}}\right)\left(\frac{1.87A_{ch}}{A_g}\right)\right] \tag{10}$$

As detailed in the next section, using the results from 16 and 24 in. (400 and 610 mm) octagonal pile sections that are more commonly used, a curvature ductility term was integrated into Eq. (10), providing Eq. (11) for quantifying the confinement reinforcement for the plastic hinge region of prestressed concrete piles in high seismic regions.

$$\rho_s = 0.06\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{\mu_\phi}{18}\right)\left[2.8 + \left(\frac{1.25P}{f_c^{'}A_{ch}}\right)\left(\frac{1.87A_{ch}}{A_g}\right)\right] \tag{11}$$

where

μ_ϕ = target curvature ductility of the pile section

Simplification of Eq. (11) produces Eq. (12). Although this version of the equation appears to show that ρ_s is a function of A_g, it must be emphasized that Eq. (12) determines the confinement reinforcement using A_{ch} as the primary variable.

$$\rho_s = 0.06\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{\mu_\phi}{18}\right)\left[2.8 + \left(\frac{2.34P}{f_c^{'}A_g}\right)\right] \tag{12}$$

When a suitable value is not available, μ_ϕ should be taken as 18 for designing prestressed concrete piles in high seismic regions with adequate curvature capacity, which is subsequently justified. It is also suggested that a lower μ_ϕ value may be used for piles in low and moderate seismic regions. Similarly, a value greater than 18 may be used to increase the curvature ductility capacity of a pile section if a greater ductility capacity is required.

With the assumption that μ_ϕ is 18, $f_c^{'}$ is 8000 psi (55 MPa), f_{yh} is 60 ksi (410 MPa), and cover concrete is 2 in. (50 mm), **Fig. 6** compares the volumetric ratio of two pile sec-

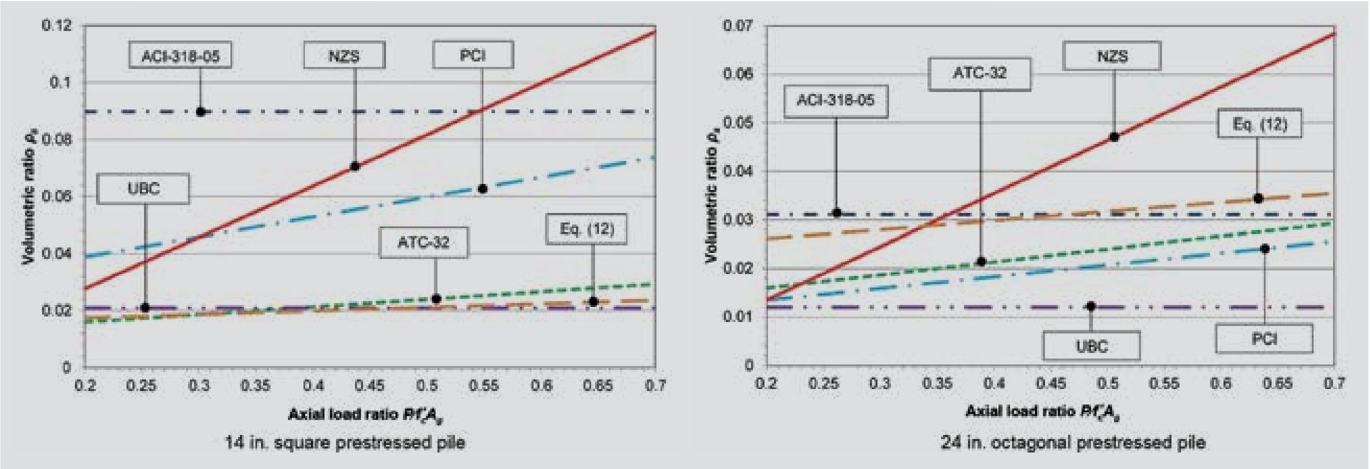

Figure 6. Comparison of required volumetric ratios of spiral reinforcement. Note: A_g = gross section area of the concrete pile section; f'_c = compressive strength of unconfined concrete; P = design axial force (derived from overstrength consideration). 1 in. =25.4 mm.

tions obtained from Eq. (12) with those endorsed by existing recommendations. The trend of the proposed equation is somewhat different from that displayed by other code equations. For the 14 in. (360 mm) square pile, Eq. (11) requires considerably lower transverse reinforcement than that required by the ACI 318-05 equation. In contrast, the proposed equation compares well with the ACI 318-05 recommended confinement requirement for the 24 in. (610 mm) octagonal pile. Compared with the current PCI and ATC-32 requirements, the proposed equation requires more reinforcement for the 24 in. pile but not as much as required by NZS for high axial load ratios. The reduced amount of confinement required for the 14 in. square pile by the proposed equation is encouraging. This is because the small pile size increases reinforcement congestion by significantly reducing the spacing of the transverse reinforcement. For large pile sizes (such as a 24 in. octagonal pile), steel congestion is not a significant issue because of the increase in the diameter of the confinement reinforcement and the subsequent increase in the spacing of the transverse reinforcement.

