SEISMIC MODELING, ANALYSIS AND DESIGN OF STRUCTURAL CONCRETE PILE-DECK CONNECTIONS

BY

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DISSERTATION

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This dissertation encompasses three primary efforts related to the assessment of the seismic behavior of shallow embedment, structural concrete pile-deck connections: (i) the implementation and validation of a computational model that includes the resulting rotations due to slip of reinforcing dowels at the pile-deck interface, (ii) the evaluation of damage indices reported in the literature, for use in characterizing the ensuing nonlinear behavior, and (iii) a parametric study of structural interactions affecting pile-deck connections.

Initially, a fiber-based computational model, comprising nonlinear distributed pile elements and a zero-length rotational spring, is developed to model dowel connection slippage at the pile end. A comparison with four, full-scale experimental specimens shows good agreement, at both the sectional and element levels, with this simplified model, which represents the prestressed concrete pile as a cantilever supported by the cast-in-place deck. The observed damage in these specimens also correlates well with earlier reported measures, such as the Park and Ang damage index. While additional research could be required to extend and verify this damage measure for high-strength concrete, the moment-rotation envelopes needed for implementation in lumped plasticity models have already been developed.

A more refined computational approach then includes full-length piles, and thus soil-structure interaction and in-ground nonlinear effects are accounted for. Rotational ductility is identified as a critical parameter to assess the structural behavior of pile-deck connections at a moderate to severely damaged state. A factorial-based statistical analysis demonstrates the importance of interactions between concrete strength, spiral reinforcement, and axial force. Global and local effects, either due to concrete- or steel-controlled structural behavior, are summarized in a series of curves generated using relationships revealed by the factorial analysis.
The collective work forms the basis for potential revisions to PCI and other code equations, where the amount of spiral reinforcement can now be prescribed as a function of rotational ductility.
ACKNOWLEDGMENTS

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CHAPTER 1: INTRODUCTION

1.1 Motivation

Civil structures are vulnerable to natural and man-made hazards. Among the common natural hazards, history had shown that earthquakes could impose not only human but also economic losses that could have a long lasting impact on modern societies. In the past few decades, earthquake engineering has undergone a remarkable development in order to control and reduce the seismic vulnerability of civil engineering systems. However, each year, new earthquakes, and their induced structural damages, show the need for continuing studying of these natural phenomena and proposing new and better analysis and design alternatives. Recent examples, such as the Haiti earthquake of 12 January 2010, which was more than twice as lethal as any previous magnitude-7.0 event (230000 deaths, 300000 injured, more than one million homeless), showed once again the need to control the quality of construction and design through strict building codes and standards. But even in this case, as for instance in the Chilean earthquake of 27 February 2010, the economic losses are still too heavy. Twelve million people, or ¾ of the population, were in areas that felt strong shaking from this 8.8-magnitude earthquake. Yet only 577 fatalities were reported. However, the economic losses totaled $30 billion or 17% of the Gross Domestic Product (GDP) of the Chilean country (American Red Cross Multidisciplinary Team 2011). Even countries with state-of-the-art building codes are not totally immune from earthquake disasters. The magnitude-9.0 Tohoku earthquake in Japan on 11 March 2011 showed that engineering designed structures can resist earthquakes with relatively controlled damage, but post-earthquake phenomena such as tsunamis, can still be catastrophic. The Christchurch 6.1-magnitude earthquake in New Zealand that occurred on 22 February 2011 showed that urban devastation can be triggered even by moderate-sized earthquakes, if they
occur directly beneath the cities. This last earthquake, even though 22000 times less energetic than the Tohoku earthquake, killed nearly 200 people and caused more than $12 billion in losses (Hamburger and Moorey 2011).

In USA, Pacific coast is an area of high seismic risk and vulnerability. It is also the gateway for Asia trade. In case of a major earthquake, the economic flow that passes through it should not be interrupted or at least reduced the least possible. An important element for this to occur is the ability of port structural systems to withstand earthquakes. Among these crucial structural systems, wharves supported on piles are very common. Figure 1.1 shows a picture of a container wharf in the Port of Los Angeles as well as a typical wharf elevation, including soil conditions and the huge crane used to move containers.

(a) Berth 100 in Port of Los Angeles          (b) Typical wharf elevation

Figure 1.1 – Container wharf in Port of Los Angeles

Past experiences, such as the San Francisco earthquake in April 18, 1906, or the Alaska earthquake in March 27, 1964 have shown that a combination of seismic activity, subsidence, post-earthquake tsunamis and/or earthquake induced fires produced urban devastation, including damage to structures such as those shown in Figure 1.2.
Figure 1.2 – Urban and port devastation triggered by earthquakes

Figure 1.2 a) shows the fires induced by the San Francisco earthquake (1906) that devastated the city. Figure 1.2 b) shows that the deck of the highway bridge at Womens Bay was washed away by seismic sea waves after the Alaska earthquake in 1964.

Liquefaction of underlying soils is one of the main reasons for the damages sustained in past earthquakes, since it triggers settlement, deformation and soil instability. Figure 1.3 shows some effects of soil failure in the Imperial Valley earthquake, in October 15, 1979, as well as damage produced by soil liquefaction in the Loma Prieta earthquake, in October 17, 1989.

Figure 1.3 – Damage triggered by soil settlement and liquefaction
Figure 1.3 a) shows slumping of the riverbank around the piles of the bridge at site 32 west of Brawley. The soil moved 10 cm. past the stationary piles, a gap opened on the downslope side, and soil bunched up on the upslope side. Figure 1.3 b) shows the damage approach and abutment of the bridge linking the Moss Landing spit to the main land. Liquefaction of the beach and Salinas River deposits caused ground cracking and differential settlement.

The Loma Prieta earthquake (1989), among others, had shown that a crucial element in port structures such as wharves supported on piles is the pile-deck connection. Indeed, under major earthquakes, this connection suffers considerable damage as is described in detail in Chapter 2. Although this damage does not necessarily cause the wharf to collapse, it requires excessive repairs, which will most likely result in the shutdown of the port for an extended period. Moreover, while the damage is being repaired, the commercial flow goes to other ports, and hardly returns to the original port. A typical and relatively recent example is the Port of Kobe. It was affected by the 1995 earthquake which, similar to the earthquakes that hit the U.S. West Coast, was induced by phenomena related to tectonic movements in the circum-Pacific seismic belt. Before the 1995 earthquake, Kobe was the sixth largest port in the world in terms of container cargo throughput and, in the aftermath of the earthquake, is now the 42\textsuperscript{th} largest (AAPA 2010). Such experiences illustrate the dire need for reliable analytical tools capable of predicting with accuracy the extent of damage sustained by pile-deck connections under severe earthquakes, allowing for appropriate pre- and post-disaster actions to be taken to mitigate such damage and its consequences.
Figure 1.4 shows a typical pile-deck connection. As illustrated in the figure, the conventionally reinforced concrete deck is usually cast-in-place, while the octagonal piles are made of precast-prestressed concrete. The pile typically has a shallow embedment into the deck of about 50 to 75 mm (2 to 3 inches), with the deck and pile joined together by T-headed dowel bars that are grouted into ducts in the pile and well-anchored into the wharf deck slab. The piles are conservatively reinforced with spirals in the transverse direction; however, as shown in Figure 1.4, this reinforcement usually does not extend into the deck. The type of connection described above is important because it is being widely used in port facilities as an economical and constructible structural solution for joining the wharf deck slab to the supporting piles.

On the other hand, extensive experimental research has been developed with respect to the pile-deck connection (Joen et al. 1988, Silva et al. 1997, Sritharan and Priestley 1998, Roeder et al. 2002, Krier 2006, Restrepo et al. 2007, Bell 2008, Jellin 2008, Stringer 2010, Blandon et al. 2011, Foltz 2011). Figure 1.4 shows a schematic view (elevation and section) of a typical
example of such a pile-deck connection.

Unfortunately, it should be remarked that the experimental research reported on the behavior of wharf pile-deck connections has not been accompanied by a comparable development in analytical models. Only relatively recently, several analytical works have been presented to the research community, among them is the work done by (Shafieezadeh et al. 2012) and (Yang et al. 2012). There are several behaviors that have been observed in the laboratory, yet still need to be described more accurately analytically. An example of these behaviors is bar slip at the pile-deck connection and its impact on the overall seismic/cyclic behavior of the connection. Low-cycle fatigue and buckling of steel dowels are two other important features that, in fact, control the strength degradation of the pile-deck connection under cyclic loads. Any analytical model capable of describing accurately the pile-deck connection behavior must take them into account. In summary, the plastic zone likely to develop at the pile-deck connection is unconventional, since other factors such as those explained above should be added to the usual flexural factors to explain its structural behavior, and requires special care when being described analytically.

Once the history of displacements and forces or moments in the pile-deck connection experimental research is simulated analytically, it is important to relate the experimental damage to the values achieved by certain parameters in the analytical model. Currently, in guidelines such as the Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), concrete and steel crucial strains are used as damage status flags. However, the commercial software used by practitioners does not provide such type of data. Therefore, there is a need for guidelines for practitioners to assess damage using data available in commercial software. The possibility to use alternative indexes rather than crucial strains, in the case of pile-deck connections, has not
yet been explored.

While experimental research has focused on the behavior of the pile-deck connection, the importance of a second plastic zone (in-ground hinge) in the pile is also recognized. Some studies (Goel 2010) indicated that its presence could affect the behavior of the pile. It is therefore necessary to examine whether this is indeed a significant factor.

The study of the structural behavior of the wharves shows that seismic damage concentrates in the piles closer to the land, see in Figure 1.5 the zones marked with circles. Therefore, it is necessary to pay special attention to them when modeling the wharf structure. In fact, these piles are called “seismic piles” since, due to their rigidity, they attract larger seismic forces. The other more flexible piles only suffer minor damage during earthquakes and are called “nonseismic piles”. The substructure formed by the seismic piles has got the primary attention of researchers including experimental studies that only comprise this type of pile (Kawamata 2009).

![Figure 1.5 - Damage zones in wharf structures and soil characteristics](image)

Figure 1.5 also shows the importance of the soil-structure interaction. An analytical model of the wharf has to take into account this important factor as well.
Finally, and according to current design guidelines, pile-deck connections are heavily confined to ensure adequate compressive capacity and ductility. This is also a key point, because the longitudinal steel can only develop its competency in the nonlinear range, if the concrete is capable of relatively high compressive stresses at relatively high strains. However codes such as the Precast/Prestressed Concrete Institute (PCI) Code, the International Building Code (IBC), or even empirical values used for pile design, specify spiral reinforcement amounts that vary substantially. Therefore, it is necessary to clarify the interactions between spiral reinforcement ratio and other structural parameters that affect the behavior of the pile-deck connection. In this way, the relative accuracy of these code equations can be clarified and, if necessary, new equations can be proposed.

1.2 Objectives

As a first objective, this work develops and calibrates optimal analytical models for structural concrete shallow pile-deck connections that can be used in future research. The main original contribution to this topic is that the effect of the slip of the rebars is explicitly modeled with a separate independent fiber-based rotational spring. Note that the word “optimal” is used to refer to the complexity that allows maximum realistic realizations. In other words, the analytical models should take into account enough details to allow an accurate estimation of nonlinear phenomena, but at the same time should be easy to understand and apply by practitioners. As discussed briefly in a previous paragraph, despite the experimental works that have been done in this area, there is a general lack of companion analytical tools and models to use for investigating and predicting the structural behavior and damage of such connections under seismic loading. This can be an important issue for the rapidly emerging performance-based and/or displacement-based design approaches, as structural engineers require rational and yet sufficiently simplified
analytical models to define behavior and damage in terms of local parameters such as strain and curvature, as well as global parameters such as rotational ductility.

A second objective of this work is studying the feasibility of damage estimation of pile-deck connections through the use of damage indexes. In fact, to date, damage indexes have not previously been used for pile-deck connections. Providing reliable methodologies for predicting the level of damage either locally or globally is essential for assessing whether desired performance level can in fact be satisfied for certain hazards.

As a third objective, the most important structural factors needed to obtain a reliable structural behavior at the pile-deck connection are assessed. Detailed descriptions of the interaction between critical structural parameters, and comparison with results using PCI design handbook spiral reinforcement equations is the main contribution of this dissertation on this topic.

Finally, and as a fourth objective, envelope curves for use in simplified analytical models of pile-deck connections are developed, taking into account performance-based criteria. Even though these curves are based in part on ASCE/SEI 41 (Elwood et al. 2007), their improved level of detail could be considered as the main achievement on this topic of the dissertation.

1.3 Organization of this dissertation

This Ph.D. dissertation is divided into eight chapters including this “Introduction” chapter (Chapter 1). Chapter 2 presents literature review on research topics which are relevant to the proposed dissertation topics. Chapter 3 describes the development of sectional and member models, and validation using experimental results. Chapter 4 provides a brief review of damage indexes and an examination of their applicability to pile-deck connections. Chapter 5 describes a 2D wharf model that highlights the crucial damage zones in the pile considering soil-structure
interaction. Chapter 6 presents a detailed parametric study to investigate the impact of several design parameters on the structural behavior of the pile-deck connection. Chapter 7 presents recommendations for developing an envelope moment-rotation curve to be used in the design/assessment of pile-deck connections using simplified lumped plasticity models. Finally, Chapter 8 is devoted to conclusions.
CHAPTER 2: LITERATURE REVIEW

This chapter is mainly dedicated to three objectives:

- Document problems encountered with pile-deck connections during past earthquakes.
- Highlight the experimental and analytical work done in this area.
- Present an overview of the design guidelines and codes dealing with the design of pile-deck connections.

