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STRUCTURAL CONCRETE PILE-WHARF CONNECTIONS UNDER CYCLIC LATERAL LOADING

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ABSTRACT

This paper discusses the development of analytical models, using the fiber section technique, to describe the nonlinear structural behavior of concrete pile-wharf connections subjected to cyclic lateral loading. The models are validated in part using experimental results obtained from the literature. At the section level, comparisons are primarily focused on the stress-strain and moment-curvature behaviors, while at the element level comparisons of lateral force and moment vs. displacement are utilized to shed light on the global behavior of a pile-wharf connection. Results indicate that the developed models are able to accurately predict the global and local experimental behavior, even including the occurrence of various damage states. Finally, a study is conducted to explore the impact of using Carbon Fiber Reinforced Polymer (CFRP) jackets at the plastic hinge zone of the pile on the cyclic behavior of the connection. The application of CFRP jackets can delay spalling of the cover concrete and thus enhances the strength and ductility of the connection.

Introduction

The seismic response of port structures like wharves supported on piles can be assumed to be strongly related to a couple of factors: geometric configuration of the structure and soil conditions. Two critical zones are recognized in the piles – one within the soil and the other at the pile-wharf connection. This latter zone may deserve some special consideration because it is usually accessible and therefore could be relatively easy to retrofit. Two types of pile-wharf connections are considered common in the United States. The first is pile-wharf connections with deep pile embedment (representative of construction in the eastern and central U.S.), while the second is pile-wharf connections with very shallow embedment typical on the west-coast of the U.S. Due to increased commercial business between the U.S. and Asia, there is demand for expanding ports and building new ones on the U.S. west-coast. Therefore, special attention is required in the construction of new port facilities or the expansion of existing ones, as for example the Port of Seattle in Fig.1(a).

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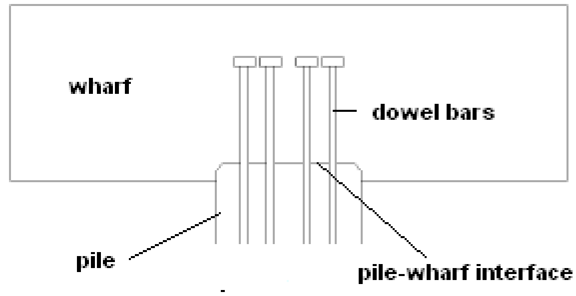


Figure 1. (a) Port of Seattle (Courtesy P. of S.), and (b) Schematic of the pile-wharf connection

Among shallow embedment pile-wharf connections, a common sub-type comprises a precast, prestressed concrete pile, with a cast-in-place wharf deck. The pile and wharf are connected through dowel bars, with the pile embedded 2 to 3 in. into the wharf, as can be seen in Fig. 1(b). Pioneering work on this subject was performed at the University of Canterbury, New Zealand (Joen et al. 1988). Their specimen (15.7 in. octagonal section with four D20 dowel bars) was ranked as the worst connection being tested, because the plastic rotation concentrated undesirable damage at a wide crack near the pile-pile cap interface, but researchers suggested that it would behave better with additional dowel bars. A 1997 study conducted at the University of California San Diego (Silva et al. 1997) illustrated that a full scale pile cap (Caltrans Class 70 ton pile with 6-#6 longitudinal steel bars embedded into the cap) can be susceptible to significant reductions in moment capacity due to major spalling of the pile's cover concrete under cyclic lateral loading with varying axial load. A pile-deck connection that utilized longitudinal dowel bars and T-headed bars acting as bond bars in the joint region was then tested under cyclic lateral loading (with no axial load) for the Port of Los Angeles (POLA) (Sritharan and Priestley 1998). They concluded that the connection details were sufficient to develop the necessary connection ductility.

More recently, a study conducted at the University of Washington (Roeder et al. 2001) involved testing several pile-wharf connections and details with shallow embedment, indicating that such connections could sustain significant damage under reversing lateral loads. In an experimental and analytical study sponsored by POLA, two full-scale pile-deck connections (24 in. octagonal section) were tested under cyclic lateral loading (Restrepo et al. 2007). One test represented the pile-deck connection of non-seismic piles (4#9 dowel bars embedded 17 in. into the deck), while the second test represented a pile-deck connection for seismic piles (8-#10 dowel bars embedded 29 in. into the deck). Both specimens showed that shallow embedment connections can have predictable responses and that the lateral displacement corresponding to the strain limit required by the Code for Seismic Design, Upgrade and Repair of Container Wharves of the Port of Los Angeles (The Port of Los Angeles 2004) can accurately be predicted. Finally, four full-scale specimens were recently tested at the University of Washington to evaluate and compare the performance of different connection details (Jellin 2008). Connection details that included partial debonding of the dowel bars, placing a bearing pad between the pile and the deck, and addition of soft foam wrap around the perimeter of the short embedment length of the pile seemed to have less extensive damage at the pile-wharf interface (though with some loss in connection stiffness).