Verification

The validity of the confinement equation presented in Eq. (12) was investigated by varying the concrete strength (from 6 to 10 ksi [41 to 69 MPa]), axial load ratio (from 0.2 to 0.5 or the maximum recommended limit, whichever occurred first), initial prestress (from 700 to 1200 psi [4800 to 8300 kPa]), pile size, and pile shape. In all cases, the curvature ductility capacity of the pile section was quantified by running a moment-curvature analysis and idealizing the calculated response as defined in Fig. 4. The axial load ratio was varied from 0.2 to the maximum limit as defined according to the recommended approach presented previously. **Figure 7** shows the results of 152 different octagonal prestressed pile sections, which had an average ductility of 19.4 and standard deviation of ±1.1. Although most of the data points fall above the mean–minus–standard deviation line (μ_ϕ equal to 18.3), some of the analyses produced a smaller ductility capacity, less than 18.3. The smallest ductility capacity achieved was 17.2, which is only 4.4% less than the target ductility of 18. Given the different variables used in this particular verification, the proposed equation is considered simple and sufficiently accurate for quantifying confinement reinforcement of octagonal prestressed concrete pile sections. Of the various analyses completed, the section ultimate curvature capacity varied from 0.00194 to 0.00364 in.$^{-1}$ (0.0764 to 0.143 m^{-1}).

For comparison purposes, the analyses of the 16 and 24 in. (410 and 610 mm) octagonal sections were repeated with confinement reinforcement as suggested by ATC-32 and NZS because these two equations use target curvature ductilities of 13 and 20, respectively. ATC-32 and NZS produced average section ductility of 14.2 and 19.2, respectively. The corresponding error between the target and average curvature ductility was 14.6 and -4.0%. Equation (12) led to an error of 7.8%. Most importantly, the standard deviations obtained for the three data sets were 15, 4.77, and 1.02, respectively, implying that Eq. (12) leads

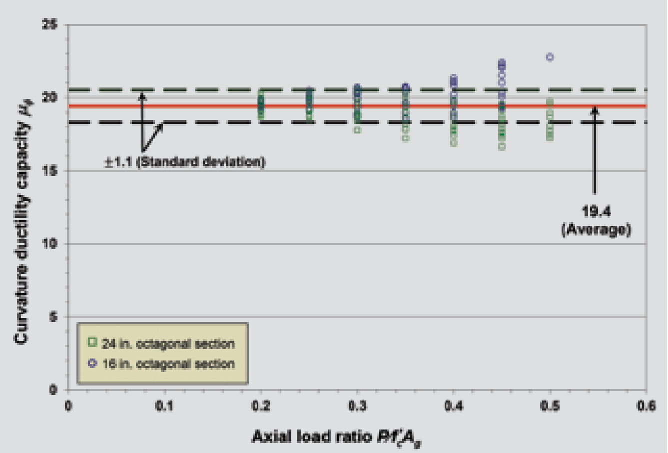

Figure 7. Curvature ductility capacities of 16 and 24 in. prestressed pile sections with confinement reinforcement based on Eq. (12). Note: A_g = gross section area of the concrete pile section; f'_c = compressive strength of unconfined concrete; P = design axial force (derived from overstrength consideration). 1 in. =25.4 mm.

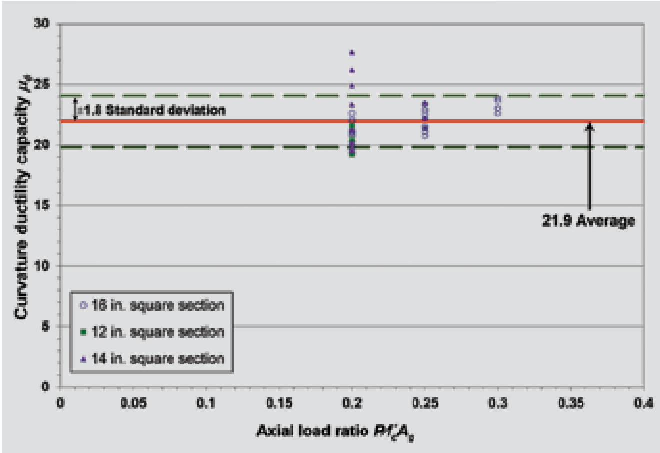

Figure 8. Curvature ductility capacities of different square pile sections with confinement based on Eq. (12). Note: A_g = gross section area of the concrete pile section; f'_c = compressive strength of unconfined concrete; P = design axial force (derived from overstrength consideration). 1 in. =25.4 mm.

to reduced scatter and reduced overestimation between the actual and target curvature ductility capacity compared with the ATC-32 and NZS approaches.

Figure 8 presents the results of square pile sections with an axial load ratio up to 0.3, above which the response of the pile was found to be unstable with significant reduction in moment capacity as a result of ϕ_{cr} being greater than ϕ_{sp}. Within the established axial load limits, all pile sections produced curvature ductility greater than 18, with a lower bound of the ultimate curvature capacity being 19.2. The average curvature ductility of this square pile group was 21.9, with a standard deviation of ±1.8. No further refinement to the equation was considered necessary because the required amounts of confinement per Eq. (12) are generally less than those of the existing requirements.

The confinement reinforcement requirement of Eq. (12) was also examined for several other octagonal and square prestressed concrete pile sections with target curvature ductilities of 12 and 6; the reduced curvature demands were considered satisfactory for foundation piles in moderate and low seismic regions. **Figure 9** presents the results of these analyses and shows that the ductility capacity of pile sections with confinement per Eq. (12) was greater than the target ductility in all cases. The 16 in. (410 mm) pile section with high axial load ratios consistently produced a greater ductility capacity than the target value. Although further refinement may be possible, this was not investigated because the reduction to the confinement reinforcement due to the use of small target ductilities resulted in significant reduction to ρ_s compared with the current requirements.

