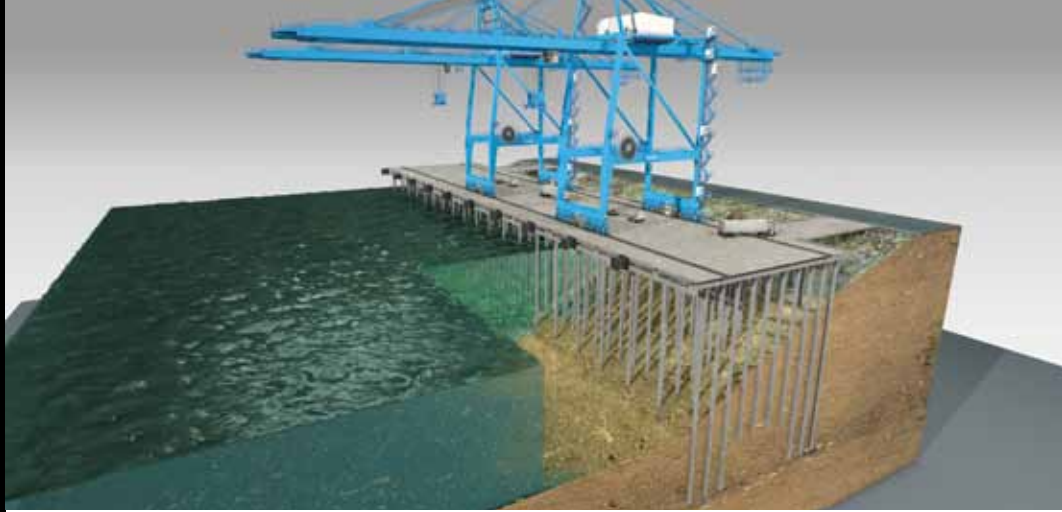


# CODES AND STANDARDS

updates and discussions related to codes and standards



*Three-dimensional schematic showing a typical container wharf and an example of highly variable pile lengths. Water is cut away to show slenderness of waterside piles, often having a length of 60 feet or more from deck level to the mudline below. Piers would be oriented normal to the shoreline with many long piles being needed to support the pier deck.*

## ACI 318 and Pile Stability

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When learning reinforced concrete and structural steel design as an undergraduate in college, one might begin to think that unlike the potential for local and global buckling that might occur for structural steel elements,

concrete accommodates compression very well. While mostly true, concrete piles used for piers and wharfs may be the exception, at least where global instability is concerned. This article reviews slenderness effects for concrete compression elements needed for the pile design, and the apparent direction of future codes. It is possible provision changes in ACI 318 may compromise a critical provision for structures similar to wharfs and piers that may be susceptible to overall instability.

Typically, wharfs run parallel and are often connected to the shoreline and piers are oriented perpendicular to the shoreline. Both types of structures can have pile lengths varying dramatically over their width and/or length. Because the mudline typically slopes near the shore and then remains fairly flat over a considerable distance away from shore, piers generally have far longer piles as a percentage of the total pile count compared to wharfs. However, pile design for wharfs and piers involves one of the more slender concrete compression elements faced by the structural engineer, particularly when the structure must accommodate ships with deep drafts where required water depths along the face needs to be 50 feet or more. While slenderness must be considered for both piers and wharfs, piers are more susceptible to overall pile instability and will thus be the primary focus of this discussion.

Because of their economy and availability, solid 24-inch octagonal prestressed concrete piles are used for many major waterfront

structures along the west coast of the United States. When used in a deep water pier, pile lengths near the shore can be 40 to 50 feet long from the soffit of the pier deck to an assumed point of fixity below the mudline, resulting in an average slenderness ratio of  $l/r = 85$  for the 24-inch octagonal pile section. Piles farthest from shore can often support decks in water as deep as 50 feet or more, resulting in a pile slenderness of  $l/r = 175$  or greater between soffit to point of fixity for the typical octagonal pile section.

Piles supporting a pier may be installed in a plumb (vertical) or battered configuration. In the high seismic region of the west coast, plumb piles are typically preferred, where resistance to lateral loads is due exclusively to flexural resistance of the pile at the top and bottom. Consequently, the effective length factor must be larger than  $k=1$ , and generally a value of  $k=1.2$  is used as a minimum assuming a pile length with good fixity at the pier deck and good knowledge of the location of fixity of the pile below mudline. When considering that the average length of a pile for a long pier can be over 65 feet long, the pier is effectively a 5- to 6-story building with virtually all of its mass at the roof.