Past earthquakes have shown that wharf structures supported on piles are susceptible to damage mainly at the pile-deck connection. Typical examples are the Loma Prieta (1989), and Kobe (1995) earthquakes. Recently, the Haiti’s earthquake (2010) showed again the sensibility of the pile-deck connection to seismic damage, and the resulting loss of functionality of the port structures which are key elements for the economic and social life of a community.

A number of important research works on full-scale pile-deck connections are available in the literature. The pioneering work was developed at the University of Canterbury, New Zealand (Joen et al 1988). Other additional and important experimental work has been developed at the University of Washington (Roeder et al. 2002, Jellin 2008, Brackman 2009, Stringer 2010), the University of California at San Diego (Silva et al 1997, Restrepo et al. 2007, Bell 2008, among others), and the University of Illinois at Urbana-Champaign (Foltz 2011), as explained in this chapter.

Different design codes, among them the widely used Precast/Prestressed Concrete Institute (PCI) and International Building Code (IBC) codes, recommend different amounts of spiral reinforcement to be used at a pile-deck connection. The differences among these codes will be also highlighted in this chapter.
2.1 Performance of pile-deck connections during past earthquakes

2.1.1 Loma Prieta earthquake (1989)

On October 17, 1989, the San Francisco Bay Area was shaken by the Loma Prieta earthquake (magnitude 6.9). This earthquake caused severe damage in some very specific locations, most of them related to water-saturated unconsolidated mud, sand and rubble soils which suffered liquefaction. Figure 2.1 shows the damage at a precast prestressed pile-deck connection.

Figure 2.1 – Damage at a pile-deck connection, Loma Prieta earthquake (1989)

(source: Serventi et al. 2004)
Figure 2.1 shows structural damage which is concentrated at the pile-deck connection. It is evident that the concrete cover has completely spalled off, and even the core concrete has been affected. Barely visible in this figure is the transverse reinforcement. Damage can be estimated as severe, since the prestressed steel strands as well as the dowel bars are visible.

2.1.2 Kobe earthquake (1995)

On January 17, 1995, the southern part of Hyogo Prefecture in Japan was shaken by the Kobe (Great Hanshin) earthquake (magnitude 6.8). Among other important damage, this earthquake caused the destruction of 120 of the 150 quays in the port of Kobe (Akai 1995). Liquefaction of the soil was one important phenomenon that triggered such damage. Figure 2.2 shows the damage at a pile-deck connection in the Kobe port.

![Damage at a pile-deck connection, Kobe earthquake (1995)](source: Akai 1995)

Figure 2.2 shows steel bars as the only structural elements still joining the deck to the pile, since concrete has completely spalled off. This illustrates the relative importance of elements such as steel dowel bars to explain the structural behavior, and control damage, at pile-deck connections.
2.1.3 Haiti earthquake (2010)

The Haiti earthquake of 12 January 2010 also caused important damage in pile-deck connections. Liquefaction-induced lateral spreading was a significant factor contributing to the extensive damage at the Port Port-au-Prince (see Figure 2.3).

![Damage at Port Port-au-Prince, Haiti earthquake (2010)](source: Eberhard et al. 2010)

As can be seen in Figure 2.3, the main dock, a pile-supported marginal wharf, collapsed. Part of the pier, at the bottom of Figure 2.3, collapsed too. This is a pile-supported structure that was originally 380 m (1250 ft.) in length and 18 m (60 ft.) in width (Eberhard et al. 2010). A pedestrian bridge, also pile-supported, which connected the three dolphins, at the left bottom of
Figure 2.3, with the pier collapsed too. U.S. Navy divers found that approximately 40% of the “surviving” piles in the pier were broken, 45% were moderately damaged, and 15% were slightly damaged, see Figure 2.4:

![Figure 2.4 – Schematic of the damage in the surviving piles in the pier at Port Port-au-Prince due to Haiti earthquake (2010) (source: U.S. Navy 2010)](image)

The piles supporting the pier were approximately 510-mm square (20 in.) concrete piles on 4.3 to 4.9-m (14 to 16-ft) centers and included both vertical and battered piles (Eberhard et al. 2010). Figure 2.5 shows the typical damage at a pile-deck connection.

![Figure 2.5 – Damage at a pile-deck connection, Haiti earthquake (2010) (source: U.S. Navy 2010)](image)
The damage observed in Figure 2.5 is similar to that observed in previous figures. The concrete has completely spalled off, and only steel bars are still joining the deck to the pile. Note that here again this type of severe damage was concentrated precisely at the pile-deck connection.

### 2.2 Pile-deck connection experimental research

Pioneering work on pile-deck connections was performed at the University of Canterbury, New Zealand (Joen et al. 1988). Even though their work consisted of six pile to pile cap connections, this dissertation is focused only on their specimen PC6 (see Table 2.1 for additional details about the specimen characteristics) since it is similar to the common shallow embedment pile-deck connections currently used on the West Coast of the United States.

#### Table 2.1- Specimen details (Joen et al. 1988) (1 kip=4.45 kN)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Connection Steel</th>
<th>Axial Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PC1</td>
<td>None</td>
<td>259.2</td>
</tr>
<tr>
<td>PC2</td>
<td>None</td>
<td>259.2</td>
</tr>
<tr>
<td>PC3</td>
<td>Extended Strand</td>
<td>333.8</td>
</tr>
<tr>
<td>PC4</td>
<td>Ext.Strand, Ext.Mild Steel</td>
<td>306.4</td>
</tr>
<tr>
<td>PC5</td>
<td>Extended Strand with “Olive”</td>
<td>317</td>
</tr>
<tr>
<td>PC6</td>
<td>Inwardly Bent Dowels</td>
<td>299.2</td>
</tr>
</tbody>
</table>

Their specimen PC6 was made by a 400 mm (15.7 in.) octagonal section having four #6 (D19) diameter dowel bars bonded to the pile by epoxy resin in 1.6 in.(40 mm) diameter holes drilled 21 in.(0.53 m) deep into the pile end and anchored in the pile cap by 90° standard hooks. Among other details, the spiral steel reinforcement within the pile next to its connection to the pile cap has 0.5 in.(13 mm) diameter at 2 in.(50 mm) pitch, and was double of that in the pile cap (see Figure 2.6). Specimen PC6 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. The researchers suggested that it would behave better with additional dowel bars. Some other
general observations included that the addition of mild longitudinal steel reinforcement within the pile adjacent to and into the joint region helped minimize damage in the pile.

Figure 2.6 - Connection detail for unit PC6 (University of Canterbury, New Zealand) (Joen et al. 1988) (1 in. = 25.4 mm)

A 1997 study conducted at the University of California San Diego (Silva et al. 1997; see Table 2.2 for details of the specimens tested) illustrated that a full-scale pile cap and Caltrans Class 70 ton pile with six #6 (D19) longitudinal steel bars embedded into the cap, W6.5 (0.288 in. or 7.3 mm diameter) spiral with pitch varying between 1 and 2.5 in.(25-65 mm) (Specimen STD1; see Figure 2.7), 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. In
fact, even though Specimen STD1 reached its theoretical flexural capacity, the failure mode type can be characterized as brittle as a result of spalling of the cover concrete in an explosive manner. In addition, minimum cracking of the pile cap was observed with reinforcement steel strains below yielding levels, indicating that the flexural or shear capacity of the pile cap were never reached.

Table 2.2 - Specimen details (Silva et al. 1997)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>STD1</td>
<td>Precast, prestressed class 70 ton pile connected to a reinforced concrete pile-cap</td>
</tr>
<tr>
<td>STD2</td>
<td>A composite steel jacket, unreinforced concrete core class 70 ton pile connected to r.c.pile-cap</td>
</tr>
<tr>
<td>STD3</td>
<td>A composite steel jacket, reinforced concrete core class 200 ton pile connected to r.c.pile-cap</td>
</tr>
</tbody>
</table>

Figure 2.7 - Lateral view of specimen STD1 (University of California at San Diego) (Silva et al. 1997) (1 in.=25.4 mm)

A different study of the 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; see Fig. 2.8).
The connection was achieved using eight #10 (D32) dowel bars with bulb ends cast into the end of the pile with 29” (0.74 m) of embedment into the joint region. Eight 26” (0.66 m) long #9 (D29) T-headed bond bars were then cast next to the dowel bars. Surrounding the bond bar assembly was a #5 (D16) spiral at 2.5” (65 mm) on center extending from the pile cutoff to the bond bar T-heads. It should be noted that, since the specimen was constructed using one monolithic pour to cast the pile and the deck section, the structural behavior was altered. The specimen acted much more like an extended pile and not a bond bar connection. The specimen performed well with minimal deterioration of resistance once the connection reached its peak load. First, the pile sustained cracking that developed later into minimal spalling. Eventually a fully plastic hinge formed, extending from the pile-deck interface 12 in. (305 mm) up the pile. The hysteretic cycles showed minimal deterioration in lateral load resistance (partially due to no axial load and thus no P-Δ effects), and minimal pinching. The researchers 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. 2002) involved testing several pile-wharf connections and details with shallow embedment, indicating that such connections could sustain significant damage under reversing lateral loads (see Table 2.3 for additional details about the specimen characteristics).

Table 2.3 - Specimens tested at the University of Washington (Roeder et al. 2002)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Pile Type</th>
<th>Steel Configuration</th>
<th>Axial load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Cast-In-Place</td>
<td>Outwardly Bent Dowel</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>Cast-In-Place</td>
<td>Outwardly Bent Dowel</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>Precast, Prestressed</td>
<td>Outwardly Bent Dowel</td>
<td>0</td>
</tr>
<tr>
<td>4</td>
<td>Precast, Prestressed</td>
<td>Outwardly Bent Dowel</td>
<td>222</td>
</tr>
<tr>
<td>5</td>
<td>Precast, Prestressed</td>
<td>Inwardly Bent Dowel</td>
<td>222</td>
</tr>
<tr>
<td><strong>6</strong></td>
<td><strong>Precast, Prestressed</strong></td>
<td><strong>T-headed Bar</strong></td>
<td><strong>222</strong></td>
</tr>
<tr>
<td>7</td>
<td>Precast, Prestressed</td>
<td>Bond Bar</td>
<td>222</td>
</tr>
<tr>
<td>8</td>
<td>Precast, Prestressed</td>
<td>Outwardly Bent Dowel</td>
<td>222</td>
</tr>
</tbody>
</table>

The study highlighted differences between extended pile connections (Specimens 1 and 2) and embedded dowel connections (Specimens 3, 4, 5, 6, 7 and 8). The extended pile connection is more flexible in the elastic regime, and once yielding occurs there is very little deterioration in resistance, with it almost acting in an elastic-perfectly-plastic mode. This is in stark contrast to the embedded dowel connection, which is much stiffer and stronger but experiences bigger deterioration in resistance. The final observation from the test results is that the presence of axial load increased the maximum strength of the connection but caused significantly more deterioration at higher connection rotation.
The Specimen 6 reinforcement detail shown in Figure 2.9 is especially important since it has been used in several port facilities as a means of improving the economy and constructability of those structures. The most significant characteristics of this specimen are the use of a prestressed pile, and T-headed dowel bars. The specimen was subjected to cyclic loading (under a constant axial load). The specimen suffered cover spalling, but it reached without problem its flexural capacity. The concrete deck behaved in the elastic range.

In another experimental and analytical study sponsored by POLA, two full-scale pile-deck connections, with 24 in. (0.61 m) octagonal sections, were tested under cyclic lateral loading (Restrepo et al. 2007). One test represented the pile-deck connection of so-called “nonseismic” piles and the other that of “seismic” piles.

In the “nonseismic” case, lateral load was applied with two 220 kip (980 kN) capacity
hydraulic actuators. Two additional 115 kip (512 kN) capacity actuators were placed at the plane of inflection of the deck on each side of the pile for a total of four actuators (see Figure 2.10 for the experimental setup).

Figure 2.10 - Experimental setup UCSD: non-seismic pile (Restrepo et al. 2007)

A different experimental setup was used for the “seismic” case. Lateral load was applied with one 220 kip (980 kN) capacity hydraulic actuator. Two vertical 165 kip (734 kN) capacity actuators were used for equilibrium and to ensure the deck would remain horizontal throughout the test (see Figure 2.11 for the experimental setup). Note that push cycles will result in compression and pull cycles in tension. An additional axial load of 100 kip (445 kN) was applied to the pile of this specimen in the third of the three cycles to a given amplitude.

Figure 2.11 - Experimental setup UCSD: seismic pile (Restrepo et al. 2007)
The “nonseismic” pile differs from the “seismic” pile in a number of details including: number of dowel bars, length of dowel embedment, spiral reinforcement ratio, and use of bond bars. (Additional details are given in Chapter 3). Both specimens showed that shallow pile 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 (Port of Los Angeles 2004) can accurately be predicted.

In 2008, the University of California-San Diego published testing on two additional full-scale pile-deck connections (Bell 2008). The general research objective was to evaluate the seismic performance at Berth 145 in the Port of Los Angeles. Specimen “A.1” was a shallow embedment connection between a prestressed concrete pile and a cast-in-situ deck. The pile was embedded 2 in. (50 mm) into the deck. The pile and the deck were connected with eight #9 (D29) L-shaped dowels, with outward bends, and 5 feet (1.50 m) embedment into the pile. The pile transversal reinforcement consisted of an A82 W20 (0.50 in/ 12.8 mm diameter) spiral, with 2.5\ in. (65 mm) pitch. Pile prestressing was achieved through 16 strands, 0.6-in. (15 mm) in diameter. No axial load was applied. The experimental set-up was similar to the previously described “seismic” case: the pile was supported using a pin connection while the deck was supported by a vertical actuator. The lateral load was applied using a horizontal actuator. This specimen had satisfactory performance with minimal pile and deck spalling. Near the end of the test, several dowel bars fractured.

