CONFINEMENT REINFORCEMENT REQUIREMENT FOR PRESTRESSED CONCRETE PILES IN HIGH SEISMIC REGIONS

S. Sritharan[1], A. Fanous[2], M. Suleiman[3] and K. Arulmoli[4]

[1] *Wilson Associate Professor, Dept. of Civil, Const. & Env. Eng., Iowa State University, Ames, IA 50011, USA*
[2] *Structural Engineer, Burns & McDonnell, 9400 Ward Parkway, Kansas City, MO 64114, USA*
[3] *Assistant Professor, Civil and Environmental Eng., Lafayette College, Easton, PA 1804, USA*
[4] *Principal, Earth Mechanics, Inc., 17660 Newhope Street, Suite E, Fountain Valley, CA 92708, USA*
Email: sri@iastate.edu, afanous@burnsmcd.com, suleimam@lafayette.edu, arulmoli@earthmech.com

ABSTRACT :

Prestressed concrete piles used in high seismic regions are required to be designed with spiral reinforcement in the potential plastic hinge regions for confinement purposes. However, the design requirements available in the current codes and standards for quantifying this reinforcement are not consistent, nor do they allow the designer to account for the expected maximum curvature ductility of the pile section. In recognition of these challenges, a rational approach to designing confinement reinforcement in potential plastic hinge regions was investigated and an equation to quantify the minimum spiral reinforcement required for prestressed concrete piles in seismic regions was developed. The spiral reinforcement quantified according to this equation leads to a minimum curvature ductility capacity of about 18, while it is shown that the resulting ultimate curvature capacity of a typical pile section would be about 28% above an estimated target curvature for seismic regions around the world. By analyzing different prestressed pile sections with variables such as concrete strength, initial prestressing and axial load ratio, it is shown that the developed equation is adequate for quantifying the confinement reinforcement in prestressed concrete piles in high seismic regions.

KEYWORDS: Precast, presetressed, concrete, pile, seismic, confinement, reinforcement

1. INTRODUCTION

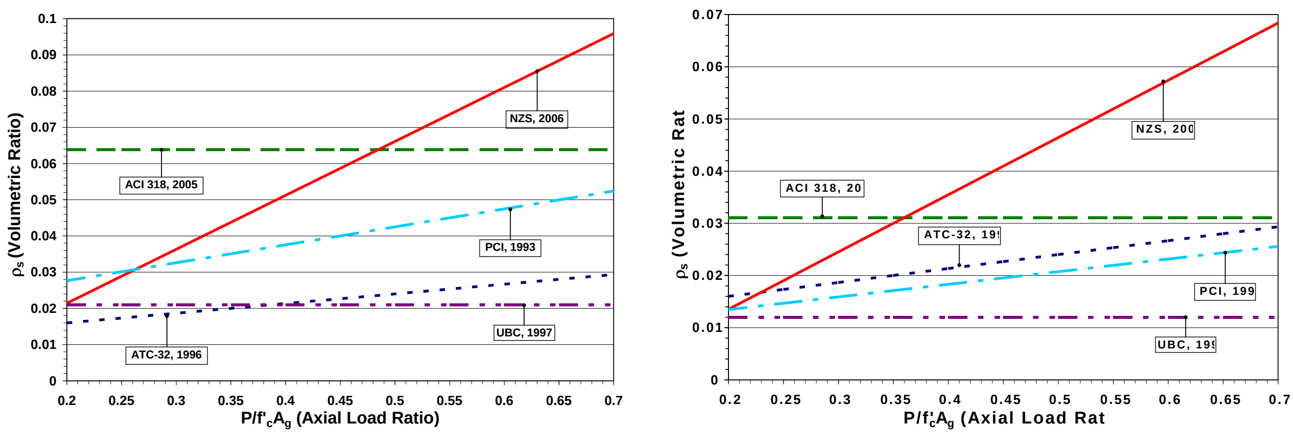
Precast, prestressed piles have been widely used in the design of foundations for bridges, buildings, and wharf structures. In the United States, high seismic regions such as California, Washington, South Carolina, and Alaska adopt certain standards for the design of foundations so that satisfactory performance of structures can be achieved when they are subjected to earthquake motions. As described by Paulay and Priestley (1992) and Priestley et al. (1996), the seismic design philosophy adopted in these regions generally follows the capacity design principles. As stated by Priestley et al. (1996), this design philosophy may be summarized as follows: 1) under design-level earthquakes, structures are allowed to respond inelastically through flexural yielding; 2) locations of plastic hinges are pre-selected and detailed carefully to ensure that structures can develop ductile response; and 3) suitable strength margins are provided to ensure that undesirable mechanisms of inelastic responses cannot occur.