All of the specimens described above experienced significant physical damage in the form of pile and deck cover spalling and exposure of the spirals and longitudinal reinforcement of the pile at “drifts” between 2% and 5%, even if their flexural capacity were acceptable. Despite the previously discussed experimental work that has been done in this area, there is a considerable lack of analytical tools and models that could be used to investigate and predict the behavior of pile-wharf connections under seismic loading. This paper will focus on developing and calibrating accurate analytical models for structural concrete pile-wharf connections that can be used in future research. The developed models will also be utilized to explore the behavior of connections when retrofitted using Carbon Fiber Reinforced Polymer (CFRP) jackets.

Analytical Model – General Description

The fundamental components of the developed analytical model are the pile, the wharf and their interface. The pile is modeled in a position over the wharf because this is the typical experimental set-up. Its different sections, although always octagonal, model the variable presence and characteristics of concrete (including confinement effects), prestressing strands, and dowel bars, as can be seen schematically in Fig. 2. The wharf itself is globally considered to act as almost a rigid body, but its influence on pile behavior is modeled through the pile-wharf interface characteristics. Due to the fact that the connecting elements are dowel bars, their effect on the interface plastic behavior (mainly due to yield penetration) is carefully modeled. The software used is OpenSees (Mazzoni et al. 2009) due to its convenient fiber approach implementation and material library that includes a wide range of nonlinear material models that have been developed specifically for seismic applications.

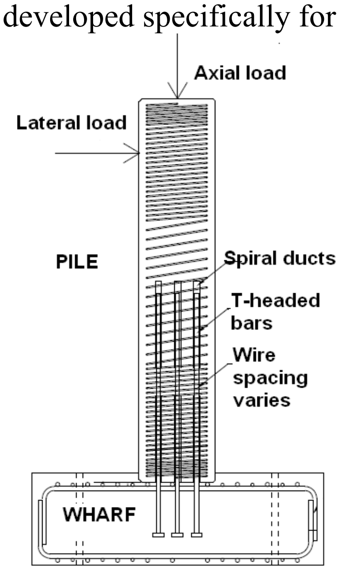


Figure 2. General model characteristics and typical pile section

Pile Model

The fiber-section modeling technique, which is to date the most efficient way to analyze prestressed concrete elements with plastic behavior, is used to model the pile sections. A key role within this analysis is played by the material modeling. Four basic material models are required: unconfined concrete for the cover of the pile section, confined concrete for the core, prestressing steel for the strands, and regular steel for the dowel bars that connect the pile to the wharf.

Concrete Modeling

The OpenSees Concrete01 uniaxial concrete model was used to represent the stress vs. strain behavior of the concrete; it is based on the uniaxial Kent-Scott_Park model with degraded linear unloading/reloading stiffness according to the work of Karsan and Jirsa (1969). In this model, the tensile strength of concrete is neglected. On the other hand the compression behavior is defined by the stress and strain values at the peak and ultimate points. The curve until peak strength is parabolic and then linear thereafter. The curves for unconfined concrete and confined concrete (at the plastic hinge zone) used in this study can be seen in Fig. 3, as well as a curve for concrete retrofitted with CFRP (which will be explained later). In the unconfined concrete the strain at peak strength is calculated from the Thorenfeldt et al. (1987) equations, because they cover a wide range of concrete strengths up to until 18 ksi. Concrete strength is assumed to be zero at the ultimate strain point, which was again calculated using the Thorenfeldt et al. equations.

In the vicinity of the pile-wharf connection, the core of the pile is typically confined with a relatively large amount of transverse reinforcement in the form of spirals. Considering the effect of such confinement on the constitutive behavior of the core concrete is crucial. Input parameters of the Concrete01 material model were modified to consider the effect of confinement. Stress and strain at the peak point were determined based on the model presented by Mander et al. (1988), but the concrete residual stress (20% of the peak strength) and strain (approximately ten times the strain at peak strength) were considered taking into account typical values for an ultimate sliding frictional shear failure mode (Mazzoni et al. 2009).