Even more recently, four full-scale specimens (Specimens 9, 10, 11, and 12) were tested at the University of Washington to evaluate and compare the performance of different connection details (Jellin 2008).
All these specimens are quite similar. For instance, they have T-headed dowel bars embedded 59 in. (1.50 m) into the pile, and W11 (0.374 in. or 9.5 mm diameter) spirals with pitch varying between 1 and 3 in. (25 to 75 mm), but they differ in 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 (see Figure 2.12 - Connection details (University of Washington) (Jellin 2008)).
All specimens described above experienced significant physical damage, with pile and deck cover spalling and exposure of the spirals and longitudinal reinforcement of the pile occurring at “drifts” between 2% and 5%, even if their flexural capacities were acceptable. Specimens 9 and 11 will be described in more detail in Chapter 3.

Four additional full-scale specimens were tested at the University of Washington (Stringer 2010), aiming to improve pile-deck connections for wharf structures. The specimens and experimental set-up were similar to the previous tests (Specimens 9 through 12), but this time the influence on cyclic response by the variation of axial load on the pile, bearing pad configuration, and bearing pad material, was studied. Specimen 13 had a full cotton duck bearing pad, and 450 kip (2000 kN) axial load. It was considered the reference or control specimen. Specimen 14 had a full cotton duck bearing pad too, but 900 kip (4000 kN) axial load. Specimen 15 had an annular cotton duck bearing pad, and 450 kip (2000 kN) axial load. Finally, Specimen 16 had an annular Fiberlast bearing pad, and 450 kip (2000 kN) axial load. All these specimens showed higher rotations at the pile-deck connection due to the flexibility of the pads. Based on the visual damage observed, it was concluded that the core of the pile, as well as the deck, never experienced spalling. The pads significantly delayed pile spalling, but rapid strength loss ensued after initial spalling. As a final point, higher axial loads causes earlier onset of pile spalling and significantly more rapid strength degradation. Based on the measured response, it was concluded that the pad connections displaced in more or less rigid body rotation.

Finally, an important project for seismic risk management of port systems has been underway (the NEESR Grand Challenge Project). As described on its web page www.neesgr.gatech.edu, see Figure 2.13, the three most important tasks of this project (which includes the work to test Specimens 9-16, as described above, and additional testing and analysis
here at the University of Illinois) are broadly focused on: predicting the seismic response and resulting damage states of key port components via large-scale experimentation and numerical simulation, estimating the effects of damage to these components, and mitigating possible losses.

![NEESR Grand Challenge Project](https://www.neesgc.gatech.edu/content/view/49/1/)

**Figure 2.13 – The NEESR Grand Challenge Project for seismic risk management of port systems**

(source: web page [http://www.neesgc.gatech.edu/content/view/49/1/](http://www.neesgc.gatech.edu/content/view/49/1/))

The University of Illinois tested a pile-deck connection very similar to University of Washington Specimen 9 (Foltz 2011). However, it differs from Specimen 9 in the manner that loads and displacements were applied. In fact, it can be considered the first large-scale pile-wharf connection test with realistic load and boundary conditions. The test results showed that the primary rotation source at the pile-deck connection was lumped rotation as opposed to rotation due to distributed flexural action of the pile. At lower loads, flexural rotation of the pile already accounts for only 30% of the total rotation of the pile. However, if the confined concrete core remains intact, the pile-wharf connection can accommodate large lateral deformation with only a modest reduction in capacity. A reduced axial load (0.02 \( f'_c A_g \)) relative to the other experimental
tests caused the pile-deck connection to behave similar to tests with bearing pads located at the end of the pile, with deformation localizing in the connection.

It can be noted that all this experimental work, and additional analytical research, has not yet crystallized into any national standard for the design of pile-wharf connections. The American Society of Civil Engineers (ASCE) has been working on this since 2005 through its Coast, Oceans, Ports, and Rivers Institute (COPRI) committee. Currently, the seismic design of pile-deck connections is performed based on several design guidelines, including Title 24, California Code of Regulations, Chapter 31F, informally known as the Marine Oil Terminal Engineering and Maintenance Standards (MOTEEMS 2007), whose antecedents can be traced to guidelines such as the Port of Long Beach (POLB 2009) Wharf Design Criteria, the Port of Los Angeles (POLA 2004) Container Terminal Seismic Code, and the Permanent International Association for Navigation Congresses (PIANC 2001) Seismic Design Guides for Port Structures. POLA and POLB wharf design guidelines have been used in these ports for at least 10 years. All these standards recognize the importance to accurately estimate damage, and to be able to control it when providing the required displacements at different performance levels. However, and as an example of still needed improvement, they use limit strains as a measure of damage, which can be cumbersome for the practitioner.

2.3 Pile-deck connection analytical research

As explained in Chapter 1, the development of pile-deck connection analytical models is not as extensive as the related experimental research. In fact, even though the experimental research described in Section 2.2 has clearly shown the importance of the non-linear behavior of the pile-deck connection, the pile is, in many cases, simply assumed as elastic, since much more attention is paid to the nonlinear behavior of the soil around it.
Even in the case that the non-linear behavior of the pile-deck connection is taken into account, the usual analytical model is that developed for frame structures, in other words, concentrated plastic hinges at the zones of maximum moments. According to Priestley (Priestley et al, 1998), this type of model is obtained adding to the otherwise elastic frame structure rotational springs created on the basis of moment-curvature curves of a pile section at the pile-deck connection. The curvature values are multiplied by a fixed length, the plastic hinge length, to obtain rotations, and therefore the moment-rotation curve employed in the rotational spring.

The plastic hinge length is an empirical value that approximates the actual nonlinear behavior in damaged zones. Different equations have been developed to calculate this plastic hinge length, all of them limited to the available experimental data. One of these equations (see Priestley et al. 2005) is presented in Chapter 4, regarding the calculation of the Kunnath damage index.

Note that the usual plastic hinge length equations cannot represent well the observed structural behavior at the pile-deck connection, since the rotation contribution of the slip of the rebars is much more important compared to other parameters commonly used in these equations. Therefore, some specific procedures have been developed to calculate moment-rotation curves for concentrated plastic hinges at the pile-deck connection. The most simple of them is that reported by Goel (2008-a). In this model the moments and corresponding rotations are calculated following an iterative procedure based on simple statics, see Figure 2.14:
This process begins by selecting a value of strain in the outermost dowel on the tension side (for instance, the yield strain), and establishing the location of the neutral axis of the section. Then, the calculated steel strain at the outermost dowel is multiplied by an estimate of the slip of the rebars equal to 15% of the product between the dowel stress and the dowel area. Note that
other alternative estimates are also possible. In this way, the elongation of the outermost dowel is calculated. Next, the rotation can be calculated as the elongation of the outermost dowel to the distance between the outermost dowel and the neutral axis. The total moment is also calculated as the summation of moments at center of the pile due to tensile and compressive forces. Repeating the previous steps, with different initial steel strains, will develop the entire moment-rotation curve.

Other alternatives have been also proposed to estimate the additional rotation due to the bar slip at the pile-deck connection. These alternatives use relatively sophisticated analytical models such as zero-length fiber based rotational springs. One typical example is the model presented by Zhao and Sritharan (2007) for the slip of the rebars. It is included in the OpenSees software as the uniaxial material BOND_SP01. Figure 2.15 shows its characteristics:

![Figure 2.15 – OpenSees uniaxial material BOND_SP01 (adapted from Mazzoni et al. 2009)](image)

In Figure 2.15, $F_y$ is the yield strength of the reinforcing steel, $F_u$ is the ultimate strength of the reinforcing steel, $S_y$ is the rebar slip under yield stress, $S_u$ is the rebar slip at the bar fracture
strength, $K$ is the initial slope of the curve slip vs. stress, and $bK$ is the initial slope at the steel hardening zone. This model has been developed based on experimental results and therefore lacks a theoretical background. It was initially used in the analytical model developed in this dissertation. However, it presented convergence problems just after the beginning of the nonlinear zone of structural behavior. For this reason, no further analysis was done with this type of model, see Figure 2.16.

![Figure 2.16 – Force vs. displacement curves](image)

(Experimental and analytical using the BOND_SP01 model curves)

Shafieezadeh et al. (2012) present the following model for the pile-deck connection, see Figure 2.17:
In this model, the deck is modeled as a rigid body, as well as the link element between the centroid of the deck and the tip of the pile. The embedded portion of the pile uses a calibrated confined concrete without prestressed strands. This is the same calibrated concrete used in the plastic hinge zone, which has 1.5 m (59 in.) length. The concrete properties, specifically the concrete crushing strain, are changed to calibrate the force-deformation response of the numerical model with the corresponding response from experiments. The material properties of the prestressed strands and longitudinal reinforcement are not calibrated and are the commonly used in this type of structures. A similar approach is used by Yang et al. (2012). They used a nonlinear beam-column element between the precast pile element and wharf deck elements, which is also calibrated in terms of material strengths to match the experimental data.

Shafieezadeh et al. model (2012) was developed to study the interaction soil-wharf under
extreme phenomena such as liquefaction. The predicted wharf damage patterns apparently are similar to those observed for example at the Port of Kobe during the Kobe earthquake (1995), where large deformations occurred in piles close to the wharf deck and at the interface of liquefied and nonliquefied soil layers (Matsui and Oda 1996). However, and even though this approach can approximate the mean global behavior of wharf structures, it cannot give an accurate and detailed behavior of local zones such as the pile-deck connection. Since damage is concentrated in this critical zone, there is a need to add more detail to the analysis of this zone.

2.4 Code equations for spiral reinforcement

Focus will be given to the PCI equations (PCI 2004) because they are commonly used in prestressed concrete pile design. However, current recommendations from the Canadian (CSA A23.03-04) and New Zealand (NZS 3101-06) codes are also compared. They were basically developed for the building and bridge construction industry, but can be extrapolated to port structures. The Permanent International Association for Navigation Congresses (PIANC 2001), which is directly related to the port construction industry, presented an equation to calculate the volumetric ratio of transverse reinforcement too. The International Building Code (International Code Council 2011) also has equations to be applied for piles. In addition, empirical recommendations (Gerwick 1993) would give a better idea of the range of values usually used for spiral reinforcement.

All of these equations calculate the volumetric ratio of the spiral reinforcement, which is defined as follows:

\[ \rho_s = \frac{4A_{st}}{sD_s} \] (2.1)

Where \( \rho_s \) is the volumetric ratio of spiral reinforcement, \( A_{st} \) is cross-sectional area of one spiral, \( s \) is spiral pitch, and \( D_s \) is diameter of the pile measured to the outside of the spiral.
2.4.1 PCI equations

Following PCI, the minimum volumetric ratio of circular spiral or hoop reinforcement for high risk seismic zones $\rho_s$ should not be less than:

\[
\rho_s = 0.45 \frac{f_c'}{f_{ys}} \left( \frac{A_g}{A_{ch}} - 1 \right) \left( 0.5 + 1.4 \frac{P + f_{pc}A_g}{f_c' A_g} \right) \quad \text{(PCI Eq. 20.5.4.2.5.2-1)} \tag{2.2}
\]

\[
\rho_s = 0.12 \frac{f_c'}{f_{ys}} \left( 0.5 + 1.4 \frac{P + f_{pc}A_g}{f_c' A_g} \right) \quad \text{(PCI Eq. 20.5.4.2.5.2-2)} \tag{2.3}
\]

\[
\rho_s = 0.007 \quad \text{(PCI Eq. 20.5.4.2.5.2-3)} \tag{2.4}
\]

The concrete strength is normalized using the ratio $\frac{f_c'}{f_{ys}}$, where $f_c'$ is strength of concrete, and $f_{ys}$ is yield stress of the transverse reinforcement. The effect of the concrete cover is expressed using the term $\frac{A_g}{A_{ch}} - 1$, where $A_g$ is gross area of the concrete section, and $A_{ch}$ is area of the concrete core measured to outside of peripheral transverse reinforcement. Finally, the effect of the axial load is calculated using the term $\frac{P + f_{pc}A_g}{f_c' A_g}$, where $P$ is design axial force, including seismic load, and $f_{pc}$ is prestress in the concrete section.

The volumetric ratio of circular spiral or hoop reinforcement for high risk seismic zones $\rho_s$ is defined as follows:

PCI Eq. 20.5.4.2.5.2-1 uses the same terms as ACI-318 equation (10.5) and includes an additional term for the effect of the axial loading and prestress. ACI Eq. (10.5) is as follows:

\[
\rho_s = 0.45 \left( \frac{A_g}{A_{ch}} - 1 \right) \frac{f_c'}{f_{ys}} \quad \text{(ACI 10.5)} \tag{2.5}
\]

This equation is intended to provide additional load carrying strength for concentrically...
loaded columns equal to or slightly greater than the strength lost when the shell spalls off. This equation has no relation with deformation parameters, but ensures sufficient transverse reinforcement at low axial loads. PCI Eq. 20.5.4.2.5.2-2 was based on the curvature ductility ratio at failure. Failure was defined as a 20% reduction in lateral load resistance or when concrete and steel reach strain limits similar to those at moderate/severe damage level. PCI Eq. 20.5.4.2.5.2-3 is the minimum amount along the length of the pile that ensures good behavior during pile driving.