Accordingly, the seismic design philosophy promotes the notion that the foundation elements, including piles, should be inhibited from experiencing inelastic actions by forcing the plastic hinging to occur in the structure at or above the ground surface. 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 under seismic loading. However, preventing inelastic actions in piles that support footings is not always practicable since the moment gradient along the pile length is markedly influenced by properties of soil surrounding the pile (Priestley et al. 1996). Furthermore, the extent of inelastic action that can potentially occur in piles during an actual earthquake is not well understood because earthquake reconnaissance efforts typically do not investigate this issue unless evidence for pile failure is seen at a particular site. Given this uncertainty, a research project was undertaken to: a) determine an appropriate curvature demand through a literature review; b) establish an equation that will provide the minimum amount of transverse reinforcement for prestressed concrete piles while ensuring curvature capacity greater than that established as the potential maximum curvature demand; c) embed a curvature ductility factor within the developed equation in order to reduce the confinement reinforcement where appropriate; d) using the developed equation, determine permissible lateral displacements that the prestressed piles will be able to withstand in different soil conditions; and e) formulate recommendations suitable for the design of confinement reinforcement for precast prestressed piles in seismic regions. A summary of this project is presented; however, the outcomes of (d) are not included due to the space limitation.

2. BACKGROUND

The spiral confinement reinforcement requirements specified for prestressed concrete piles in several codes and standards, including the Uniform Building Code (UBC, 1997), International Building Code (IBC, 2000), the ASCE Minimum Design Loads for Buildings and other Structures (ASCE, 2005), the PCI Recommended Practice (PCI, 1993), the New Zealand Code of Practice for Concrete Structures (NZS, 2006), the Applied Technology Council (ATC-32, 1996), and the American Concrete Institute (ACI, 2005), were examined. Although the confinement requirements specified in some of these documents are comparable to one another, there exists significant differences between the requirements specified by several codes and standards for a give prestressed pile. This is illustrated in Figure 1 by comparing the required volumetric ratio of the transverse reinforcement for two prestressed piles using five different design methods as a function of axial load ratio. In both cases, $f'_c = 8.0$ ksi, $f_{yh} = 60$ ksi, and a 2-inch concrete cover were used. According to Figure 1:

- The required ρ_s for prestressed piles differs significantly between design codes. At both low and high axial loads, this difference is more than a factor of about three.
- Except for the ACI 318-05 and UBC 1997, the required ρ_s increases with an increase in the external compressive axial load ratio.
- The NZS 3101-2006 requires the highest amount of confinement for high external axial loads, whereas ACI 318-02 requires the highest amount of confinement for low external axial loads.
- The ACI requirement for both piles is significantly high at small axial loads and translates to #3 spiral reinforcement at spacing of less than 0.7 inches. Such a requirement is difficult to meet in practice as it causes significant construction challenges.



(a) 14-inch octagonal prestressed pile (b) 24-inch octagonal prestressed pile

Figure 1 Volumetric ratios of spiral reinforcement (ρ_s) required by different codes and standards

3. CRITICAL PARAMETERS

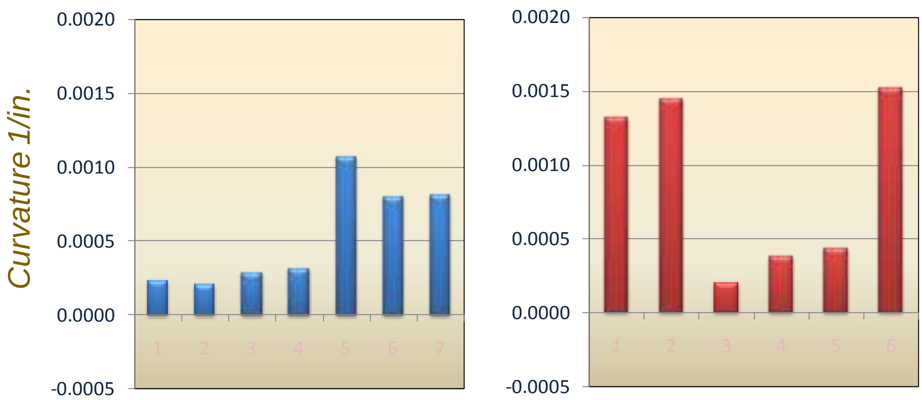
Several different parameters influence the required amount confinement reinforcement in poetical plastic hinge region of prestressed concrete piles. These variables, which were identified based on the existing design methods and theoretical confinement models, are as follows:

A_{ch} = cross sectional area of confined core concrete section, measured out-to-out of the spiral reinforcement
A_g = gross section area of the concrete pile section
A_{st} = total area of mild longitudinal steel reinforcement if exists
d_{sp} = bar diameter of the transverse reinforcement
f'_c = compressive strength of unconfined concrete
f_y = yield strength of longitudinal reinforcement
f_{yh} = yield strength of transverse reinforcement
f_{pc} = compressive stress in the concrete at the centroid of the gross section due to prestress (after losses)
p_t = ratio of non-prestressed longitudinal column reinforcement, which is equal to A_{st}/A_g
P = external axial force

The shortcomings of the existing equations are that: 1) a target curvature demand for which the confinement equation was developed is not specified; and 2) an expected level of ductility capacity is not defined for prestressed or any other pile type. While both of these shortcomings are commonly prevalent in a force-based design method, it is acknowledged that determining a target curvature demand for the given problem is very challenging. In addition, it was found that a consistent, simple approach to idealize the moment-curvature response of prestressed concrete pile sections does not exist and that the axial load limit suggested in codes for prestressed piles are not suitable when they are subjected to both flexural and axial loads. Presented below are discussions that summarize how all of these shortcomings were overcome in the project.

3.1 Target Curvature Demand

A study was undertaken to establish a possible curvature demand for precast, prestressed piles by reviewing: a) measured curvature capacity during large-scale testing of piles that were assumed to have sufficient confinement; and b) back calculated curvature demands on piles that experienced damage during past earthquakes as well as expected demands on piles subjected earthquake loading (Fanous 2007). Summarized in Figure 2 are the results of this study which includes data from testing of seven 12 in. x 12 in. to 16 in. x 16 in. square precast, prestressed piles (see Figure 2a). The data reported in Figure 2b come from cast-in-drill-hole (CIDH) shafts as well as prestressed concrete and steel piles with the maximum demand being reported for a prestresed concrete pile subjected to the 2003 Tokachi-oki Earthquake Koyamada et al. (2005). It is seen in Figure 2 that the reported curvature demand varies from 0.0002 to 0.0011/inch while the capacities range from 0.0002/inch to 0.00107/inch. Furthermore, the maximum reported curvature capacity of 0.0011/inch is about 72 percent of the maximum reported demand of 0.00152/inch, indicating that the piles might have been designed with insufficient curvature capacity and that they may sustain flexural damage when subjected to significant earthquake loading. Despite the limited number of data, this study provided an indication for the maximum curvature demand that should be accounted in the development of a design equation to quantify the confinement reinforcement for prestressed concrete piles.



(a) Curvature capacity from large-scale testing (b) Estimated curvature demand for piles in the field

Figure 2 Curvature demand and capacity reported for piles in the literature

3.2 Moment-Curvature Idealization

The moment-curvature response of prestressed concrete pile sections has somewhat unique characteristics due to the presence of prestressing and significantly large thickness of cover concrete. Therefore, the idealization used for the moment-curvature response of reinforced concrete sections may not be used to idealize a moment-curvature response envelope, which is needed to quantify the curvature ductility of the section. In the absence of an easily applicable bi-linear idealization approach in the literature, several different moment-curvature idealizations were examined. Presented below are the chosen methods to define the first yield, nominal and ultimate moment resistance of a prestressed concrete pile section, which enables the definition of curvature ductility by dividing the ultimate curvature by the curvature corresponding to the nominal moment.

3.2.1 First yield moment

In typical prestressed concrete pile sections with no mild steel reinforcement, their nonlinear response originates when concrete begins to respond in a nonlinear manner. Consequently, the first yield moment for prestressed concrete pile sections is defined using a concrete strain of 0.002 in/in, at which point the stress-strain behavior of concrete is assumed to begin responding nonlinearly. The first yield curvature, ϕ_y', is thus equal to the curvature corresponding to a concrete strain value of 0.002 in./in. or the first yield moment.