Finally, the minimum value of the curvature capacity calculated for all of the analyses conducted as part of the study with the target ductility of 18 was 0.00194 in.$^{-1}$ (0.0764 m^{-1}). This value is about 28% greater than the curvature of 0.00152 in.$^{-1}$ (0.0598 m^{-1}) that was established as a possible maximum curvature demand for piles in high seismic regions in Fig. 3, adding more assurance to the proposed confinement requirement in Eq. (12).

Integration of pile design in seismic design

In current seismic design practice, there is a significant disconnect between pile foundation design and how the design of the aboveground structure is accomplished. This disjoint arises from not integrating the expected lateral displacement of pile-supported footings into the design of the structure, though the piles are designed to sustain inelastic flexural actions under design-level and greater earthquake intensity. Despite providing adequate ductility capacity for the piles, the routine design approach assumes that the piles would remain elastic, and thus their lateral displacements are ignored in the design of the aboveground structure, which is targeted to achieve a specific system ductility capacity under design-level earthquake loads. When foundation piles experience elastic or elastic-plus-inelastic

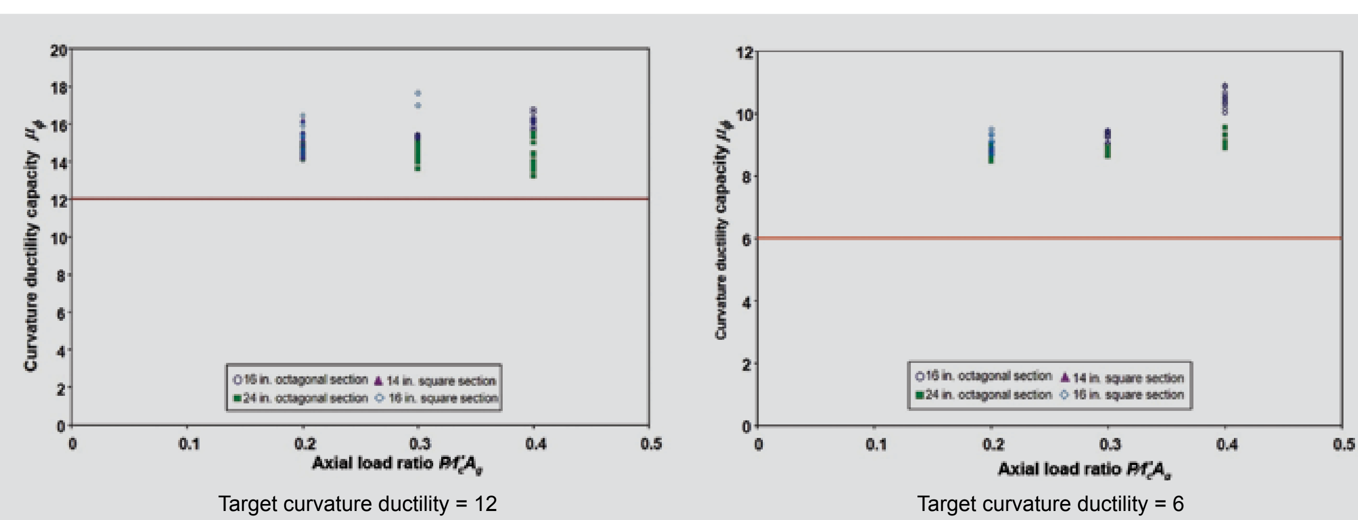

Figure 9. Curvature ductility capacities of octagonal piles designed with small target ductility. Note: A_g = gross section area of the concrete pile section; f'_c = compressive strength of unconfined concrete; P = design axial force (derived from overstrength consideration). 1 in. =25.4 mm.

displacements, the inelastic demand on the aboveground structure may be reduced, causing the superstructure to sustain a reduced level of ductile inelastic response and decrease in hysteretic energy dissipation. Therefore, in order for both the pile foundation and the aboveground structure to achieve a dependable and expected seismic response in accordance with the design assumptions, it is important to integrate the expected pile foundation response into the seismic design of the aboveground structure. In this regard, the following points are emphasized:

- The lateral displacement of piles, including their potential to experience inelastic actions, heavily depends on the properties of the soil surrounding the pile and the interaction between the pile and soil.
- In medium and soft soils, the confinement reinforcement suggested in Eq. (12) will enable piles to undergo several inches of lateral displacement, which will lead to unusable structures after being subjected to sufficiently intense earthquake input motion. In order for structures to be functional after experiencing an earthquake, the pile lateral displacements should be limited. Due to a lack of better information, 2 in. (50 mm) is suggested for this limitation based on discussion with practicing seismic design engineers. When lateral displacement is limited to 2 in. in medium and soft soils, the required confinement reinforcement for piles can be quantified using Eq. (12) with a reduced value for μ_ϕ.
- In stiff or dense soils, lateral displacement capacity of piles with high axial load ratio may be less than 2 in. (50 mm) despite providing confinement as per Eq. (12), which should be recognized in the design of the aboveground structure.