### 2.4.2 CSA A23.03-04 equations

The Canadian code CSA A23.03-04 uses the following equations:

\[
\rho_s = 0.45 \left( \frac{A_g}{A_{ch}} - 1.0 \right) \frac{f_c'}{f_{yt}} \tag{2.6}
\]

\[
\geq 0.4 k_p \frac{f_c'}{f_{yt}} \tag{2.7}
\]

\[
k_p = \frac{P}{P_0}; \quad P_0 = 0.7 \left( A_g - A_{sl} \right) f_c' + A_{sl} f_{yl} \tag{2.8(a), 2.8(b)}
\]

Where \( \frac{A_g}{A_{ch}} \) is the ratio area of the section to area of the core, \( \frac{f_c'}{f_{yt}} \) is the ratio of concrete strength to yield strength of the transverse reinforcement, \( P \) is axial loading, \( A_{sl} \) is area of non-prestressed reinforcement, and \( f_{yl} \) is yield strength of longitudinal non-prestressed reinforcement. Note that the Canadian equations for the volumetric ratio of the spiral reinforcement use similar parameters than those used in the PCI equations.

### 2.4.3 NZS 3101-06 equations

The New Zealand code NZS 3101-06 uses the following equations:

\[
\rho_s = \frac{(1.3 - \rho_t m)}{2.4} \frac{A_g}{A_{ch}} \frac{f_c'}{f_{yt}} \frac{P}{0.85 f_c' A_g} - 0.0084 \tag{2.9}
\]

\[
\frac{A_g}{A_{ch}} \geq 1.2; \quad \rho_t m \leq 0.4; \quad m = \frac{f_{yl}}{0.85 f_c'} \tag{2.10(a), 2.10(b), 2.10(c)}
\]
Where some of the terms have already been defined, $A_{st}$ is total area of longitudinal reinforcement, $\rho_l$ is longitudinal reinforcement ratio, $d''$ is diameter of concrete core of circular column measured to the outside of spiral, and $d_b$ is nominal diameter of non-prestressed bar. Note that earlier New Zealand equations were used as reference by PCI to develop their own volumetric ratio of the spiral reinforcement equations.

### 2.4.4 PIANC equation

PIANC (2001), which is an international technical non-political and non-profit making association constituted in accordance with and governed by Belgian law and sponsored by national, federal and regional governments or their representative bodies, uses the following equation:

$$\rho_s = \frac{f_{ce}'}{f_{ye}} \left(0.5 + \frac{1.25(P + f_{pc}A_g)}{f_{ce}A_g}ight) + 0.13(\rho_l - 0.01)$$ (2.12)

Where $f_{ce}'$ is expected concrete compression strength ($= 1.3f'_{ce}$), and $f_{ye}$ is expected reinforcement yield strength ($= 1.1f_{yl}$). All other terms have been defined previously. Note that PIANC uses higher values for the concrete and steel strengths instead of the usual conservative low values. This is a performance based design characteristic, since the use of usual nominal material strengths and strength reduction factors in design or assessment will place a demand for corresponding increases in the required strength of capacity protected structural elements. This will have an adverse economic impact on the design of new structures and may result in an unwarranted negative assessment of existing structures (PIANC 2001). PIANC equation, which is based on ATC-32, includes the longitudinal reinforcement as an independent term.

According to the performance based design approach, equations such as that presented by
PIANC can be used as an initial estimate of the spiral reinforcement amount needed to obtain an acceptable structural behavior at the pile-deck connection. However, it should be additionally checked, under different earthquake hazards, that critical strain material limits are not overcome. For instance, a summary of the strain limits for both concrete and reinforcing steel by the Port of Los Angeles seismic code (POLA 2004) is presented in Table 2.4 for two levels of earthquake hazard, the so called Operational Level Earthquake (OLE) and Contingency Level Earthquake (CLE). Currently performance based design is fundamental for damage estimation and will be explained in more detail in Chapter 4.

Table 2.4 - Strain limit states in the POLA seismic code

<table>
<thead>
<tr>
<th>Strain Limit-States</th>
<th>OLE</th>
<th>CLE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pile-Deck connection:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete Compression Strain</td>
<td>≤ 0.005</td>
<td>≤ 0.020</td>
</tr>
<tr>
<td>Dowel Reinforcement Tension Strain</td>
<td>≤ 0.010</td>
<td>≤ 0.050 ≤ 0.6 $e_{smd}$</td>
</tr>
</tbody>
</table>

$e_{smd} = \text{strain at peak stress of dowel reinforcement}$

2.4.5 IBC equations

International Building Code (IBC 2012) uses the following equations:

\[
\rho_s = 0.25 \frac{f'c}{f_{yt}} \left( \frac{A_g}{A_{ch}} - 1.0 \right) \left( 0.5 + 1.4 \frac{p}{f'c A_g} \right) \quad \text{(IBC Eq. 18-6)}
\]

(2.13)

\[
\geq 0.12 \frac{f'c}{f_{yt}} \left( 0.5 + 1.4 \frac{p}{f'c A_g} \right) \quad \text{(IBC Eq. 18-7)}
\]

(2.14)

\[
\leq 0.021 \quad \text{(IBC Eq. 18-8)}
\]

(2.15)

All of the terms used in the IBC equations have been defined previously. IBC Eq. 18-6 is similar to PCI Eq. 20.5.4.2.5.2-1. The only differences are the factor 0.25, which is intended to be less conservative than the corresponding factor 0.45 in the PCI equation, and the axial load due to
prestress, which is not taken into account in the IBC equation. IBC Eq. 18-7 is identical to PCI Eq. 20.5.4.2.5.2-2, except that the IBC equation does not consider the axial load due to prestress. IBC Eq. 18-8 is an upper limit, the corresponding PCI Eq. 20.5.4.2.5.2-3 is a lower limit.

2.4.6 Empirical equations

Values commonly used in the pile construction industry and which have shown relatively good structural behavior could give a guide of the spiral reinforcement needed at the pile-deck connection. For example, the volumetric ratio of the transverse reinforcement recommended in practical use by Gerwick (1993) varies between 0.015 and 0.025.

2.5 Comparison among spiral reinforcement code equations

In Figure 2.18, spiral volumetric ratio results using PCI (PCI 2004), IBC (International Building Code 2012), New Zealand (NZS3101 2006), Canada (CSA A23.3-04) and PIANC standards or guidelines, as well as recommended empirical values, are represented if axial load is varied. These results show significant differences among different codes and empirical values.
In Figure 2.18, the following values were used: concrete strength = 55 MPa (8 ksi), spiral yield stress = 490 MPa (71 ksi), concrete cover = 64 mm (2.5 in.), initial prestress in the concrete section = 9.7 MPa (1.4 ksi), ratio between the area of longitudinal reinforcement to the area of the concrete section = 0.022. These are values similar to those of the University of Washington specimens (Jellin 2008). Note that this figure illustrates the significant differences in the amount of spiral reinforcement calculated using different codes. Clearly, the parameters that affect the spiral reinforcement ratio and their relationships are still not completely understood. The work presented in this dissertation will contribute to better understand these parameters and their interactions.
CHAPTER 3: SECTIONAL AND MEMBER BEHAVIOR

This chapter deals with a description of the analytical model using the fiber approach for the pile-deck connection with shallow embedment. In particular, the geometry of the section, the type of pile elements used, and the material constitutive models that are employed in it, namely concrete, dowel bars and prestressing strands are described in details.

Experimental results of five test specimens from the literature have been used in this study to validate and calibrate the analytical modeling approach adopted for pile-deck connections. These specimens include typical pile-wharf connections of the type that have been built in the last 10-15 years in the Port of Los Angeles, Port of Long Beach, and/or Port of Oakland, as well as specimens with potential improvements to current design practices. Two of the specimens (UW-1 and UW-2) were tested at the University of Washington (Jellin 2008, where they were referred to as Specimen #9 and #11 in Chapter 2), while other two (SD-1 and SD-2) were tested at the University of California, San Diego (Krier 2006, Restrepo et al. 2007). The fifth specimen (UI specimen) was tested at the University of Illinois (Foltz 2011). All five specimens were at full scale, comprising representative cases (including some important differences) of pile-deck connection seismic and non-seismic detailing and loading.

3.1 Description of the specimens

3.1.1 UW specimens

Specimen UW-1 is taken as an initial reference for modeling, since it represents the common baseline type of shallow embedment pile-deck connection studied in this dissertation. In this specimen, the precast-prestressed concrete pile had an octagonal cross-section of 0.6 m (24 in.) across and a length of 2.6 m (103 in.) from the connection interface to the point of simple (zero-moment) lateral load application. The cast-in-place deck was represented by a rectangular
reinforced concrete block that was 2.3 m (92.5 in.) long, 1.3 m (52 in.) wide and 0.7 m (29 in.) thick.

Figure 3.1 presents connection details of Specimen UW-1, as well as of Specimen UW-2, only with the piles shown over the deck since this is the experimental setup position used for testing (even though the specimens were cast upright).

The concrete used in these piles had 55 MPa (8000 psi) design (28-day) compressive strength. Eight T-headed bars were grouted into ducts within the pile and cast into the deck to create a moment connection. These ASTM A706 bars had a diameter of 32 mm (#10), specified minimum yield strength of 420 MPa (60 ksi), and total length of 1.9 m (76 in.). The pile also had spiral reinforcement of 10 mm diameter (W11) wire, at a center-to-center pitch varying from 25 to 75 mm (1 to 3 in.). Prestressing was achieved with twenty-two 13 mm (0.5 in.) diameter 1860 MPa (270 ksi) low-relaxation strands, stressed to 138 kN/strand (31 kips/strand).

Connection details in Specimen UW-2 (see Figure 3.1) were almost identical to those in Specimen UW-1, except for: a) unbonding the T-headed connection steel for 0.4 m (15 in.),
centered on the end of the pile, and b) adding a 20 mm (0.75 in.) thick “cotton duck” bearing pad between the precast concrete pile end and the reinforced concrete deck. Unbonding the longitudinal steel caused less concentration of bar strain and therefore reduced the tendency for concentrated dowel bar and connection damage during large connection rotations. The cotton duck bearing pad was specifically used to reduce edge stresses, and delay pile spalling. The type of pad used is referred to as Sorbtex (Voss Engineering, Lincolnwood, IL), a layered material created with cotton-polyester fabric duck impregnated with oil resistant synthetic rubber. It was specified as having a hardness of 90±5 (measured using the Shore A scale for “softer” rubbers) and a minimum ultimate compressive strength of 69 MPa (10,000 psi). Other material properties of Specimen UW-2 were similar to those of Specimen UW-1, with the exception of the concrete strength in the deck. Specimen UW-2 had an actual deck compressive strength at testing of only 47 MPa (6.8 ksi), in contrast with the Specimen UW-1 deck concrete that had a 72 MPa (10.5 ksi) test-day compressive strength. Both piles were subjected to a constant axial load of 2000 kN (450 kips) throughout testing (approximately 10% of the product of design $f_c'$ and $A_g$), and then to a lateral cyclic loading history up to a drift ratio of 9%.

3.1.2 SD specimens

In an experimental and analytical study sponsored by the Port of Los Angeles (POLA), two full-scale structural concrete pile-deck connections with 0.6 m (24 in.) octagonal pile sections were tested under cyclic lateral loading (Krier 2006, Restrepo et al. 2007). The “nonseismic” specimen (SD-1) represented an interior connection in which the 0.61 m. (24 inches thick) deck frames in to both the left and right of the pile centerline (from neighboring piles in each direction). In contrast, the “seismic” specimen (SD-2) was an exterior connection in which the 0.91 m (36 inches thick) deck only has moment-resisting frame continuity to one side
of the pile. For these SD specimens, the experimental setup was a bit different than that for the UW specimens, which among other things resulted in the deck being over the pile (upright) during testing.

The SD-1 pile specimen (see Figure 3.2 for more details) had 57 MPa (8.3 ksi) compressive strength concrete at test-day, was prestressed with sixteen 16 mm (0.6 in.) diameter low-relaxation strands, and had an A82 (W11) spiral spaced 75 mm (3 in.) center-to-center. Longitudinal pile reinforcement at the connection consisted of four 28 mm (#9) dowel bars of ASTM A706 steel, with yield stress $F_y = 455$ MPa (66 ksi), ultimate stress $F_u = 635$ MPa (92 ksi), and ultimate strain $\varepsilon_{su} = 12.3\%$, grouted 1.5 m (5 ft.) into the pile and anchored 0.7 m (29 in.) into the deck.

The SD-2 pile specimen had an $f'_c$ of 56 MPa (8.1 ksi), was prestressed in the same fashion as SD-1 and had an A82 (W20) spiral spaced 65 mm (2.5 in.) center-to-center. This pile was cut to expose 1-½ turns of the smooth wire spiral that was then spliced with another length of A82 (W20) spiral that wrapped the dowel bars into the deck. In this case, the longitudinal pile reinforcement consisted of eight 32 mm (#10) dowel bars of ASTM A706 steel with $F_y = 475$ MPa (69 ksi), $F_u = 665$ MPa (97 ksi), and $\varepsilon_{su} = 11.2\%$, similarly grouted into the pile and embedded into the deck as was done in SD-1.

Pile SD-1 was subjected to an axial load representative of self-weight, whereas pile SD-2 had varying axial load. Both specimens were subjected to lateral cyclic loading up to a displacement ductility of 18 (Krier 2006).