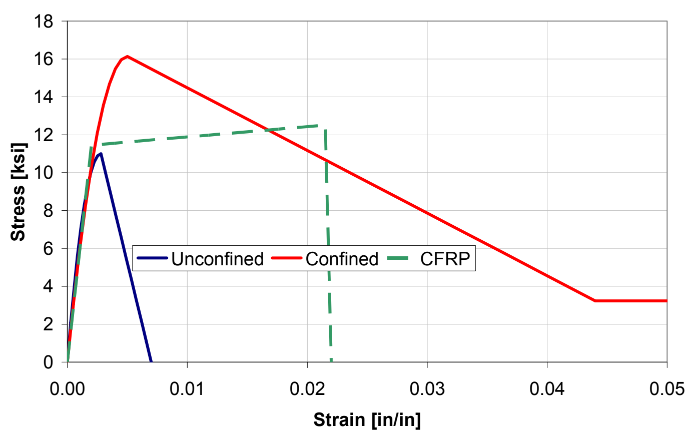


Figure 3. Unconfined, confined and CFRP retrofitted concrete models

Steel modeling

As illustrated in Fig. 2, the pile is typically reinforced with mild steel (in the form of dowels crossing the pile-wharf interface) and prestressing steel. The uniaxial material model Steel02 available in the OpenSees material library was used to describe the behavior of both reinforcement types. In this model, the reinforcing steel stress-strain behavior is described by the nonlinear model of Menegotto and Pinto (1973). For members like piles, which have low to moderate axial compression levels, the monotonic stress-strain curve provides a reasonable envelope to the cyclic response in the tension range but not in the compression range (Priestley et al. 1996). Fortunately the contribution of steel to compression is not as important as that of concrete; the Menegotto-Pinto model is therefore used in its standard form. It is expected, however, that the prestressing steel would have minimal effect on the behavior of the pile near the connection, mainly due to the limited stresses transferred between the prestressing steel and the surrounding concrete in that region. Nevertheless the prestressing steel plays an important role in reducing cracking throughout the pile length.

Pile-Wharf Interface Model

Careful modeling of the inelastic rotation at the end of the pile is of the utmost importance because it is associated with the extent of structural damage and to retrofit strategies. Ignoring this can overestimate the contribution of the regular flexural mechanisms and thereby overestimate the structural damage (Zhao and Sritharan 2007). The pile end inelastic rotation consists of two components: concrete softening and reinforcement slip. Because of the existence of these two components, the pile end inelastic rotation will be modeled through a rotational spring with fiber section that includes concrete and steel, each of them working independently (like parallel members).This spring will use a zero-length section element, because of its generality that allows the strain penetration effect to be captured regardless of the cross-sectional shape and direction of the lateral load (Zhao and Sritharan 2007).

Rotational Spring

Rotation on a flexural crack is equal to slip of the rebar divided by the height of the reinforcing bar to the crack tip. This height depends on the concrete plastic (softening) characteristics (Oehlers et al. 2009). The concrete model presented by Oehlers et al. (2009) was deemed a convenient method to replicate the plastic behavior of the concrete at the interface. As illustrated in Fig. 4(a), under excessive lateral load and due to upward pressure on the pile base, a compressive wedge will develop. Upon formation of this wedge, the plastic behavior of the concrete at the interface will be primarily governed by the transverse reinforcement in the pile (as well as by the wharf concrete surrounding the pile). As illustrated in Fig. 4(a), the forces parallel and perpendicular to the inclined face (the shear plane) can be calculated by equilibrium and then related using classical Mohr-Coulomb shear failure plane theory. The effective longitudinal compressive strength of the softening zone at the onset of shear friction failure is σ_{soft}:

$$\sigma_{soft} = \frac{c + \sigma_{lat} \cos\alpha(\sin\alpha + m\cos\alpha)}{\sin\alpha(\cos\alpha - m\sin\alpha)} \tag{1}$$

where c is the cohesive term, σ_{lat} is confinement pressure due to internal steel spiral and/or external FRP jacket, m is slope of the Mohr-Coulomb failure plane equation, and α is the angle between the horizontal and inclined faces of the weakest wedge:

$$\alpha = \tan^{-1}\left(-m + \sqrt{m^2 + 1}\right) \tag{2}$$

The rising branch of the concrete stress vs. strain curve follows the usual parabola. Its peak values are the unconfined compressive strength f_{co} and strain at peak unconfined strength ε_{co}. Upon further increase in strain, even though concrete strength has not reached its confined maximum, shear-friction wedges occur. Note that the depth of the softening zone can increase until a limit that depends on interface slip. The approach described above was implemented through the Concrete01 model and can be seen in Fig. 4(b).