In consideration of the aforementioned points, an overall seismic design process that integrates the expected foundation displacement (**Fig. 10**) involves the following steps:

1. Define pile properties: length, section dimensions, reinforcement details, section area, moment of inertia, modulus of elasticity, moment-curvature relationship that includes the effect of confinement reinforcement, and external loading.
2. Define the soil profile and appropriate properties, taking into account the variability of the average undrained shear strength, the strain at 50% of the ultimate shear stress of the soil, and the initial modulus of subgrade reaction.
3. Define the pile head conditions.
4. Define target and permissible displacements. The target displacement refers to the desired maximum pile displacement assumed by the designer (which may be the same as the permissible or a lesser value). For example, this displacement may be limited to 2 in. (50 mm) or a similar value to ensure functional structures in the immediate postearthquake period. The permissible displacement refers to the lateral displacement limit that the pile can sustain without failure. The permissible displacement should be defined with due consideration to the ultimate displacement of the pile, pile head boundary condition, and influence of the soil surrounding the pile.
5. If the target and permissible displacements are the same, provide the critical pile region with confinement as per Eq. (12) with μ_ϕ equal to 18. If the target and permissible displacements are different, provide the critical pile region with confinement as per Eq. (12) with an appropriate μ_ϕ value.
6. Define the ductility of the structural system, including the effect of the target displacement of the pile-supported footing.
7. Complete the design of the aboveground structure, ensuring that the foundation displacement will never exceed the target displacement.

Permissible displacement limits

To demonstrate the potential variations in pile lateral displacement capacities resulting from the properties of the soil surrounding the pile, a series of lateral load analyses were conducted to examine the permissible displacement limits of piles in high seismic zones. The plastic hinge regions of these piles were assumed to have confinement reinforcement in accordance with Eq. (12) with μ_ϕ equal to 18.

To obtain the permissible limits, lateral load analyses of piles in different soil conditions were conducted using pile modeling software. This software models a pile subjected to lateral loading by treating it as a beam on an elastic foundation with soil resistance represented by nonlinear springs with prescribed load-deflection curves, which are defined by the soil type and the corresponding key properties. The behavior of piles in the software was accurately represented by defining the nonlinear moment-curvature response of pile sections, including the effects of confinement reinforcement, at appropriate places along the pile length. The piles were embedded sufficiently into the soil such that rotation of the pile at the bottom end would not be possible.

For the purpose of demonstration, the software analyses were conducted on seven selected 16 in. (410 mm) octagonal piles. Based on the previously completed verification analyses, these piles were selected to represent the maximum and minimum curvature capacities of