To accommodate a wide variety of present and future demands, owners are more frequently requiring piers and wharfs to be designed to accommodate substantial live load, in the range of 600 to 1,000 pounds per square foot. The large gravity load requirements and long slender piles in flexural resistance coalesce into a fairly extreme stability demand on the piles. Design standards, such as the *Minimum Design Loads for Buildings and Other Structures* by the American Society of Civil Engineers, ASCE 7-05 and others, typically specify the requirements of the 2005 edition of the *Building Code Requirements for Structural Concrete* by the American Concrete Institute, ACI 318-05.

The design of slender elements such as these and the method in which elastic buckling is considered in the ACI provisions may not necessarily be apparent. This article summarizes our understanding of the current design philosophy of ACI 318-05 Chapter 10.10 "Slenderness Effects in Compression Elements" and, more importantly, our understanding of the future provisions in ACI 318-08.

In general, the ACI 318 provisions for slender column design address column instability exclusively by the moment magnification method. The 1989 ACI suggests that this is "similar to the procedure used as part of the American Institute of Steel Construction (AISC) specifications." However, there are some distinct differences in the way each code accommodates instability that may not be apparent by casual inspection. In structural steel, the column demand is intended to be amplified for the P-δ and P-Δ effects (slenderness along the element chord and slenderness due to frame sway), in all cases. In addition, the column compressive capacity is limited by either elastic Euler buckling or by the inelastic behavior of the column considering residual stresses. In contrast, the capacity of a long concrete column is the same as the short column capacity (the axial load-moment P-M interaction surface for the cross section) and is therefore not reduced for Euler buckling directly. For long concrete columns, slenderness is accommodated by the moment magnification method in which the moments are amplified depending on if the applied loads induce sway or no sway on the structural system. The general form of this moment amplification is:

$$\delta M_s = \left( \frac{1}{1-Q} \right) M_s \quad \text{Equation 1}$$

where δ is the moment amplification factor and M<sub>s</sub> is the moment on the column induced by loads producing sway of the structural



Section of wharf showing the dramatic variation in pile lengths.

system. Non-sway moments, M<sub>ns</sub>, are amplified by a similar method with associated factors needed for the P-δ effect. ACI 318-05 allows amplified moments, δM<sub>s</sub>, to be computed by one of three methods:

- A second-order elastic frame analysis, often called a P-Δ analysis.
- Use of the approximation  $Q = \Sigma P \Delta / \Sigma V h$ , where ΣP is the sum of all of the gravity loads in the story, Δ is the story drift, ΣV is the total story shear and h is the story height. Q is termed the "stability coefficient" by other codes such as ASCE 7-05, Chapter 12.8.7.
- Substituting Q with the ratio of  $\Sigma P_u / 0.75 \Sigma P_c$  where ΣP<sub>u</sub> is the sum of the factored gravity loads in a story, ΣP<sub>c</sub> is the sum of the critical buckling loads for all columns in the story and 0.75 is a stiffness reduction factor, φ<sub>k</sub>.

Regardless of which method above is used in computing the amplified moments, it is important that the stiffness is reduced for the

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cracked moment of inertia and for creep under sustained loads. These conditions effectively reduce the Euler buckling load to the critical buckling load,  $P_c$ , of a concrete column.

While the moment magnifier method is intended to account for slenderness and ultimately elastic buckling, it is clearly more effective when applied loads, such as wind and seismic, induce large column end moments. A structure that has large axial loads, such as a pier loaded by large gravity loads only, may not necessarily induce large column moments and therefore may be subject to only the minimum required

eccentricity of the axial load. Because the concrete column capacity is not reduced directly by the elastic Euler buckling, and the  $P-\delta$  magnification would not necessarily amplify the small bending moments induced by gravity loads beyond the minimum eccentric moment, column capacities supporting large gravity loads may not be reduced properly for Euler buckling.

One set of provisions that are particularly applicable to this situation are found in Chapter 10.13.6 of ACI 318-05. Ironically, these requirements are within the moment magnification for sway section of the column slenderness provisions of ACI 318-05; however, heavy gravity loads only do not necessarily cause large second-order moments in the sway case, even in a long pier with highly variable pile lengths. Per Section 10.11.4, if applied loads do not cause second order moments greater than 5%, only the nonsway moment magnification provisions of Chapter 10.12 must be considered and there is no need to use provisions of Chapter 10.13. Although not codified, it is intended that a lateral deformation is induced on the structural system, or a "unit load" imposed, to determine if the second-order effects are larger than 5%. So if performed properly, heavy gravity loads in sidesway frames would most likely induce second-order effects larger than 5% which would ultimately lead to the overall frame stability provisions.