The loading set-up for all of these experimental specimens was based in part upon test protocols developed in prior research (ATC 24 1992, Roeder et al. 2002, Sritharan et al. 1998); cyclic tests like these are usually considered to represent an envelope of actual seismic behavior.
The use of isolated subassemblies should still be able to catch the general trends in marginal wharf connection behavior, but it is clearly a bit of a simplification with respect to actual continuous port structures, so things such as variable points of inflection have been neglected in this approach – in any event, the analytical loading patterns used in this work are identical to those of the tests.

**Figure 3.2 - Geometric characteristics of “Nonseismic” and “Seismic” pile-deck connections**

(adapted from Krier 2006)
3.1.3 UI specimen

The UI specimen (Foltz 2011) consisted of a 99-in. (2.5 m) in length and 24-in. (610 mm) in diameter, octagonal, precast, prestressed pile connected to a cast-in-place reinforced concrete deck slab. The concrete pile was embedded 2 in. (50mm) into the cast-in-place deck, and the connection was achieved by using eight #10 (D32) T-headed dowel bars grouted into the pile. The pile was reinforced with twenty-two 0.50 in. (12.5 mm) diameter, 270 ksi (1860 MPa) low-relaxation strands, with each strand prestressed to 31 kips (138 kN). Spiral reinforcement was W11 (0.374 in. or 9.5 mm diameter) smooth wire. Spiral pitch varied between 1 in. (25 mm) at the end of the pile, to 3 in. (75 mm) along the middle of the pile, see Figure 3.3 a). The test specimen was heavily instrumented with traditional instrumentation such as strain gages, linear variable displacement transducers (LVDTs), string potentiometers, and inclinometers. An optical coordinate measuring machine (Krypton K600), which uses 3 linear charge-coupled device cameras to triangulate light emitting diodes (LEDs) in order to locate their position in space, was also used. A grid of 132 LEDs in total was applied to the specimen. A “Loading and Boundary Condition Box” (LBCB) was used to control all 6 degrees of freedom at the “non-connection” end of the pile, see Figure 3.3 b). The LBCB introduced displacements and rotations obtained from related 2D analytical models of the wharf structure. Three records provided by the SAC project for the Los Angeles area were used to produce the analytical displacements and rotations applied at the “non-connection” end of the pile. The Imperial Valley record (LA44) was selected for the lowest hazard level, 50% probability of exceedance in 50 years; Northridge (LA18) was selected for the 10% probability in 50 years return period; and Kobe (LA2) was selected for the 2% probability in 50 years return period.
a) Pile reinforcement and other details of the UI specimen (source: Foltz 2011)
b) The “Loading and Boundary Condition Box” (LBCB) position on top of the UI specimen

Figure 3.3 – Structural details and LBCB of the UI specimen

3.2 Fiber based modeling approach

One of the fundamental objectives of this work is to develop optimal analytical models for pile-deck connections. Evidently, continuum finite element models will lead to high accuracy results; however, they can be quite computationally intensive. Consequently, the present work attempts to advance relatively more simplified modeling approaches by retaining the simplicity of frame-based macro-modeling approaches while avoiding the complexity of continuum-based micro-modeling (and still capturing key nonlinear aspects of the structural behavior). The fiber-section modeling technique is in fact currently one of the most powerful and efficient ways to analyze prestressed and reinforced concrete elements with plastic behavior, and it will be used to model the pile sections, including at the connection. In this technique, the section force-deformation relationship is derived by integration of simple uniaxial stress-strain material relations across all the fibers. Obviously the accuracy of this method will therefore depend on the
reliability of the material stress-strain relations used.

The software used to implement this technique is OpenSees (Mazzoni et al. 2009), due to its flexible fiber approach and extensive material library that includes many nonlinear material models that have been developed specifically for seismic applications. It is worth noting the excellent alternatives that OpenSees offers for the analysis of structural elements using the fiber approach. These alternatives are briefly explained in the following paragraphs since they describe the current state-of-the-art in material modeling.

3.3 Concrete

Modeling the cyclic stress-strain behavior of concrete is fundamental to the successful computer simulation of a pile-deck connection. A first concrete model classification can be related to the inclusion or not of the concrete tensile strength. A typical model that does not include the concrete tension strength is that of Kent-Scott-Park (1971). The principal virtue of this model is its simplicity, which helps to improve the efficiency of the structural computer program. OpenSees Concrete01 is one model of this kind, see Figure 3.4.

Figure 3.4 - OpenSees Concrete01 Material (adapted from Mazzoni et al. 2009)
In this model, the stress-strain curve until peak compressive strength is parabolic, then linear thereafter until a residual strength value is reached, and then continues on as a constant horizontal line. These curves are defined by the stress and strain values at the peak (fpc, epsc0) and ultimate (fpcu, epsU) points and the initial modulus of elasticity.

Additional improvements can be added to this model, such as for instance the effect of pieces of crushed concrete in cracks that produce a smoother change in stiffness when these cracks close (Stanton and McNiven 1979). In order to maintain its computational simplicity, the hysteretic behavior of the model usually follows a degraded linear unloading/reloading stiffness according to the work of Karsan-Jirsa (1969).

On the other hand, concrete models that include tensile strength can show tension stiffening and softening due to the contribution of tensile stresses to the flexural stiffness of the member. Their use is important when the precracked and preyield response of the reinforced concrete section is of particular interest. The most simple of the tension strength models is linear (Yassin 1994).

Chang and Mander (1994) define the most desirable characteristics of the general monotonic stress-strain curve for concrete in compression as: (1) the slope at the origin is the initial modulus of elasticity $E_c$, (2) it should show a peak point in the strain-stress curve, (3) it should describe both the ascending and the descending parts of the concrete behavior, and (4) it should have control over the descending or softening branch. In fact, the equation known as Popovics’ (1973), which satisfies these requirements, has proven to be very useful in describing the monotonic compressive stress-strain curve for concrete. This model, when applied to cyclic behavior, can again be complemented with the simple but efficient work of Karsan-Jirsa.
Additional improvements to the model can add, for instance, tensile strength with exponential decay (Mazzoni et al. 2009).

Near the pile-deck connection, the core of the pile is typically confined with a relatively large amount of transverse (spiral) reinforcement. Considering the effect of such confinement on the constitutive behavior of the core concrete is crucial. The confined concrete model developed by Mander et al. (1988) is one of the most widely used. In fact, this model contains the effort done since the first experimental works about the effect of concrete confinement (Richart et al. 1928). This model, for instance, uses the monotonic stress-strain curve originally proposed by Popovics (1973). It also uses a modified version of confinement effectiveness through geometrical factors initially proposed by Sheikh and Uzumeri (1980, 1982). A constitutive model developed by William and Warnke (1975) and Shickert and Winkler (1979) to determine the confined concrete peak stress is also used by the Mander et al. model. Furthermore, Mander et al. also included their own previous work related to prediction of the longitudinal concrete compressive strain at first hoop fracture using an energy approach (Mander et al. 1984). Other detailed models, which take into account confinement effects due to different arrangements of transverse reinforcement and/or external strengthening such as steel jackets or FRP wraps, have been presented relatively recently (Braga et al. 2006).

It should also be noted that over the last few years there has been increased use of high-strength concrete and steel in construction in general, and in pile construction specifically, which has given an important impetus to the development of new models that describe its stress-strain behavior, especially for seismic prone areas where strength and ductility are both of primary importance. In fact, due to the brittle nature of high-strength concrete under axial compression, the presence of lateral reinforcement becomes even more significant than in normal-strength
concrete counterpart structures. An additional assessment of these new models for high-strength concrete highlights that the differences in the ascending branch of the stress-strain curve are moderate among them, but there are important deviations in the descending branch, which controls the toughness and ductility of concrete (Kappos and Konstantinidis 1999).

Among these studies, the most promising seems to be Legeron and Paultré’s model (2003), which is the basis of the current Canadian code. An important characteristic of this model is that it recognizes that the yield stress of ties is not fully used for confinement effects. Sharma et al. (2005) compared 8 models for high-strength concrete using 44 high-strength concrete column specimens, and recommended the model by Legeron and Paultré in order to be able to analytically predict the uniaxial response of high-strength concrete columns with a reasonable degree of accuracy. However, there are other models such as that by Kappos and Konstantinidis (1999), based on statistical analysis of 10 models applied to predict the results of 108 large-scale specimens tested under uniaxial compression, and used in structural programs such as SeismoStruct (seismosoft.com).

Note also that “classical” models, such as the Mander et al. model (1988), can give misleading results when applied to high-strength concrete and high-strength transverse steel, but it can be better adapted by modifying the factor R, defined as the ratio of increase in strain to increase in strength at peak stress due to confinement (and which, indirectly, controls the descending branch of the confined concrete stress-strain curve, see Figure 3.5). The R factor basically changes the amounts of strength descend in the confined concrete curve strain-stress. If R is small, then the strength descend is bigger. For instance, R=3 produces a faster descend compared to the other cases in Figure 3.5. Note that high strength concrete has a faster strength reduction than regular concrete, and therefore smaller R values than the standard value 5 should
be used for it. In this work, a factor R equal to 3 has been used to improve the estimation of the descending concrete branch if high-strength concrete is used.

![Stress-strain curves for confined concrete using different R factors](image)

**Figure 3.5 - Stress-strain curves for confined concrete using different R factors**

In addition to this general type of model, several authors have studied in detail crucial points of the concrete stress-strain curve. In particular, Fantilli et al. (1998, 2002), Debernardi and Taliano (2001, 2002), and Oehlers et al. (2008), have proposed a sliding frictional shear failure model to describe the concrete ultimate stress, and other authors, including Priestley et al. (1996) and Fardis (2009), have proposed equations to independently calculate the associated ultimate strain. In fact, the problem of estimating the crucial ultimate deformation at ultimate strength can be rephrased as one of lateral deformation determination at shear failure (Mostafaei et al. 2009).

The literature review of current concrete stress-strain curves is not complete without reference to its hysteretic behavior. Concrete hysteretic behavior models are based on the following experimental observations (Karsan and Jirsa 1969): the cyclic load history possesses an envelope curve, which is the monotonic curve; in addition, it possesses a locus of common points which are defined as the point where the reloading portion of any cycle crosses the
unloading portion. Stresses above the common points produce additional strains, but stresses at or below the common points do not. It was also observed that the common points are a function of the stress amplitude.

From a “computational” point of view, three basic components can be identified in the hysteretic behavior of any material or structural elements (Chang and Mander 1994): envelope, connecting and transition curves. The envelope curves are the “back-bones” of the general hysteretic behavior. Usually shifting and scaling is used to simulate degradation. Changing or shifting the return point, and not the entire curve, is also used to simulate degradation. The connecting curves are the connection between the envelope curves. More than one equation is normally used to represent this kind of curve. Finally, transition curves are necessary when a reversal from a connecting curve takes places, providing a “transition” to the connecting curve that goes in the opposite direction, see Figure 3.6.

**a) Schematic of the components of the structural hysteretic behavior**
a) Example of envelope, connecting and transition curves

Figure 3.6 – Hysteretic behavior of any material or structural element
(adapted from Chang and Mander 1994)

Current concrete hysteretic models use a variety of curves to describe the connecting and transition curves. This variety is produced by the attempts to find solution to the lack of numerical stability under large displacements exhibited by many of them (Martínez-Rueda and Elnashai 1997).

In summary, the ascending branch of the concrete stress-strain curve can be approximated by different curves. Among them, those with a parabolic shape are widely used. The descending branch of the stress-strain curve can be assumed to be controlled by the peak strength and the first spiral/tie fracture points. Confined concrete also has a residual strength, which is approximately 20% of the peak strength. Finally, the concrete cyclic behavior can be approximated following models such as the one developed by Karsan and Jirsa (1969), which has the advantage of its relative simplicity. The concrete model adopted in this dissertation has
been based on all these observations, and is explained in detail in Section 3.6.1.

3.4 Steel

The pile is typically reinforced with mild steel dowels crossing the pile-deck interface at the connection, and also with prestressing steel. The most basic material model to describe the steel structural behavior could be the elastic-perfectly-plastic material. In this model, the strains and stresses are linear up to a point of yielding, after which the stress is constant and independent of the strain.

The hysteretic behavior in this simple model does not consider any stiffness degradation, or effects such as pinching or buckling. The unloading curves simply have the same slope as the loading curves, and the maximum stress remains equal to the initial yield stress. The choice of zero post-yield stiffness is suitable for many applications in which strain-hardening is not anticipated, and conservative for predicting the plastic collapse load and deformations whenever strain-hardening would be expected to develop (Bruneau et al. 2011). OpenSees ElasticPerfectlyPlastic model has the above characteristics, see Figure 3.7.

![Figure 3.7 - OpenSees ElasticPerfectlyPlastic Material (adapted from Mazzoni et al. 2009)](image_url)
This model can also be used to describe the fact that, in transfer and/or development regions, once the steel reaches a limit stress it cannot exceed, the deformations continue to increase. For instance, the transfer length and development length play an important role in the maximum capacity of the prestressed steel according to its position along the length of the pile (PCI Design Handbook 2010). However, it will have minimal effect on the behavior of the pile near the connection, mainly due to the limited stresses transferred between the prestressed steel and the surrounding concrete in that region and the more dominant relative effect of the conventional reinforcing steel at the connection. Nevertheless, it is essential in reducing cracking throughout the pile length (Priestley et al. 1996).