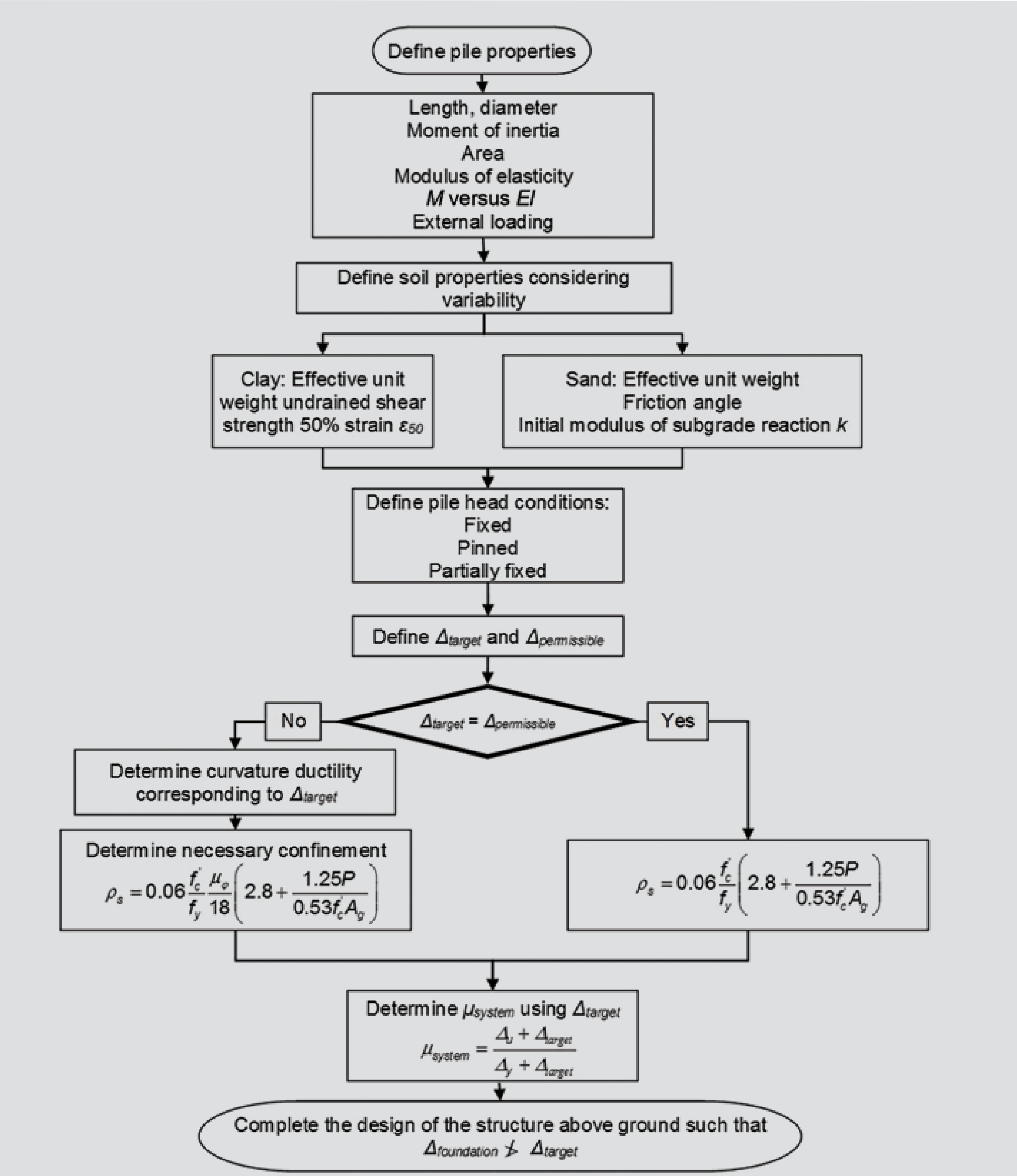

Figure 10. Proposed design process that integrates the expected lateral displacement of pile foundation into the seismic design of the structure. Note: A_g = gross section area of the concrete pile section; f'_c = compressive strength of unconfined concrete; f_y = yield strength of transverse reinforcement; P = design axial force (derived from overstrength consideration); $\Delta_{foundation}$ = foundation deflection; $\Delta_{permissible}$ = permissible deflection; Δ_{target} = target deflection; Δ_u = ultimate deflection; Δ_y = yield deflection; μ_{system} = system displacement ductility; μ_ϕ = target curvature ductility of the pile section; ρ_s = volumetric ratio of spiral confinement reinforcement. 1 in. =25.4 mm.

confined prestressed sections when designed with Eq. (12) as well as to account for variations in f_{pc}, f'_c, and axial load ratio.[11] Following selection of these piles, appropriate soil profiles and boundary conditions were defined. Using the full range of the soil conditions defined in ASCE 7-05[7] as a guide, nine different soil types and the corresponding

Table 1. Parameters selected for the soil models used in software analysis for the ASCE 7 soil classes

Site class (ASCE 7-05)	Site description (ASCE 7-05)			Soil type established for prestressed concrete pile study	Soil parameters established for prestressed concrete pile study				
	v_s, ft/sec	$\overline{N}$	s_u, lb/ft^2						
A. Hard rock	> 5000	n/a	n/a	n/a	n/a				
B. Rock	2500 to 5000	n/a	n/a	n/a	n/a				
Sand					N_{SPT}	ϕ, degree	k (saturated), lb/in.^{3}	k (dry), lb/in.^{3}	γ_{dry}, lb/ft^3
C. Very dense soil and soft rock	1200 to 2500	> 50	> 2000	Very dense sand (API sand)	> 50	41 to 42	145 to 160	240 to 270	110 to 120
D. Stiff soil	600 to 1200	15 to 50	1000 to 2000	Dense sand (API sand)	30 to 50	36 to 40	95 to 135	160 to 230	100 to 110
				Medium sand (API sand)	15 to 30	31 to 35	40 to 80	60 to 135	90 to 100
E. Soft clay soil	< 600	< 15	< 1000	Loose to medium sand (API sand)	< 15	28 to 30	10 to 30	10 to 45	80 to 90
Clay					s_u, lb/ft^2		ε_{50}	k, lb/in.^{3}	γ_{dry}, lb/ft^3
C. Very dense soil and soft rock	1200 to 2500	> 50	> 2000	Hard clay (Matlock)	4000 to 8000		0	n/a	108
				Very stiff clay (Matlock)	2000 to 4000		0	n/a	108
D. Stiff soil	600 to 1200	15 to 50	1000 to 2000	Stiff clay (Matlock)	1000 to 2000		0.01	n/a	108
E. Soft clay soil	< 600	< 15	< 1000	Medium clay (Matlock)	500 to 1000		0.01	n/a	73 to 93
				Soft clay (Matlock)	250 to 500		0.02	n/a	73 to 93
F. Soil requiring site analysis	n/a								

Note: k = initial modulus of subgrade reaction (either saturated or dry); N_{SPT} = field standard penetration resistance for top 100 ft; $\overline{N}$ = average field standard penetration resistance for top 100 ft; s_u = average undrained shear strength; v_s = average shear wave velocity; γ_{dry} = effective unit weight; ε_{50} = strain at 50% strength; ϕ = internal friction angle. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 lb = 4.448 N.

parameter values were established. **Table 1** summarizes the blow count, internal friction angle ϕ, initial modulus of subgrade reaction (either saturated or dry) k, and effective unit weight γ_{dry} assumed for the sand models and average undrained shear strength s_u, strain at 50% strength ε_{50} and effective unit weight γ_{dry} taken for clay in the pile software analyses. ASCE 7-05 soil conditions and the corresponding parameter values are also included in Table 1 for the purpose of classification and comparison. In addition, the pile analyses used three different boundary conditions at the pile head: fixed head, pinned head, and partially fixed head.

Tables 2 and **3** provide the permissible displacement limits that were established for each of the piles analyzed with a fixed pile head and a pinned pile head in sand and clay, respectively. The upper-bound values of the permissible displacement limits in the tables were obtained from the pinned-head analyses, while the lower-bound values were determined by the fixed-head analyses. Further analyses were conducted on these piles with a partially fixed-head condition.[11]

The findings from these analyses can be summarized as follows:

- When embedded in the same soil profile, a pile with a pinned head experienced a larger lateral displacement at the pile head than that with a fixed head, while a partially fixed-head condition generally produced a

Table 2. Permissible displacement limits established for 16 in. octagonal prestressed piles with a fixed pile head and a pinned pile head in different sand soil types