The provisions of ACI 318-05 Chapter 10.13.6 are particularly applicable to piers and wharfs, since these structures inevitably have variable pile lengths and can be susceptible to large axial loads. With the variable pile lengths, it could be argued that a single pile cannot buckle individually. Thus, even though all piles might be plumb piles which would qualify it as a sway frame having an effective length factor of  $k=1.2$  minimum, if an individual column cannot buckle independently the effective length can be assumed more consistent with a nonsway frame column, or  $k \leq 1$ . Thus, the pier is only susceptible to instability if all columns are on the verge of buckling, or if enough columns are on the verge of buckling and there is insufficient lateral restraint by the piles not near Euler buckling to prevent global instability. Consequently, determining if the sum of all columns are close to Euler buckling is the appropriate method to determine if a structure with dramatically different pile (column) lengths is near instability.

The provisions of ACI 318-05 Chapter 10.13.6 require the amplification of  $\delta M_u$  for Methods a. and c. to be less than 2.5 and the  $Q$  value in Method b. to be less than 0.6 (which effectively makes it consistent with Methods a. and c.). The understanding of this provision is probably best illustrated by resolving the requirement in Method c. to:

$$\Sigma P_u = 0.6 * 0.75 * \Sigma P_c = 0.6 * \phi_k * \Sigma P_c \quad \text{Equation 2}$$

where  $P_c$  is in effect the Euler buckling capacity reduced for cracked sectional properties and creep. This requirement is comparable to the elastic buckling portion of the AISC steel column capacity requirements  $P_u \leq \phi_c 0.877 P_c$ .



Elevation of wharf showing pile slenderness.

When the provisions of Methods a., b. and c. are plotted for applied axial load vs. the amplification factor, the curve becomes asymptotic to the buckling capacity as predicted by each method. The predicted asymptotic value, without reduction for cracked moment of inertia and creep, is an indicator of how well that provision predicts the Euler buckling capacity. Method a. becomes asymptotic at Euler buckling provided the second-order analytical model is performed properly. Method b. provides the least accurate estimate for elastic buckling, actually overestimating Euler buckling by 1.22 times for the end restrained sway case for plumb piles in piers and wharfs. Method c. results in a value 0.75 times Euler buckling because of the stiffness reduction value,  $\phi_k$ , included in the denominator. Consequently, the method chosen in analysis may have significant consequences on the true estimate of instability of the structure.

Noticing this scatter in the stability estimates from ACI 318-05, it is of interest to compare how the new provisions of the code, ACI 318-08, accommodated this global instability requirement. As has been the case with many concrete code cycles, the slenderness provisions for concrete columns changed. Instead of the global stability provisions being embedded within the *Magnified Moments – Sway Frame* of ACI 318-05, the ACI 318-08 has lumped this into a single general provision of Chapter 10.10.2.1. This provision states that the demand on the structure from second-order effects cannot exceed 1.4 times the linear elastic demand on the system. With this reduction in the limit of second-order effects, the commentary of ACI 318-08 suggests it is no longer necessary to retain the ACI 318-05 Chapter 10.13.6 global instability provisions.

However, we also understand that the ACI column committee is considering adopting the requirement of Chapter 10.10.2.1 of ACI 318-08 as a seismic condition only. It is possible this provision was always intended to be a seismic requirement since the commentary for this section also compares it to the ASCE7-05 Chapter 12.8.7 that is strictly a seismic criterion. In addition, pier and wharf structures are often designed to Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), which also has a limitation on second-order effects for seismic behavior and is nearly identical to ASCE7-05.

Consequently, another provision to limit the amount of  $P-\Delta$  influence on the seismic behavior of the structural system is not really needed by the ACI 318 provisions, at least for most structures using ACI as a reference code for concrete. Plumb piles on a wharf or pier design is perhaps one of the more slender elements experienced in the design of many types of structural systems. A consistent, effective and accurate methodology to prevent the instability of the piles is needed. If the current ACI 318-08 Chapter 10.10.2.1 becomes limited to a seismic provision, the global stability provision of ACI 318-05 Chapter 10.13.6 should be reinstated and should be revised so that regardless of the method used, the predicted overall stability capacity of the structure is consistent. ■