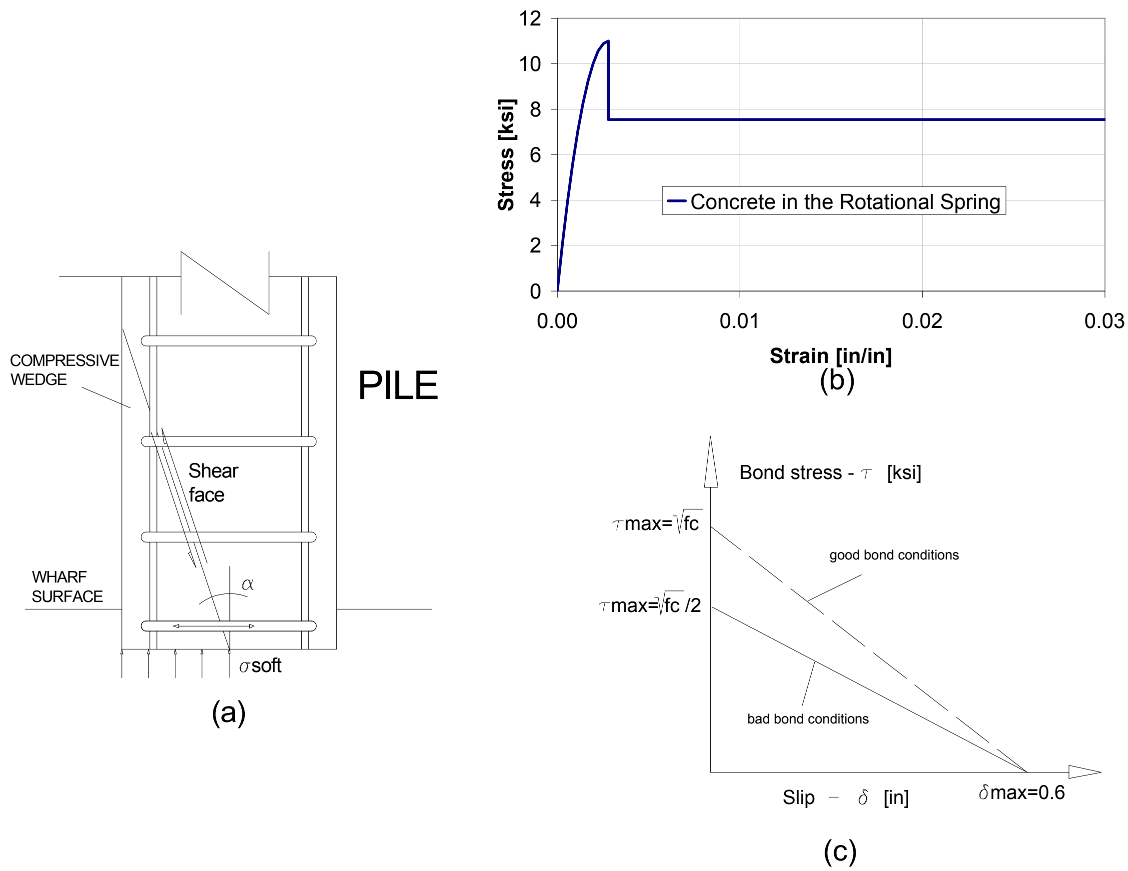


Figure 4. (a) and (b) Concrete softening (Oehlers et al. 2009) and (c) Bond stresses for reinforcing bars (Haskett et al. 2009)