Site class (ASCE 7-05)	$\overline{N}$ ASCE 7-05 site description	Soil type interpreted for prestressed concrete pile study	$\overline{N}$ interpreted for prestressed concrete pile study	Permissible displacement limits (16 in. octagonal pile)						
				Pile 1 $f_c' = 6000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 2 $f_c' = 8000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 3 $f_c' = 10{,}000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 4 $f_c' = 6000$ psi $f_{pc} = 1200$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 5 $f_c' = 8000$ psi $f_{pc} = 1600$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 6 $f_c' = 10{,}000$ psi $f_{pc} = 1600$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 7 $f_c' = 6000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.5$
C. Very dense soil and soft rock	> 50	Very dense sand (API sand)	> 50	2.10 to 2.40	2.00 to 2.30	2.10 to 2.10	1.60 to 4.60	2.10 to 5.05	1.90 to 4.60	1.25 to 1.95
D. Stiff soil	15 to 50	Dense sand (API sand)	30 to 50	2.35 to 2.55	2.20 to 2.65	2.65 to 2.75	1.85 to 5.10	2.60 to 6.25	2.40 to 4.90	1.40 to 2.00
		Medium sand (API sand)	15 to 30	2.75 to 2.90	2.90 to 3.00	3.10 to 3.20	2.35 to 6.60	3.00 to 6.90	2.90 to 6.40*	1.65 to 2.30
E. Soft clay soil	< 15	Loose to medium sand (API sand)	< 15	3.30 to 3.60	3.40 to 3.65	3.65 to 4.10	3.30 to 7.00	3.85 to 7.20*	4.00 to 6.60*	2.10 to 2.55

Note: A_g = gross section area of the concrete pile section; f_c' = compressive strength of unconfined concrete; f_{pc} = compressive stress in the concrete at the centroid of the gross section due to prestress (after losses); $\overline{N}$ = average field standard penetration resistance for top 100 ft; P = design axial force (derived from overstrength consideration). 1 in. = 25.4 mm; 1 psi = 6.895 kPa.

*Did not reach ultimate condition due to significantly high P-Δ effects.

lateral displacement capacity between the displacement bounds established for the pinned-head and fixed-head conditions.

- The lateral displacement limits of piles embedded in clay with both fixed-head and pinned-head conditions decreased as the undrained shear strength and the effective unit weight increased.

- The lateral displacement limits of piles embedded in sand with fixed-head and pinned-head conditions decreased as the friction angle, the initial modulus of subgrade reaction, and the effective unit weight increased.

- At large lateral displacements, the displacement component induced by the axial load (that is commonly referenced as the P-Δ effect) was larger than that caused by the lateral load acting on the pile, which was analyzed in several different soil conditions with a pinned pile head. (These values are identified by an asterisk in Tables 3 and 4). Consequently, the ultimate condition could not be reached for these cases, and thus the reported results do not always appear to follow some of the aforementioned trends. However, the displacements calculated for these piles far exceed the displacements that may be permitted for these piles to experience under seismic lateral load without causing instability to the entire structure.

- Even with μ_ϕ equal to 18 in the recommended confinement Eq. (12), the permissible limits for piles could be limited to values less than 2 in. (50 mm). Based on the completed analyses, the following was found:

  — For a fixed pile head and pinned pile head embedded in sand, the minimum permissible displacement capacities are 1.25 and 1.95 in. (31.8 and 49.5 mm), respectively.

  — For a fixed pile head and pinned pile head embedded in clay, the minimum permissible displacement capacities are 0.95 and 1.25 in. (24.1 and 31.8 mm), respectively.

Site class (ASCE 7-05)	s_u ASCE 7-05 site description	Soil type interpreted for prestressed concrete pile study	s_u interpreted for prestressed concrete pile study	Permissible displacement limits (16 in. octagonal pile)						
				Pile 1 $f_c' = 6000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 2 $f_c' = 8000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 3 $f_c' = 10{,}000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.2$	Pile 4 $f_c' = 6000$ psi $f_{pc} = 1200$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 5 $f_c' = 8000$ psi $f_{pc} = 1600$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 6 $f_c' = 10{,}000$ psi $f_{pc} = 1600$ psi $\frac{P}{f_c' A_g} = 0.45$	Pile 7 $f_c' = 6000$ psi $f_{pc} = 700$ psi $\frac{P}{f_c' A_g} = 0.5$
C. Very dense soil and soft rock	> 2000	Hard clay (Matlock)	4000 to 8000	1.30 to 1.40	1.35 to 1.85	1.45 to 2.00	1.05 to 3.00	1.10 to 3.40	1.10 to 3.20	0.95 to 1.25
		Very stiff clay (Matlock)	2000 to 4000	1.65 to 2.50	1.90 to 2.20	2.00 to 2.35	1.40 to 4.05	1.65 to 4.45	1.55 to 4.15	1.15 to 1.60
D. Stiff soil	1000 to 2000	Stiff clay (Matlock)	1000 to 2000	2.45 to 3.20	2.60 to 3.30	2.80 to 3.05	2.00 to 5.05	2.80 to 6.05*	2.35 to 5.60*	1.60 to 2.00
E. Soft clay soil	< 1000	Medium clay (Matlock)	500 to 1000	3.90 to 4.40	4.20 to 4.45	4.50 to 4.70	3.75 to 6.45	4.15 to 6.10*	4.20 to 4.60*	2.55 to 2.70
		Soft clay (Matlock)	250 to 500	6.55 to 6.50*	6.85 to 6.05*	7.50 to 6.60*	5.50 to 4.85*	6.45 to 3.95*	5.35 to 3.60*	4.15 to 4.25*

Table 3. Permissible displacement limits established for 16 in. octagonal prestressed piles with a fixed pile head and a pinned pile head in different clay soil types

Note: A_g = gross section area of the concrete pile section; f_c' = compressive strength of unconfined concrete; f_{pc} = compressive stress in the concrete at the centroid of the gross section due to prestress (after losses); P = design axial force (derived from overstrength consideration); s_u = average undrained shear strength. 1 in. = 25.4 mm; 1 psi = 6.895 kPa.