3.2.2 Nominal moment

In consideration of the unique moment-curvature response of prestressed concrete pile sections, it was found that defining the nominal moment capacity using a concrete strain of 0.004 in./in. or a strain value in extreme prestressing strand was not satisfactory. Consequently, the nominal moment capacity is defined as the average of the maximum moment and the minimum moment that occurred between the first yield moment and the ultimate moment. This approach was found to be not only simple, but also fairly consistent in providing satisfactory idealized responses. Note that the minimum moment would typically occur 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.

3.2.3 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:

- the ultimate moment is equal to 80 percent of the peak moment resistance of the section;
- the moment corresponding to the first occurrence of a strain of 0.04 in./in. in a prestressing strand; or
- 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.

In typical prestressed pile sections, the ultimate moment is expected to be controlled by the third condition.

Illustrated in Figure 3 is an idealized and actual response of a prestressed concrete pile section, which shows a satisfactory correlation between the two responses.

3.2 Limit on Axial Load Ratio

In consideration of lack of rationales for the existing code limits on external axial loads on prestressed concrete piles, a new limit for axial load ratio was introduced. 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. 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 reduction in moment drop due to spalling of cover concrete is so significant when $\phi_{cr} > \phi_{sp}$. In such cases, the difference between the idealized moment and the actual resistance at curvatures close to ϕ_{cr} was found to be as high as 80 percent and defining the ultimate moment of the pile section and the corresponding curvature was challenging. Also it can be argued that the stability of the pile experiencing significant moment drop may not be dependable and thus the curvature capacity of the pile should be limited to a value close to ϕ_{cr}.

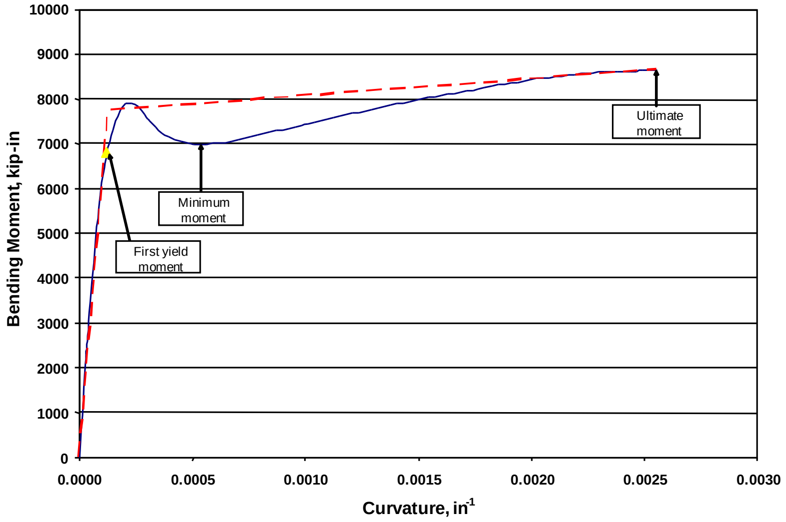


Figure 3 Moment-curvature idealization of a 24-inch octagonal shaped prestressed concrete pile section

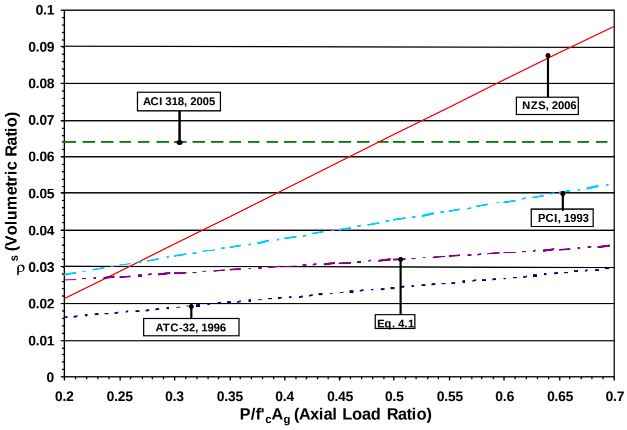
4. PROPOSED EQUATION

Recognizing that axial load ratios on concrete sections are defined using the pile gross section area while the confinement reinforcement is defined for the core concrete of the section, the following equation was established for quantifying the confinement reinforcement for the plastic hinge region of prestressed concrete piles in high seismic regions:

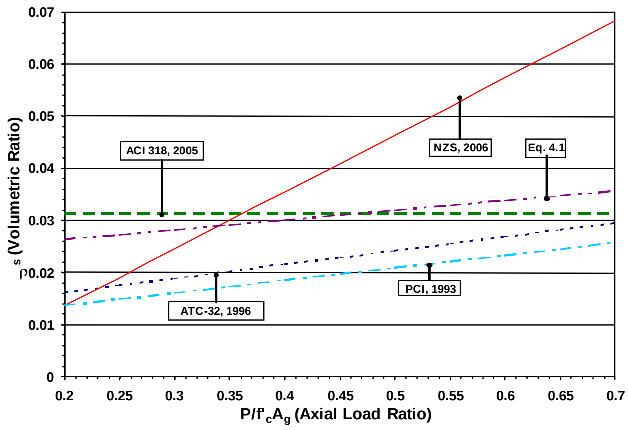
$$\rho_s = 0.06\left(\frac{f_c^{'}}{f_{yh}}\right)\left(\frac{\mu_\phi}{18}\right)\left(2.8+\left(\frac{1.25P}{0.53\,f_c^{'}A_g}\right)\right) \tag{4.1}$$

where μ_ϕ defines the target curvature ductility of the pile section. When a suitable value is not available, μ_ϕ should be taken as 18 for designing prestressed piles in high seismic region. A lower value may be used for piles in low and moderate seismic regions. If required, a value greater than 18 may be used to increase the curvature ductility capacity of a pile section.

With the assumption of $\mu_\phi = 18$, $f_c^{'}$ = 8000 psi, f_{yh} = 60 ksi, and 2 inches of cover concrete, Figure 4 compares the volumetric ratio of two pile sections obtained from Eq. 4.1 with those recommended by existing codes. As can be seen in this figure, the trend of the proposed equation is somewhat different than that proposed by other code equations. For the 14-in. octagonal pile, Eq. 4.1 requires considerably lower transverse reinforcement than that required by the ACI code equation. In contrast, the proposed equation compares well with the ACI recommended confinement requirement for the 24-in. octagonal pile. In comparison to the current PCI and ATC-32 requirements, the proposed equation requires more reinforcement for the 24 in. pile, but not as much as that is required by the NZ standard for high axial load ratios. The reduced amount of confinement required for the 14-in. octagonal pile by the proposed equation is very encouraging. This is because the small pile size increases reinforcement congestion by reducing the spacing of the transverse reinforcement significantly. For the 24-in. pile, the congestion issue could be tackled by increasing the diameter of the confinement reinforcement, which is not practical for small pile sizes.



(a) 14-inch octagonal prestressed pile



(b) 24-inch octagonal prestressed pile

Figure 4 Comparison of required volumetric ratios of spiral reinforcement

5. VERIFICATION

The validity of the confinement equation presented in Eq. 4.1 was investigated by varying the concrete strength, axial load ratio, initial prestress, 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 above. In all cases, the axial load ratio was varied from 0.2 to the maximum limit as defined in Section 3.2. Figure 5 shows results of 152 different octagonal shaped prestressed pile sections, which resulted in an average ductility of 19.4 and standard deviations of ±1.1. Although most of the data points fall above the mean minus standard deviation line (i.e., $\mu_\phi = 18.3$), some of the analyses produced lower ductility capacity below 18.3. The lowest ductility capacity achieved was 17.2, which is only 4.4 percent below the target ductility of 18. Given the different variables used in this particular set of verification, the proposed equation is considered simple and sufficiently accurate for quantifying confinement reinforcement of octagonal shaped prestressed pile sections.