Careful modeling of steel slip is the most important factor for describing yield penetration, as is also explained by Zhao and Sritharan (2007). The steel model presented by Haskett et al. (2009) is used in this paper because it gives a mathematical explanation, in contrast with experimental, to yield penetration of the reinforcing bars and therefore can be easily adapted to different bond conditions. This model states that for reinforcing bars remaining linear elastic and that do not fracture prior to debonding:

$$P_{rebar} = \frac{\tau_{\max}\,\pi\, db}{\lambda} \sin\left\{\arccos\left(\frac{\delta_{\max} - \Delta_{rebar}}{\delta_{\max}}\right)\right\} \tag{3a}$$

where $\lambda^2 = \frac{4\tau_{\max}}{\delta_{\max} db\, E}$ (3b)

P_{rebar} is the force in the dowel reinforcement, $\tau_{\max}$ is the maximum bond strength, db is the diameter of longitudinal steel, $\delta_{\max}$ is the slip after which the bond stress remains at zero, Δ_{rebar} is the bar slip, and E is the steel modulus of elasticity. Fig. 4(c) shows two curves for different bond conditions. These equations are then modified to take into account the steel elastic and strain hardening regions and are limited to the fracture strength of the steel dowel bars. This limit is often used for shallow embedment pile-wharf connections (Zhao and Sritharan 2007). These equations can be modeled through the Hysteretic OpenSees material model because of its simplicity regarding the cyclic modeling behavior. (In a zero-length section element, which is assumed to have a unit length, the element "deformation" (in this case slip) is equal to the section deformation (in this case, longitudinal strain).)

CFRP Retrofitting

Part of the scope of this work was to explore the possibility of using CFRP jackets to reduce the damage sustained by piles adjacent to the connection under cyclic loading. To do so, the analytical model proposed by Saiidi et al. (2005) was adopted and used to predict concrete properties assuming that it is confined by CFRP sheets. A CFRP jacket (12 inch. in length) was designed for confinement stress of 300 psi and design ultimate strain of 0.004 in the fiber direction (Caltrans 1996). The modulus of elasticity of CFRP was assumed to be 29,200 ksi. The model gives among other results a 0.5% CFRP volumetric ratio, which corresponds to 14 layers of CFRP.

Model Validation

The analytical model presented in this paper was checked against the experimental results obtained at the University of Washington in their Specimen 9 (Jellin 2008). The precast, prestressed concrete pile had an octagonal section 24 in. in diameter and a length of 103 in. from the interface to the point of lateral load application. The cast-in-place wharf was simulated by a rectangular deck: 92.5 in. in length, 52 in. in width and 29 in. in height.

Specimen Material Description

The concrete used in the piles had 8000 psi design compressive strength. T-headed bars were grouted into ducts within the pile and cast into the deck to create a moment connection. These bars were eight #10, ASTM 706 (F_y = 70 ksi), 76 in. in length. The pile also had spiral reinforcing of W11 wire, at a center-to-center pitch varying from 1 to 3 in. Prestressing was achieved using twenty-two, 0.5 in. diameter, 270 ksi, low-relaxation strands, stressed to 31 kips/strand. The pile was subjected to an axial load of 450 kips and to a lateral load history consisting of cycles between 0 and 9% "drift".

Moment vs. Curvature Graphs

At the critical section (near the end of the pile), the analytical and experimental moment vs. curvature curves are shown in Fig. 5.

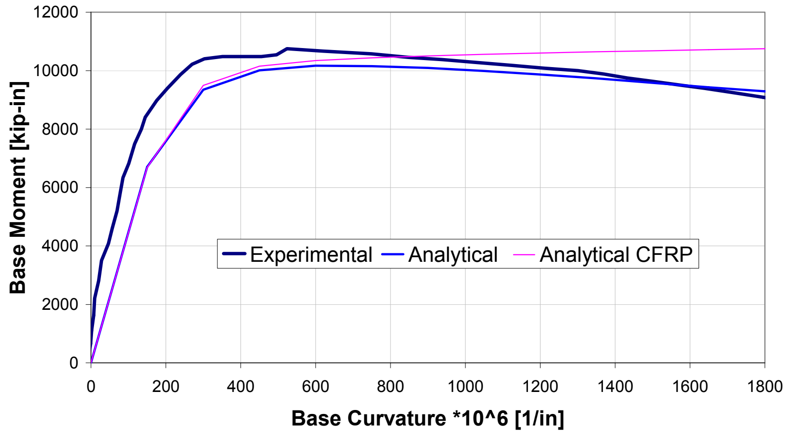
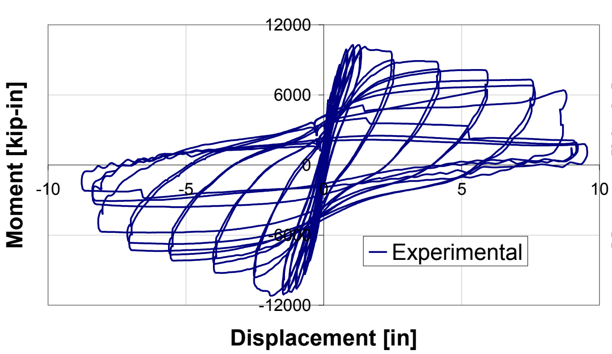