* Did not reach ultimate condition due to significantly high P-Δ effects.

If a greater lateral displacement limit is preferred, the required confinement could be established from Eq. (12) with $\mu_\phi > 18$, which can cause reinforcement congestion in the pile and further reduce the inelastic action in the aboveground structure. Both are considered unnecessary consequences.

- Additional analyses were completed on selected 24 in. (610 mm) octagonal, 14 in. (360 mm) square, and 16 in. (410 mm) square piles with a fixed-head condition because this provides the lower-bound permissible values. The permissible lateral displacements determined for these piles ranged from 1.15 to 3.5 in. (29.1 and 89 mm) in very dense sand, dense sand, hard clay, and stiff clay; while the corresponding values for the piles in loose sand and soft clay ranged from 2.25 to 3.95 in. (57.2 to 100 mm). As with the 16 in. octagonal piles, the displacement capacity of each pile decreased with an increase in density or stiffness of the soil.

Impact of soil variation

Unlike the properties of structural materials such as concrete and reinforcement, the properties of soils of a specific type can vary significantly. This will, in turn, affect the load-deflection curves and, thus, the lateral displacement capacities of piles. To understand the influence of the variability in subsurface soil conditions and the selection of soil parameters on pile displacement capacities designed with the proposed confinement equation, an upper-bound load-modification factor of 3/2 and a lower-bound load-modification factor of 2/3 were assumed for the software analyses. This approach essentially provided a ±50% variation for the soil parameters. As before, several 16 in. (410 mm) octagonal piles were analyzed to examine the impact of soil variations on lateral displacement capacity.[11] This analysis set revealed the following:

- The percentage differences in permissible displacement limits caused by ±50% variation in soil parameters were within ±30%.

- The percentage differences in permissible displacement limits ranged from ±3% to ±27% for piles in sand and from ±10% to ±28% for piles in clay.

- The average of the percentage difference in permissible displacement limits for piles in sand was relatively smaller than the average of the percentage difference in permissible displacement limits for piles in clay, which were ±14% and ±17%, respectively.

- The average of the percentage difference in permissible displacement limits for piles with a pinned head was relatively smaller than the average of the percentage difference in permissible displacement limits for piles with a fixed head, which were ±14% and ±18%.

- Piles with axial load ratios of 0.2, 0.45, and 0.5 had average differences of ±15%, ±17%, and ±15% in permissible displacement limits, respectively, indicating no significant influence of the axial load ratio in this investigation.

In consideration of these findings, the pile displacements recognized within Fig. 10 can vary on average by ±15% unless more realistic soil parameters from the site are accounted for, which should be given consideration in the seismic design of structures.

Conclusion

The main objective of the study presented in this paper was to provide a rational and satisfactory approach to quantify the amounts of confinement reinforcement in the plastic hinge regions of prestressed concrete piles to be used in high seismic regions while ensuring constructible transverse reinforcement details. After examining the existing recommendations, which lead to significantly different amounts of confinement reinforcement, a new equation was developed as a function of concrete strength, yield strength of confinement reinforcement, external axial load, area of the pile section with consideration to core area being the effective pile section, and target curvature ductility. Unlike many existing equations, the introduction of the target curvature ductility in the new confinement equation enables reduction in the amount of confinement reinforcement for piles in low and high seismic regions while allowing the reinforcement to be increased if a large curvature ductility demand is expected in a pile section.

The proposed equation displays a somewhat different trend from the existing design equations but provides constructible amounts of confinement reinforcement. Verification of the proposed equation was conducted by performing hundreds of moment-curvature analyses of prestressed concrete pile sections designed with the proposed equation and comparing their curvature and ductility capacities with the target values. These analyses were performed on 16 and 24 in. (410 and 610 mm) octagonal piles as well as on 14 and 16 in. (360 and 410 mm) square piles by varying the concrete strength, axial load ratio, and initial prestress.

Furthermore, a design process that connects the lateral displacements of piles to the required amount of transverse reinforcement was examined as part of this study. Software analyses were performed to establish permissible limits for the lateral displacement of precast, prestressed concrete piles in different soil conditions prior to reaching the curvature capacity of piles that used confinement reinforcement as per the proposed equation. Finally, the impact of the variation of soil parameters on the permissible displacement limits was examined. Conclusions drawn from this study are as follows:

- A review of literature published on pile testing and back analysis of piles subjected to real earthquakes revealed that piles in high seismic regions should be designed with an ultimate curvature capacity of at least 0.00152 in.$^{-1}$ (0.0598 m^{-1}).

- To simplify the design of precast, prestressed concrete piles, a satisfactory approach to idealize actual moment-curvature responses for these piles was developed. This idealization can be summarized as follows:

  — First yield moment ε_c is defined to occur at 0.002 in./in. (0.002 mm/mm) at the extreme concrete compression fiber.

  — Nominal moment is the average of the smallest and the largest moment resistance occurring between the first yield moment and the ultimate moment.

  — Ultimate moment is defined by the first occurrence of either a 20% reduction of the maximum moment resistance, moment corresponding to an ultimate strain in the strand of 0.004 in./in. (0.004 mm/mm), or moment corresponding to the strain capacity of the confined core concrete.