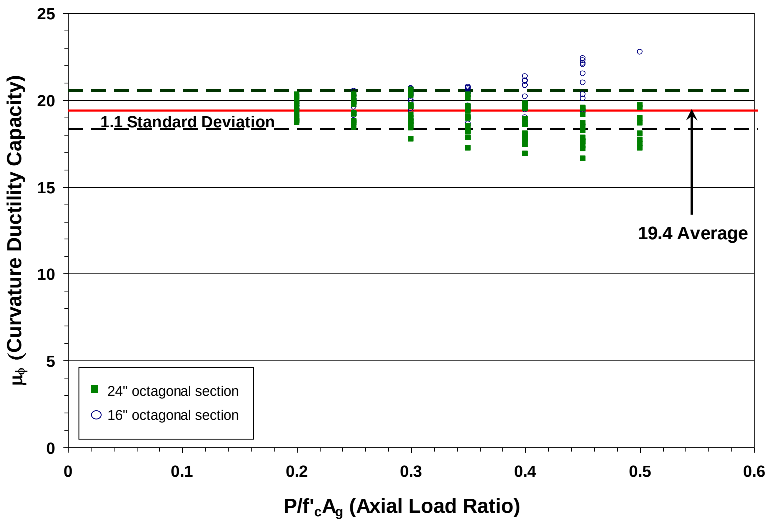


Figure 5 Curvature ductility capacity of 16-inch and 24-inch prestressed pile sections

Similarly, Figure 6 presents the results of 14-in. square pile sections with axial load rations up to 0.25, 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 above 18. No further refinement to the equation was considered necessary since the required amounts of confinement is generally below the currently recommended values.

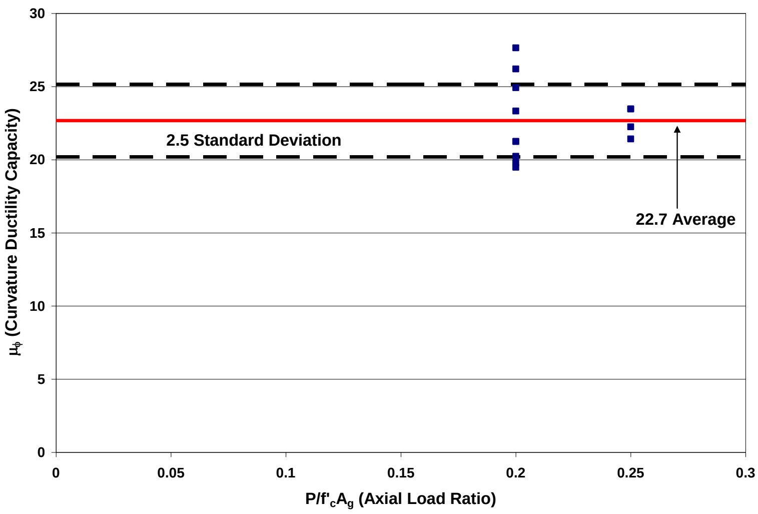
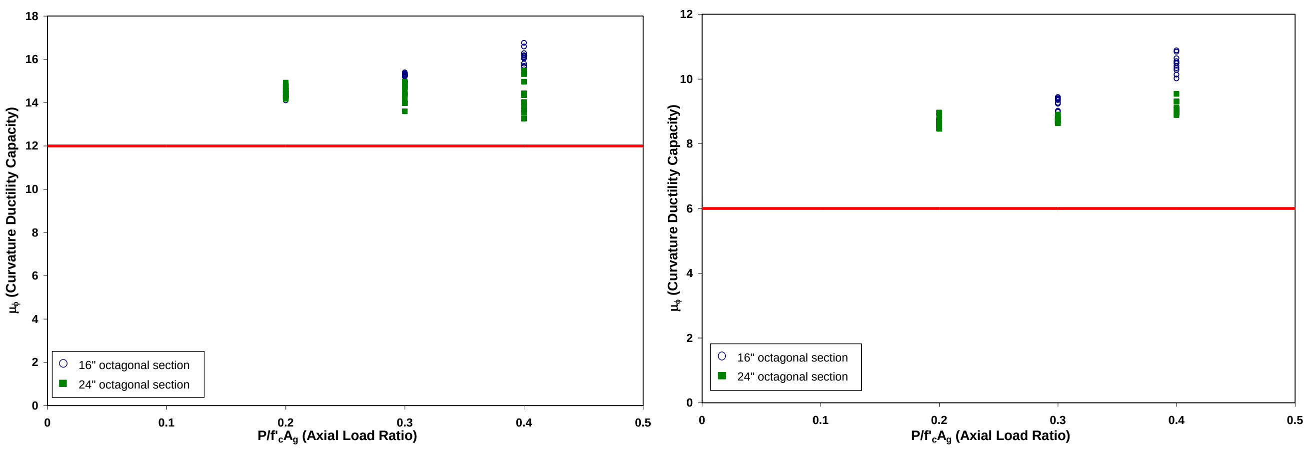