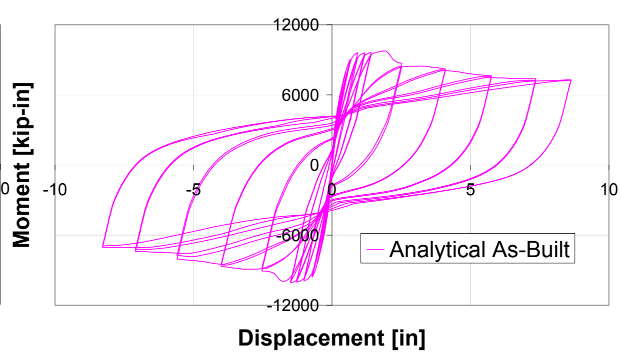
Figure 5. Curvature vs. Moment Graphs

Note that results using CFRP wrapping have also been included in the figure and show that the CFRP confinement produced a significant strength enhancement. To simplify the analysis, the octagonal pile section was modeled as a circular section; also the tensile strength of concrete was neglected. These simplifications are expected to be the main reasons for the slight difference in strength and initial slope observed between the analytical and experimental results. It is important to note that as illustrated in Fig. 5, since the concrete cover represents a relatively large portion of the pile section, it plays an important role in defining the moment capacity of the section. Hence, after cover spalling, a relatively large reduction in the moment capacity was observed.

Displacement vs. Moment Graphs

Displacement vs. moment relationships are shown in Fig. 6 for two analytical models and compared with the experimental results. The first analytical model ("Analytical As-Built") considered the pile-wharf connection modeled as described above, including the rotational spring (but not the CFRP retrofit). The second analytical model ("Analytical Retrofitted") shows the effect of including a CFRP jacket of a length of 12 in. at the pile end adjacent to the connection to the wharf.





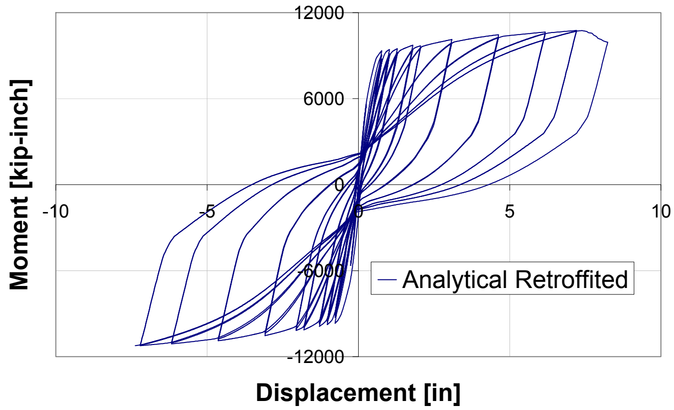


Figure 6. Displacement vs. Moment Graphs.

The results of the model "Analytical As-Built" are in very good agreement with the experimental results, except for in the extreme strength deterioration zone. On the other hand the CFRP additional confinement basically produces a moderate but continuous increase in peak moment capacity in comparison to the model without retrofit. This behavior extends to almost the entire range of cyclic displacements.

Conclusions

This paper developed an analytical fiber model to explain the behavior of a structural concrete shallow embedment pile-wharf connection typical of the western coast of the United States. Materials like unconfined and confined concrete, as well as regular and prestressing steel, have been presented, in addition to models for crushing concrete and steel bar slip (which where necessary to accurately describe the rotation at the pile-wharf interface). The analytical results were compared with different experimental results, emphasizing local and global behavior through moment vs. curvature and moment vs. displacements graphs, respectively; good agreement was found between them. And finally a basic model for CFRP retrofitting was introduced to explore the effect of strength enhancement in the pile-wharf connection.

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