Other more advanced steel models include options such as kinematic or isotropic hardening (Bruneau et al. 2011). The kinematic hardening postulates that the distance between yield points in opposite loading directions remains constant. By contrast, the isotropic hardening postulates that if a stress higher than yielding is reached, then yielding would occur at the same higher stress in the reversed loading direction. Kinematic hardening in some measure accounts for the Bauschinger effect (upon stress reversal, a sharp “corner” in the stress-strain curve is not found at the onset of yielding, and stiffness softening occurs gradually). Both effects can be combined, as for example in the Dafalias-Popov (1975) two-surface model. In this model, the yield surface varies as a function of past loading history, while remaining within two linear bounding surfaces. Other complex piece-wise linear stress-strain models, for instance trilinear elasto-plastic with strain-hardening, have also been used for analysis.

Another class of material model, the power function, is particularly useful for computer programming purposes, since it expresses the entire stress-strain relationship by a single continuous function. The fundamental characteristic of a power function is that the strain-stress
relationship can be divided into an elastic part and a plastic part. In earthquake engineering, a particular type of power function, the Ramberg-Osgood function, is often used to model the behavior of structural steel materials. The Ramberg-Osgood function expresses strains as a function of stresses. If one derives stresses ($\sigma$) as a function of strains ($\varepsilon$), then the Menegotto-Pinto function is obtained. Its general form is as follows:

$$\frac{\sigma}{\sigma_0} = b \left( \frac{\varepsilon}{\varepsilon_0} \right) + d = b \left( \frac{\varepsilon}{\varepsilon_0} \right) + \frac{(1-b)\left(\frac{\varepsilon}{\varepsilon_0}\right)}{\left[1+\left(\frac{\varepsilon}{\varepsilon_0}\right)^n\right]^{\gamma/n}}$$  \hspace{1cm} (3.1)

Where b is the ratio of the final to initial tangent stiffness, the intersection of these two tangents gives $\varepsilon_0$ and $\sigma_0$, and n is obtained from the above equation if $\varepsilon_0/\varepsilon_0 = 1$. The Menegotto-Pinto function is particularly important for the computational economy of analysis. It is also important when the Bauschinger effect is significant. The limitation of this model lies in its inability to reach the point of last unloading upon reloading in the same stress direction (Bruneau et al 2011).

The multilinear and continuous models presented above describe monotonic response, but can be extended for cyclic behavior. The simplest model of this type, for instance, adds the kinematic hardening to the Menegotto-Pinto function (see the OpenSees Steel02 model) (Mazzoni et al 2009). More advanced models, such as the Chang and Mander steel model (1994), describe even more carefully the monotonic curve, including the elastic branch and strain hardening branch, a yield plateau, and postultimate strain or strain softening. The hysteretic behavior in these models is controlled by a number of rules. These rules describe phenomena such as pinching of force and deformation, damage related to ductility and energy, or degraded unloading stiffness. Other complex physical behaviors such as low cycle fatigue and local buckling can also be
incorporated through these hysteretic rules. The Gomes and Appleton (1997), or the Dhakal and Maekawa (2002) buckling models, as well as the Coffin-Manson fatigue and strength reduction factors, have been implemented in structural programs such as OpenSees (see Fig. 3.8).

Figure 3.8 describes the general or backbone curve for OpenSees ReinforcingSteel, which is based on the Chang and Mander model. It also shows the use of the buckling parameters \( r \), \( \beta \), and the strength reduction by low-cycle fatigue.
and $\gamma$, modified from Gomes and Appleton (1997). The buckling curve is scaled through the amplification factor $\beta$, controlling the location of the point at which buckling starts. The $r$ factor is used to adjust the curve between the buckled curve and the unbuckled curve. The $\gamma$ factor controls the positive stress location about which the buckling factor is initiated.

In summary, the steel elastic-perfectly-plastic material is a simple and conservative model that can describe the steel behavior in transfer and/or development regions. In fact, this model is used in this dissertation to describe the prestressed steel strain-stress curves. On the other hand, more sophisticated steel models include elastic, yield plateau, and strain hardening zones, as well as options such as kinematic and/or isotropic hardening. The cyclic behavior of these models include pinching of force and deformation, damage related to ductility and energy, and degraded unloading stiffness. Of particular importance are the low-cyclic fatigue and buckling phenomena. The dowel bar steel model adopted in this dissertation has been based on these observations, and is explained in detail in Section 3.6.1.

3.5 Link elements

Careful modeling of the inelastic rotation at the end of the pile into the wharf deck at the connection is of the utmost importance because it is associated with the extent of structural damage (and may also have implications on retrofit strategies). Even though simple methodologies, such as using the concept of a plastic hinge, can be useful (Priestley et al. 1996), a more detailed approach can take into account that the pile end inelastic rotation consists of two components: concrete softening and reinforcement slip (Maekawa et al. 2003). Because of the existence of these two components, the pile end inelastic rotation can be modeled through an additional spring element with a fiber section that also includes both concrete and steel (see Figure 3.9). This spring can use a zero-length fiber section element, because of its generality that
allows the strain penetration effect to be captured despite the cross-sectional shape and direction of the lateral load (Zhao and Sritharan 2007). Different models were proposed for steel as well as for concrete in this type of element (Zhao and Sritharan (2007), Oehlers et al. (2008), and Hasket et al. (2009)).

![Diagram](image)

**Figure 3.9 - Analyzing inelastic rotations at the base of a pile in a fiber-based model**

*(Zhao and Sritharan 2007)*

### 3.6 Analytical modeling of pile-deck connections

The fundamental aspects of structural behavior to be captured by the developed analytical model are related to the pile and deck components, as well as their interface. The pile has therefore been modeled as a simple cantilever supported on the deck (see Figure 3.10), with the deck/pile interface (connection) behavior represented by a rotational spring. The thick deck itself acts locally as almost a rigid body, as indicated in the experiments, so the rotational spring primarily represents the interface connection behavior. The model was developed using the finite element software OpenSees (Mazzoni et al. 2009), which has specifically been designed to perform advanced nonlinear seismic analysis.
The pile itself is modeled using a number of frame (beam-column) elements with different transverse reinforcement along the pile length, as well as taking into account the transfer length and development length of the prestressing steel and the development length of the dowel bars. A mesh sensitivity study was carried out to define the number of elements that should be used along the pile length. The nonlinear behavior of these elements is distributed along the pile length and determined by numerical integration of the constitutive behavior at a finite number of control sections (integration points). The OpenSees geometrically nonlinear force-based beam-column element was used since it provides a favorable balance between accurate assessment of nonlinear curvatures along the length of the element (flexibility method) and capability to directly include geometric nonlinearity (stiffness method) (Denavit 2009).

A special type of fiber-based element available in OpenSees, a “zero-length” element, has been incorporated at the base of the pile to model the inelastic rotation at the end of the pile. This
particular element (a rotational spring type) avoids any change in the geometry of the connection but allows including a fiber section that can take into account the interaction between the softening of concrete and slipping of reinforcement at the end of the pile.

The experimental specimens modeled in this study showed high flexural ductility values and eventual structural failure by reinforcing steel fracture, which can typically only be reached with a behavior dominated by bending (as opposed to shear). Based on the lack of pile shear damage, as well as there being no evidence of local horizontal shear slip deformations at any of the connections, modeling of shear deformations was not considered necessary.

3.6.1 Pile section model

As can be seen in Figure 3.10, to simplify modeling of the pile, the octagonal pile section was modeled as circular with an equivalent moment of inertia. The fiber-approach technique has been used to derive the non-linear constitutive behavior at the control sections of the elements described previously. In this technique, the section is discretized into a large number of fibers, and the sectional behavior is derived through the integration of uniaxial stress-strain relationships assigned to each fiber (depending on the material that the fiber represents). Four basic material models are required: a) unconfined concrete for the cover of the pile section, b) confined concrete for the pile core, c) prestressing steel for the strands, and d) regular steel for the conventional dowel reinforcing bars that connect the pile to the wharf deck slab.

Based on the literature review in Section 3.3, the OpenSees uniaxial material model *Concrete01* was used in this study to represent the behavior of unconfined and confined concrete in the section’s cover and core, respectively. The ascending branch of the unconfined concrete model (a parabola) is defined by the initial modulus of elasticity and the peak stress, while the descending branch is simply linear (as seen in Figure 3.11a).
a) Unconfined and confined concrete material models using OpenSees `Concrete01`

b) Regular steel (OpenSees `Reinforcing Steel`)  
c) Prestressed steel (OpenSees `ElasticPerfectlyPlastic`)

**Figure 3.11 – OpenSees material models used in the study**

In the case of the concrete material, Figure 3.11 is intended to show the typical general characteristics of the model used in this dissertation. The specific strength and strain concrete values vary depending on the particular concrete data.

Concrete tensile behavior was not considered in the analytical model since pile-deck connection structural behavior is ultimately controlled by concrete compression and reinforcing steel tension in flexure; therefore any enhancement of shear friction due to tensile capacity in concrete is not crucial. Furthermore, since the connection is not monolithically cast, early
cracking is expected near the deck-pile interface, which has minimal tension capacity.

Near the pile-deck connection, the core of the pile is typically confined with a relatively high ratio of transverse reinforcement, which considerably changes the concrete behavior. Stress and strain at the confined concrete peak point were determined based on the model presented by Mander et al. (1988). In this model, following recommendations from Richart et al. (1928), the confined concrete strain at peak stress $\varepsilon_{cc}$ is calculated assuming that the ratio $R$ which represents increase in strain at peak stress to increase in strength at peak stress due to confinement is constant, as illustrated in Equation 3-2.

$$
R = \frac{\varepsilon_{cc} - \varepsilon_{co}}{\varepsilon_{co}} = \frac{\varepsilon_{cc} - 1}{\varepsilon_{co} - 1}
$$

(3.2)

Where $\varepsilon_{co}$ is the unconfined concrete strain at peak stress, $f_{cc}'$ and $f_{c}'$ are the peaks confined and unconfined concrete stresses, respectively. Preliminary analysis showed that a value of $R$ equal to 3 produces a stress-strain descending branch similar to that of the high strength concrete used in the piles, so hence $R$ was taken as 3 throughout this study. The ascending branch of the confined concrete model used in this study (see Figure 3.10) is consequently a parabola defined by the initial modulus of elasticity and the peak confined stress (calculated per Mander et al. 1988). The input modulus of elasticity is actually just based off of the specified compressive strength, $f_c'$, which is the strength indicated on the construction drawings, using the standard relationship presented in ACI 318 Section 8.5.1 (ACI 2011):

$$
E_c = 57000\sqrt{f_c'} \text{ [psi]}
$$

(3.3)

The descending branch is again linear, with the slope defined by the factor $R$ and an estimate of the first spiral fracture based on an energy balance approach (Mander et al. 1988).
The descending branch was extended until reaching a residual stress equal to 20% of the peak stress, which follows recommendations from Popovics (1998).

These piles are typically reinforced with both mild steel (as dowels crossing the pile-wharf interface) and prestressing steel. Based on the literature review in Section 3.4, the uniaxial material model *ReinforcingSteel*, available in the OpenSees library, was used to describe the behavior of the regular reinforcement (see Figure 3.10b). The local bar buckling model was based on Gomes and Appleton (1997), and low-cycle fatigue used a Coffin (1954) – Manson (1953) fatigue and strength reduction approach. The steel dowel bar properties (yield and ultimate stresses, elastic and strain hardening modulus of elasticity, initial strain hardening strain, and strain at peak stress) were all obtained from experimental coupons (Jellin 2008).

In the case of the prestressing steel, consideration was first given to the maximum stresses that this steel could reach at different pile regions, as limited by transfer and development lengths. As also explained in Section 3.4, the OpenSees uniaxial material *ElasticPerfectlyPlastic* was used to constrain the maximum stress value that the prestressing steel is able to reach at different locations along the length of the pile (as an example, see Figure 3.10c, which shows different prestressing steel models used for UW-1 and UW-2). The prestressing steel will most likely have a relatively minimal effect on the flexural behavior of the pile near the connection, mainly due to the limited stresses able to be transferred between the prestressing and surrounding concrete in that region, as well as the more dominant effect of the conventional dowel reinforcing steel at the connection. Nevertheless, prestressing can be quite helpful for reducing cracking throughout the pile length (Priestley et al. 1996).

The hysteretic behavior of the materials employed in the modeling also needs to be explained since the analytical modeling presented in this section is going to be validated by
experimental cyclic loading results. The most important of all of them are *Concrete01* and *ReinforcingSteel*. The hysteretic behavior of *Concrete01* is based on the work of Karsan and Jirsa (1969). *ReinforcingSteel* is based on the Chang and Mander (1994) uniaxial steel model. Fatigue parameters, as explained in a previous section, are based on the Coffin-Manson equation for plastic strain amplitude and the buckling simulation consists of a variation on Gomes and Appleton (1997). The *Hysteretic, ElasticPerfectlyPlastic* and *ElasticPPGap* material model hysteretic behavior does not consider any pinching or reduction in stiffness and strength.

### 3.6.2 Pile-deck interface section model

Additional rotations and displacements can be produced due to the rigid body movement concentrated right at the pile-deck connection. As shown in Figure 3.12, the additional rotation (θ) is equal to the slip of the dowel bars, which depends on the bond characteristics between the reinforcing steel and concrete, divided by the distance (*h*<sub>rebar</sub>) from the reinforcing bar to the gap tip closing point, which in turn depends on the properties of the concrete at the pile-deck interface.
This interaction between the concrete and steel reinforcement at the pile-deck connection was modeled through an additional “zero-length” element, with a fiber section that includes both concrete and steel. Zhao and Sritharan (2007) recommended that concrete in this sort of a section be modeled with a residual strength equal to 80% of its maximum value. Based on preliminary analytical results it was found that this recommendation leads to good agreement with the experimental results of piles with high transverse reinforcement ratios. However, clearly an important factor like the amount of transverse reinforcement should be explicitly taken into account if at all possible, as is the case in the concrete model presented by Oehlers et al. (2009).