- In the absence of a rational approach to limiting axial loads on piles subjected to lateral loads, a new axial load limit is proposed, which ensures that the pile section curvature at flexural cracking moment is less than the curvature at spalling of cover concrete. Accordingly, the axial load ratio is expected to be limited to 0.45 for 16 and 24 in. (410 and 610 mm) octagonal piles, 0.2 for 14 in. (360 mm) square piles, and 0.25 for 16 in. square piles.

- The proposed equation for the volumetric ratio of transverse reinforcement provides an ultimate curvature capacity of at least 0.00194 in.$^{-1}$ (0.0764 m^{-1}), approximately 28% greater than the suggested minimum ultimate curvature capacity.

- The proposed equation for the volumetric ratio of transverse reinforcement contains a curvature ductility demand term that ensures a curvature ductility capacity of the selected μ_ϕ for the pile. In the absence of a suitable value, μ_ϕ may be taken as 18 in high seismic regions, while smaller values of 6 and 12 may be appropriate for piles in low and moderate seismic regions, respectively.

- The permissible lateral displacement limits for a fixed-head pile and a pinned-head pile embedded in sand range from 1.25 to 4.00 in. (31.8 to 102 mm) and 1.95 to 7.20 in. (49.5 to 183 mm), respectively. For a fixed-head pile and pinned-head pile embedded in clay, the permissible limits on lateral displacement range from 0.95 to 7.50 in. (24 to 191 mm) and 1.25 to 6.60 in. (168 mm), respectively. The upper limits of the lateral displacement for a fixed-head pile and pinned-head pile embedded in sand and clay are excessive, and thus it is suggested that this value be limited to 2 in. (50 mm) (or a similar value) to ensure the likelihood that the structure supported by pile foundations is functional after experiencing an earthquake. This implies further reduction to the confinement requirement and the need to integrate pile foundation flexibility into the seismic design of the aboveground structure. These objectives can be accomplished using the proposed integrated design method.

- The percentage differences in permissible displacement limits caused by $\pm 50\%$ variation in soil parameters were within $\pm 30\%$ with approximate average values of $\pm 15\%$. The average of the percentage differences in permissible displacement limits for piles in sand and piles with a pinned head are relatively smaller than those for piles in clay and piles with a fixed head, respectively.

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Notation

A_{ch} = cross-sectional area of confined core concrete section, measured out to out of the spiral reinforcement as defined by ACI 318-05

A_g = gross section area of the concrete pile section

A_s = total area of prestressing steel

A_{st} = total area of mild longitudinal steel reinforcement

d_b = diameter of the reinforcing bar

D = diameter of concrete pile

D' = core concrete diameter measured to the center of the transverse reinforcement

$f_c^{'}$ = compressive strength of unconfined concrete

$f_{cc}^{'}$ = compressive strength of confined concrete

f_{pc} = compressive stress in the concrete at the centroid of the gross section due to prestress (after losses)

f_y = yield strength of longitudinal reinforcement

f_{yh} = yield strength of transverse reinforcement

k = initial modulus of subgrade reaction (either saturated or dry)

L = length of concrete pile

m = nondimensional ratio = $\dfrac{f_y}{0.85 f_c^{'}}$

M_n = nominal moment capacity

M_y = yield moment

$M_y^{'}$ = first yield moment

N = allowable external axial load

N_{SPT} = field standard penetration resistance for top 100 ft in a standard penetration test

$\overline{N}$ = average field standard penetration resistance for top 100 ft

P = design axial force (derived from overstrength consideration)

P_f = prestressing force

s_u = average undrained shear strength

γ_{dry} = effective unit weight

Δ = deflection

$\Delta_{foundation}$ = foundation deflection

$\Delta_{permissible}$ = permissible deflection

Δ_{target} = target deflection

Δ_u = ultimate deflection

Δ_y = yield deflection

ε_{50} = strain at 50% strength

ε_c = concrete compressive strain

ε_{cu} = ultimate strain capacity of the confined concrete

ε_{su} = strain corresponding to the ultimate strength of confinement reinforcement (= 0.12 for Grade 60 steel)

μ_{system} = system displacment ductility

μ_ϕ = target curvature ductility of the pile section

ρ_l = ratio of nonprestressed longitudinal column reinforcement = A_{st}/A_g

ρ_s = volumetric ratio of spiral confinement reinforcement

ϕ = internal friction angle

ϕ_{cr} = curvature corresponding to the flexural cracking moment

ϕ_{sp} = curvature that initiates crushing and spalling of unconfined cover concrete

ϕ_u = ultimate curvature

ϕ_y = idealized yield curvature

$\phi_y^{'}$ = first yield curvature

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Abstract

The design of prestressed concrete piles in seismic regions is required to include confinement reinforcement in potential plastic hinge regions. However, the existing requirements for quantifying this reinforcement vary significantly, often resulting in unconstructible details. This paper presents a rational approach for designing minimum confinement reinforcement for prestressed concrete piles in seismic regions. By varying key variables, such as the concrete strength, prestressing force, and axial load, the spiral reinforcement quantified according to the proposed approach provides a minimum curvature ductility capacity of about 18, while the resulting ultimate curvature is 28% greater than an estimated target curvature for seismic design. This paper also presents a new axial load limit for prestressed piles, an integrated framework for seismic design of piles and superstructure, the dependency of pile displacement capacity on surrounding soils, and how further reduction to confinement reinforcement could be achieved, especially in medium to soft soils and in moderate to low seismic regions.

Keywords

Axial load limit, confinement, design, foundation, moment-curvature idealization, pile, seismic.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

Reader comments

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