Figure 6 Curvature ductility capacity of 14-inch square pile sections

The confinement reinforcement requirement of Eq. 4.1 was also examined for several other octagonal prestressed pile sections with target curvature ductilities of 12 and 6. The results of these analyses are presented in Figure 7, which shows that the ductility capacity of pile sections with confinement as per Eq. 4.1 was greater than the target ductility in all cases. It is seen that the 16-in. pile section with high axial load ratios consistently produced higher ductility capacity than the target value. Although further refinement may be possible, this was not investigated as the reduction to the confinement reinforcement due to the use of small target ductilities had already resulted in significant reduction to ρ_s in comparison to the current code requirements.



(a) Target curvature ductility = 12 (b) Target curvature ductility = 6

Figure 7 Curvature ductility capacity of octagonal piles designed with small target ductility

Finally, it is noted that 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. This value is about 28% above the curvature of 0.00152/in., which was established as a possible maximum curvature demand for piles in high seismic regions in Section 2, adding more assurance to the proposed confinement equation.

6. CONCLUSIONS

The main objective of the study summarized in this paper was to provide a rational and satisfactory approach to quantifying the amounts of transverse reinforcement for the plastic hinge regions of prestressed concrete piles to be used in high seismic regions while ensuring constructible confinement reinforcement details. In consideration of the currently used confinement requirements, an equation was proposed as a function of concrete strength, yield strength of confinement reinforcement, external axial load, gross sectional area of the pile, and the target curvature ductility. Unlike many existing equations, the introduction of the target curvature ductility in the 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 high curvature ductility demand is expected in a pile section. The proposed equation has somewhat different trend than the existing design equations, but recommends constructible amounts of confinement reinforcement. Verification of the proposed equation was conducted by performing hundreds of moment-curvature analyses of pile sections designed with the proposed equation and comparing their ductility capacities with the target values. These analyses were performed on 16-in. and 24-in. octagonal piles as well as 14-in. square piles by varying the concrete strength, axial load ratio and initial prestress. This verification conclusively shown that the proposed equation for quantifying the confinement reinforcement in prestressed concrete piles is satisfactory and that it can be used in low, moderate and high seismic regions. The minimum value of the curvature capacity established for the prestressed designed according to the proposed equation was 0.00194/in. when target ductility was 18. As part of this study, a moment-curvature idealization and a new limit on axial load ratio were introduced for prestressed concrete piles. Both the new idealization and the axial load limit were found to be satisfactory.

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REFERENCES

ACI Committee 318 (2005). Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (318R-05). *American Concrete Institute*, Farmington Hills, Michigan.

American Society of Civil Engineers (2005). ASCE-7 Minimum Design Loads for Buildings and Other Structures. Alexander Bell Drive, Reston, VA.

Applied Technology Council (ATC)-32 (1996). Improved Seismic Design Criteria for California Bridges: Provisional Recommendations, Applied Technology Council, Redwood City, California.

Fanous A. (2007). Development of Design Methodology for Spiral Reinforcement in Prestressed Concrete Piles in High Seismic Regions. MS Thesis, Iowa State University, Ames, Iowa.

International Building Code (IBC) 2000. *International Code Council*, Falls Church, Virginia.

Uniform Building Code (UBC), 1997. *International Conference of Building Officials*, Whittier, California.

Koyamada, K., Miyamoto, Y., and Tokimatsu, K. (2005). Field Investigation and Analysis Study of Damaged Pile Foundation During the 2003 Tokachi-Oki Earthquake. *Proceedings of Seismic Performance and Simulation of Pile Foundations in Liquefied and Laterally Spreading Ground*, GSP, **145**, 97-108.

Standards Association of New Zealand (2006). NZS 3101 – The Design of Concrete Structures (Part 1: Practice and Part 2: Commentary). Wellington, New Zealand.

Paulay, T. and Priestley M. J. N. (1992). Seismic Design of Reinforced Concrete and Masonry Buildings, John Wiley & Sons, New York.

PCI Committee on Prestressed Concrete Piling 1993. Recommended Practice for Design, Manufacture and Installation of Prestressed Concrete Piling. *PCI Journal*, **38:2**, 64-83.

Priestley, M.J.N., Seible, F., and Calvi, M. (1996). Seismic Design and Retrofit of Bridges, John Wiley & Sons, New York.