In this model, it is suggested that under excessive lateral load and due to upward pressure on the pile base, a compressive wedge will develop (see Fig. 3.12). The effective longitudinal compressive strength of the softening zone at the onset of shear friction failure is $\sigma_{\text{soft}}$:

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

(3.4)

where $c$ is the cohesive term, $\sigma_{\text{lat}}$ is confinement pressure due to the internal steel spiral, $m$ is the
slope of the Mohr-Coulomb failure plane equation, and \( \alpha \), the angle between the horizontal and inclined faces of the weakest wedge can be computed as follows:

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

(3.5)

Note that the slope of the Mohr-Coulomb failure plane equation is that of the straight line touching all Mohr’s circles representing critical combinations of principal stresses (Goodman 1989), experimental values are presented by Oehlers et al. (2009).

The ascending and descending branches of this concrete stress-strain curve are similar to that of the unconfined concrete model presented earlier; however, the residual stress is equivalent to \( \sigma_{\text{soft}} \). The approach described above was implemented through the Concrete01 model, with the resulting stress-strain curve depicted in Figure 3.13(a).

![Figure 3.13](image)

**Figure 3.13** - Concrete stress-strain behavior (a), and stress vs. slip of reinforcing bars (b) at the pile-deck interface

Steel slip is the other most important factor for describing the additional rotation right at the pile-deck connection, as explained by Zhao and Sritharan (2007). The bond-slip steel model
presented by Haskett et al. (2009) is used in this study since it provides a mathematical explanation for slip of reinforcing bars and therefore can be easily adapted to different bond conditions. The model states that for reinforcing bars remaining linear elastic (and that do not fracture prior to debonding):

\[
P_{\text{rebar}} = \frac{\tau_{\text{max}} \pi d_b}{\lambda} \sin\left\{\arccos\left(\frac{\delta_{\text{max}} - \Delta_{\text{rebar}}}{\delta_{\text{max}}} \right)\right\}
\]

(3.6a)

where

\[
\lambda^2 = \frac{4\tau_{\text{max}}}{\delta_{\text{max}} d_b E}
\]

(3.6b)

\(P_{\text{rebar}}\) is the force in the dowel reinforcement, \(\tau_{\text{max}}\) is the maximum bond strength, \(d_b\) is the diameter of longitudinal steel, \(\delta_{\text{max}}\) is the slip after which the bond stress remains at zero, \(\Delta_{\text{rebar}}\) is the bar slip, and \(E\) is the steel modulus of elasticity. \(\tau_{\text{max}}\), depends on the concrete strength and on the type of bond failure, per Eq. (3-6c).

\[
\tau_{\text{max}} = k \times \sqrt{f_c'}
\]

(3.6c)

where \(f_c'\) is the unconfined concrete peak stress and \(k\) is a factor that takes into account the type of failure expected (2.5 if interface bond failure, or 1.25 if interface bond failure + splitting + cone pull out). In this case \(k\) is taken as 1.7, slightly below the mean value between 2.5 and 1.25, since some splitting is expected. Figure 3.13(b) shows the resulting stress vs. slip curve for the reinforcement. This curve can be used in the “zero-length” element as equivalent to a stress-strain curve because the element has unit length, and hence displacements are equal to strains. In Figure 3.13(b) it can also be seen that these previous equations are approximated through the Hysteretic OpenSees material model, which uses a tri-linear envelope. Parameters related to pinching, as well as to degrading unloading stiffness, were not considered in order for the stress-
slip model to maintain its simplicity.

3.6.3 The cotton duck bearing pad element and sectional model

For Specimen UW-2, the pile-deck connection model included the cotton duck bearing pad and unbonded T-headed connection steel, as was the case in the experimental specimen. The cotton duck bearing pad was modeled as a fiber element using the OpenSees force-based beam-column element. It was modeled as a beam-column element even though its length is only 19 mm (0.75 in). Similar results could be obtained if it was represented with a zero-length element. The sectional behavior was described using OpenSees uniaxial Material ElasticPPGap, which works only in compression, with a modulus of elasticity of 690 MPa (100 ksi). This value was estimated from compression tests (Stringer 2010), taking into account friction between the pad and the adjacent concrete surfaces, as well as the confinement provided by the deck and the pile. The same OpenSees uniaxial material ReinforcingSteel model for the pile sections was also used to describe the dowel bars through the cotton duck bearing pad section.

These experimental and analytical models of course represent only part of a complex marginal wharf structure. In an earthquake, some landward piles of a wharf may sustain damage not only at the pile-deck connection, but perhaps also at an “in-ground” hinge. The sequence of formation of these hinges could modify the capacity of a pile (Goel 2010), and therefore influence behavior of the marginal wharf. Nevertheless, current design practice is mainly oriented toward enforcing pile-deck connections as the most crucial of the hinges (Priestley et al. 2007). In this way, repair is facilitated since the connection is readily accessible, and as such analytical and experimental models like those described herein are appropriate to catch the most significant structural behaviors.
3.7 Analytical vs. experimental results

3.7.1 Specimen UW-1

Figure 3.14 shows the analytical and experimental moment versus pile curvature curves for Specimen UW-1. In the experimental case, the pile curvature was obtained as an average value between 0 and 12 inches above the pile-deck interface. Therefore, the experimental and analytical cases represent a section at 6 in. from the deck soffit. However, the experimental curve is an envelope of the cyclic results, in contrast with the analytical case which was obtained from a pushover analysis. Even though the potentiometers providing the data used to calculate curvatures in Specimen UW-1 became inaccurate after heavy deck spalling (Jellin 2008), there was enough available experimental data to check the accuracy of the moment vs. curvature analytical model at least up to the first stages of nonlinear sectional behavior after peak moment values were reached. It should be noted that this was not the case for Specimen UW-2 since the available data covered only the elastic range of values at the crucial section (at the end of the pile).

![Figure 3.14 - Specimen UW-1: moment vs. curvature graphs](image)

As indicated by the figure, there is good agreement between the analytical and
experimental results. The peak experimental and analytical moments have a slight difference of only 3%. It is also important to note that the concrete cover, which is a large portion of the pile section, plays an important role in defining the moment capacity of the section. Therefore, a relatively large reduction in moment capacity was observed after spalling of the cover.

The experimental and analytical moment-displacement relationships of Specimen UW-1 under cyclic load are shown in Figure 3.15. The pile-deck connection was modeled as described earlier, including the rotational spring. There were relatively minor differences over the first few cycles, due to the tensile strength of concrete (which was for simplicity ignored in the model). Immediately after reaching the peak moment, the experimental results showed a slight descent in the moment while the analytical model showed a slight ascent. This difference can most likely be attributed to the simplified sectional material model used for the embedded portion of the pile. However, it proved to have almost no effect on the global behavior of the connection, such as the maximum moment capacity, lateral displacement, or hysteretic energy enclosed within each loading cycle.

![Figure 3.15 - Specimen UW-1: cyclic moment vs. displacement graphs](image)

a) Experimental  

b) Analytical

After maximum displacement equals approximately 50 mm, an abrupt drop is visible in the
experimental as well as the analytical results. The next cycles showed consistent reduction in the moment capacity, by about 10% in every series of cycles. The analytical model was able to capture these reductions in strength with an acceptable level of accuracy. The inclusion of the rotational spring increased the analytical displacements by as much as 40% compared to the case without the rotational spring, even though the general shape of the curves would have been similar without it; clearly there was concentrated damage at the connection that this aspect of the model is needed to be able to catch.

Nevertheless, in the analytical as well as the experimental results, damage was concentrated along a small length at the pile end. In the analytical model, for instance, a reference point at approximately 200 mm (7.5 in.) above the pile-deck interface shows a contribution to rotation at the tip of the pile of 80% even at the beginning of damage to the core concrete (2.4% drift). It should also be noted that the reported experimental results at the pile-deck connection are lumped values, which include all behavior below the pile-deck interface (elongation / slip of the dowel bars and crushing of the pile end). On the other hand, the analytical model identifies independently the contribution from slip of the dowel bars and that from pile bending. In this model, during the first few cycles up until 2.4% drift, the contribution of dowel bar slip (represented by the rotational spring) to the total rotation at the reference point 200 mm above the pile-deck interface increases and reaches a maximum of approximately 70%, after which this contribution decreases to less than 10% by 8.5% drift.

Points of visual damage are also presented on the experimental graph in Figure 3.15 (including “Minimal spalling” and “Substantial spalling”), as well as the related analytical points (i.e., “Cover max” and “Core ult”, respectively) in the corresponding analytical graph. Note that these latter “crucial” stress-strain curve points, namely when concrete reaches its peak stress at
the extreme perimeter cover fiber (“Cover max”) and its residual stress at the extreme perimeter core fiber (“Core ult”), are clearly associated with the similar experimental observations regarding actual damage. “Cover max” is associated with severe cracking and localized spalling at the pile-deck connection. “Core ult” is associated with complete spalling of concrete cover and initial core damage. During substantial spalling of the concrete in the UW-1 specimen, buckling of some of the dowels, now exposed, was observed. Finally dowel fracture occurred. The sequence of events described above is typical for the type of deck-pile connection being analyzed, and is also observed in the other experimental specimens used in this work. The fracture of the dowels, which can be considered as the ultimate failure mode, can be noticed in the experimental moment-displacement curve by sharp drops in the moment capacity. In the analytical model, the dowel bar’s strength eventually diminishes to zero due to low-cycle fatigue. This point was defined as the “Steel Ultimate” point and was reached at cycles with similar drifts to that where bar fracture was observed in the experiment.

3.7.2 Specimen UW-2

In the case of Specimen UW-2, Figure 3.16 shows the cyclic moment-displacement plots comparing experimental and analytical results. The analytical curve includes the combined effect of the pile, the rotational spring, and the cotton duck bearing pad. From the figure, it is clear that the peak experimental and analytical moment values are quite similar, as are the overall shape of the curves. This statement is also confirmed by calculating the area inside the moment-displacement loops, which is almost identical in both experimental and analytical cases. The analytical curve, though, has a more pronounced pinching behavior. This is because the model does not consider the phenomenon where many concrete cracks are filled with detritus from adjacent damaged concrete surfaces, so they therefore close more quickly in reality than what the
model might predict.

![Graphs showing experimental and analytical results for Specimen UW-2](image)

**Figure 3.16 - Specimen UW-2: experimental vs. analytical results**

Comparison between the two analytical models (with and without bearing pads) indicates that the displacement capacity of the pile is further enhanced when a cotton duck bearing pad is incorporated at the pile-deck connection interface. Inclusion of the pad significantly changes the overall behavior of the pile-deck connection since the concrete damage is delayed compared with that without the pad. However, the last cycles continue to be controlled by buckling and low cyclic fatigue of the dowel bars.

### 3.7.3 Specimen SD-1

Due to insufficient experimental data for the SD specimens, it was not possible to draw a comparison between the experimental and analytical behavior at the sectional level (i.e., for moment vs. curvature). However, a comparison was carried out at the component level, using the force vs. displacement relationships: those experimental and analytical graphs for specimen SD-1 (with the non-seismic pile and connection detailing) are presented in Figure 3.17.
a) Experimental Results (adapted from Krier 2006)  

b) Analytical Results

Figure 3.17 - Results comparison: SD-1 (non-seismic pile) force-displacement

The analytical model gives reasonable results compared to the experimental specimen’s overall behavior. Maximum forces and their related displacements are quite similar. However, there are modest differences in the linear range of behavior, as well as in the shape of the curves, which have already been discussed in regards to the two specimens from the University of Washington – namely, minor differences in the first elastic cycles, and the experimental results showed a slight descent after reaching the peak moment but the analytical showed a slight ascent and then an abrupt descent. The principal additional differences are in the last cycles where the analytical capacity degraded faster than the experimental.

Even though changing the OpenSees ReinforcingSteel material model parameters could probably help in bringing the analytical behavior even closer to the experimental behavior, the same “standard” values used for the UW specimens were simply maintained since the model already showed a good approximation to the test in regards to global behavior such as maximum moment/lateral displacement and hysteretic energy. In Figure 3.17 it can also be seen the strain limit states specified in the Port of Los Angeles seismic code (POLA 2004) for both the Operational Level Earthquake (OLE) and Contingency Level Earthquake (CLE). The OLE
concrete strain limit could be applied for the cover concrete and the CLE concrete strain limit for
the core concrete. The OLE strain limit state was controlled, on both the analytical and
experimental curves, by steel strains equal to 0.010. However, the CLE experimental value is
only an approximation since the exact strain limit state could not be measured, because the strain
gauges at the dowel bars, which were used to obtain those values, failed very early on during
testing (Restrepo et al. 2007).

a) Experimental Results (adapted from Restrepo et al. 2007)  b) Analytical Results

Figure 3.18 - Results comparison: SD-2 seismic pile force-displacement

3.7.4 Specimen SD-2

The experimental and analytical force vs. displacement graphs for specimen SD-2 (with
the seismic pile and connection detailing) are shown in Figure 3.18. The modest differences
between the analytical and experimental results are quite similar to those already discussed for
Specimen SD-1. Both analytical and experimental OLE values were controlled by steel strains
equal to 0.010. As was the case in Specimen SD-1, the CLE strain limit state is only an
approximation since it could not be obtained experimentally (Krier 2006, Restrepo et al. 2007).
3.8 Bar slip at the pile-deck connection

The UW-1 analytical model developed in this dissertation reported the following results (see Figure 3.19).

![Graph showing analytical results at the pile-deck connection in Specimen UW-1](image)

Figure 3.19 – Analytical results at the pile-deck connection in Specimen UW-1

Figure 3.19 shows versus the ratio between rotation due to the slip of the rebars (“Rotation 1”) and total rotation at the pile-deck connection (“Rotation 2”) versus displacement at the tip of the pile. First, an increase in the contribution of the slip of the rebars to the total rotation is observed up to approximately 1.5 in. Then, this contribution remains relatively constant up to approximately 2.5 in. Finally, the relative contribution of the slip of the rebars to the total rotation is increasingly reduced.

Experimental results were hindered by instrumentation limitations that precluded a
detailed analysis of the pile-deck connection. For instance, a lumped rotation value at the pile-deck interface takes into account the contribution of slip of the rebars, as well as crushing of the concrete at the bottom of the pile, at the embedded zone into the deck. Furthermore, some critical instrumentation failed at the beginning of the nonlinear range of structural behavior (Jellin 2008).

3.8.1 Bar-slip of UI specimen

Foltz (2011) reported important experimental results on bar slip at the pile-deck connection, for a specimen tested at the University of Illinois @ Urbana-Champaign, in 2010.

The slip calculated by Foltz at the pile-deck interface over the entire earthquake loading is shown in Figure 3.20. Note that the extreme dowel bars are identified as North and South Extreme Bars.

![Figure 3.20 – Slip at the pile-deck interface during “earthquake” loading](source: Foltz 2011)
During the first 300 steps, loading corresponds to the Imperial Valley earthquake. Between the 300\textsuperscript{th} and 700\textsuperscript{th} steps, it corresponds to the Northridge earthquake. Finally, the graph sector beyond step 700 corresponds to the Kobe earthquake. Figure 3.20 shows relatively limited slip during the Imperial Valley record, relatively moderate slip of the north bars during the Northridge record, and relatively more important slip of both the north and south extreme bars over the course of the Kobe record. After the initial earthquake loading, the specimen was then tested under additional cyclic loading. As reported by Foltz, the increase in slip is fairly proportional to the increase in the amplitude of the cycle. He concluded that at higher displacements the contribution due to slip is higher.

It should be noted that slip of the rebars is considered as the main contributor to the differences between the observed total rotation at the pile-deck connection and that corresponding to the flexural behavior of the pile. Foltz stated that at relatively lower loads, flexural rotation of the pile accounts for about 30\% of the total rotation of the pile, with 70\% being attributed to a lumped connection rotation. The lumped connection rotation then later accounts for as much as 90\% of the total rotation, as damage progresses. Note that these results are actually relatively similar to those observed in Specimen UW-2, even though that specimen has a bearing pad at the pile-deck interface. Foltz explained this behavior as an effect in part of the relatively low axial load \( (0.02f'_cA_g) \) used in his specimen.

On the other hand, Figure 3.21 shows the slip at the pile deck interface and Figure 3.22 total and due to the slip rotations at the pile-deck structure using the model developed in this study.
Figure 3.21 – Slip at the pile-deck connection using the analytical model developed in this study

Figure 3.21 shows relatively similar trends as those observed in Figure 3.20 for the bar slip under the Imperial Valley and Northridge loading. During the Imperial Valley record, the slip was relatively small, and during the Northridge record, important slip peaks were present at the beginning of the record, followed by bigger slips than those recorded during the Imperial Valley earthquake. However, the magnitude of the mean calculated slip during the Imperial Valley record is more important than the experimental slip. In a similar way, during the Northridge record, it can be observed, particularly in the case of the south extreme bar, that the calculated slip is again more important than the experimental slip. In the case of the Kobe record, the calculated slip reaches values such as 0.2 inches (5.1 mm), in sharp contrast with the
experimental slip, which reaches values as high as 0.6 in. (15.2 mm). It is considered that these different behaviors could be better explained using Figure 3.22.

Figure 3.22– Total (Rotation 2) and due to the slip of the rebars (Rotation 1) rotations at the pile-deck connection using the analytical model developed in this study

Figure 3.22 shows the total rotation and the rotation due to slip of the rebars, at the pile-deck connection produced by the Imperial Valley, Northridge, and Kobe records. Even though the contribution of the rotation due to slip of the bars varies according to the load history, a peak value equal to approximately 55% of the total rotation, is recorded in the Imperial Valley record. This is similar to the value reported by Foltz. In the case of the Northridge record, a contribution equal to 71% was recorded. This is again relatively similar to the value reported by Foltz. In the case of the Kobe record, relatively important slip-related-rotation differences were observed. At this point, it should be noted that this analytical model assumed that the tip of the pile was fixed
and did not model the attachment bars or the steel plates used to connect the pile to the “Loading and Boundary Condition Box” (LBCB). In fact, the pile zone immediately adjacent to the LBCB showed relatively significant concrete cracking, and therefore, important stress concentrations. In addition, the analytical model did not take into account concrete crushing in the deck. It can be hypothesized that, due to the relatively small axial loads, the crushing of the concrete in the deck, and even the experimental set-up (the deck is below the pile), a “gap” was formed between the pile and the deck, and at certain loading steps the rebars were the main elements transmitting load between the pile and the deck. This behavior could explain the relatively higher observed contribution of the slip of the rebars to the pile total rotation in the Kobe record.

Note also that due to the relatively reduced range of displacements and/or drift in which experimental data is available, as well as the somewhat contradictory available experimental results, additional experimental research is needed to validate the analytical results reported in this section.

Finally, and in summary, this Chapter has highlighted different factors that control the structural behavior of the pile-deck connection and which are included in the analytical model. Sectional flexural factors such as those given by the concrete and steel working under compression and tension, respectively, are complemented by other phenomena such as low-cyclic fatigue and local buckling of the steel reinforcement. Moreover, the member behavior of the pile-deck connection has to include the phenomena related to the slip of the bars. In addition, the shape of the curves displacement-force has been described carefully and related to structural damage. Definitions and different methods to measure structural damage are explained in detail in Chapter 4.
CHAPTER 4: DAMAGE ESTIMATION

To implement performance- or damage-based design approaches for a given earthquake level, design engineers require rational and simplified analytical models to define the damage with engineering criteria such as strain and ductility. To facilitate repair and retrofit decisions, they must also quantify the damage in simple terms under various loading conditions, creating damage indexes that take into account various design parameters (Prakash and Belarbi 2010). A review of available damage indexes is necessary because the current research on pile-deck connections has not settled on a single damage index.

Lehman and Moehle (2001) proposed the damage states given in Table 4.1. They based their approach on flexural tests of RC columns.

<table>
<thead>
<tr>
<th>DAMAGE LEVEL</th>
<th>GENERAL DESCRIPTION</th>
<th>ADDITIONAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Minor</td>
<td>Hairline cracking</td>
<td>Light cracking throughout</td>
</tr>
<tr>
<td>2. Moderate</td>
<td>Cracking and minimal</td>
<td>Severe cracking, yielding of spiral, localized</td>
</tr>
<tr>
<td></td>
<td>spalling</td>
<td>spalling</td>
</tr>
<tr>
<td>3. Severe</td>
<td>Substantial spalling</td>
<td>Complete spalling of concrete cover, crushing of diagonal</td>
</tr>
<tr>
<td></td>
<td></td>
<td>struts and core damage</td>
</tr>
<tr>
<td>4. Collapsed</td>
<td>Onset of bar damage</td>
<td>Slight buckling of longitudinal bars, severe</td>
</tr>
<tr>
<td></td>
<td></td>
<td>core damage in the middle of the column</td>
</tr>
</tbody>
</table>

These damage states are similar to those proposed by other researchers, such as Priestley et al. (1996): cracking, first-yield, spalling, ultimate. But they also relate to limit states of overall structural response: serviceability, damage-control, and survival. In the serviceability limit state, spalling of concrete should not occur, and crack widths should be sufficiently small such that injection of grout is not needed; in the damage-control limit state, damage must be repairable;
and finally, in the survival limit state, a reserve of capacity above that corresponding to the damage control limit state should exist to ensure collapse of the structure does not take place. Protection against loss of life is the prime concern in the last limit state described above; demolition and replacement may be required.

Guidelines such as MOTEMS, and those produced by Port of Los Angeles (POLA), and/or Port of Long Beach (POLB) establish “acceptable” damage at pile-deck connections based on the level of seismic hazard. This damage can also be related to the damage observed in experimental research. For instance, zones can be established in experimental force vs. displacement curves where this “acceptable” damage is observed. These observations of damage can be further related with particular levels of certain parameters in analytical models. MOTEMS uses crucial strains in both steel and concrete. In this Chapter, it is shown that the crucial strains calculated in the analytical model developed in this dissertation are indeed related to certain levels of observed damage. This can be considered an additional confirmation of the validity of the analytical model to describe the behavior of pile-deck connections. However, strains are output data that is not usually available in current commercial structural analysis programs. Consequently, damage parameters related to output available in commercial software are needed. In this regard, it is proposed to use well-known damage indexes instead such as the Park and Ang (1985) damage index.

Damage indexes provide means to quantify the damage sustained by concrete structures during earthquakes. Damage indexes may be defined locally at the cross-section or member level for an individual element, or even for an entire structure. They are dimensionless parameters intended to range from a lowest value for an undamaged structure to a highest value for a structure near or at collapse, with intermediate values estimating the degree of damage. Among
them, the Park and Ang (1985) model is the most widely used by researchers. In this model, the seismic structural damage is expressed as a linear combination of the damage caused by excessive deformation and that contributed by repeated cyclic loading effects.

Currently, the majority of these indexes are for flexural behavior. However, pioneering works like the analytical model from Jeong and Elnashai (2006) have taken into account flexural bidirectional and torsional responses. As another example, Prakash and Belarbi (2010) proposed extending existing flexural failure damage indexes to flexural-shear-torsion loading failure modes.

Finally, the crucial role of residual or permanent deformations as a valuable damage indicator has also been recognized in the literature (MacRae and Kawashima 1997; Pampanin et al. 2002), and therefore the enhanced efficiency of re-centering solutions has increasingly been emphasized and promoted.

An initial difficulty in replacing MOTEMS’ damage strain levels with damage indexes is that the damage scales adopted for these indexes, are different from that used by MOTEMS. It is therefore necessary to review these damage scale characteristics. It is noted that these scales are based on the damage observed in structural elements controlled by bending, which is also the case in the pile-deck connection. Obviously, the description of the sequence of damage in these elements agrees closely with that observed in the pile-deck connection experimental research described in earlier Chapters. However, it has been decided to enrich their general description of damage with particular observations obtained from the pile-deck connection experimental research. This modified damage scale is then compared with the damage scale recommended by MOTEMS, and conclusions are extracted with respect to their relationship.

From preliminary studies developed in this dissertation, it was concluded that the Park
and Ang damage index is indeed the most promising alternative. However, it is compared to five additional indexes, in order to accurately establish its advantages and disadvantages. These indexes are selected from the literature review in section 4.2.3 and explained in section 4.2.7.

In summary, this chapter is divided into three parts: damage assessment using MOTEMS, damage assessment using the Park and Ang damage index, and comparison between the two methods. In the second part of this chapter, since the Park and Ang damage index is unprecedented for use with pile-deck connections, a general review of damage scales and indexes is also undertaken, as well as a detailed study of the Park and Ang damage index along with five additional indexes.

4.1 Damage assessment based on MOTEMS

The marine industry, responsible for the design of pile-deck connections in port structures, has developed specialized structural criteria to define damage states. They are somewhat different from those developed by the building industry since, due to the particular condition of use of port structures, they have paid more attention to economics than to life safety.

Note also that current marine industry seismic design is performance-based. Early examples are the guidelines developed by the Port of Los Angeles (POLA 2004) and the Port of Long Beach (POLB 2009). An additional important guideline is that developed by the Permanent International Association for Navigation Congress (PIANC 2001). This pioneering work has been principally developed in the state of California; it is not a surprise that the first legal standard for port structures in the United States, following performance-based criteria, has been approved there. MOTEMS is the acronym for Marine Oil Terminal Engineering and Maintenance Standards of the state of California, and has been approved as recently as 2007. And last, but not least, a new ASCE standard, Seismic Design of Pile Supported Piers and
Wharves, is currently being developed by the Coasts, Oceans, Ports, and Rivers Institute.

MOTEAMS is further explained in the following paragraphs. It is an important guideline, since it currently summarizes the work developed in a high seismic area such as the west coast of the United States by previous similar seismic design guidelines (POLA, POLB, PIANC). It is a pure performance-based code, where the structural requirements are essentially dictated by four components:

- Definition of earthquake levels to be considered.
- Definition of where inelastic action may occur (i.e., only in piles).
- Definition of performance limit strains for different pile types and different earthquake levels.
- Definition of how capacity protection for elastically responding members and actions is to be applied.

MOTEAMS specifies that two levels, called Level 1 and Level 2, of seismic design performance shall be considered. For high risk port structures, Level 1 corresponds to an earthquake with 50\% probability of occurrence in a 50 year time frame (i.e., with a return period of 72 years). Level 2 corresponds to an earthquake with a 10\% probability of occurrence in a 50 year time frame (a return period of 475 years). The following performance criteria apply:

Level 1: Minor or no structural damage. Temporary or no interruption in operation.

Level 2: Controlled inelastic structural behavior with repairable damage. Prevention of structural collapse. Temporary loss of operations, restorable within months.

On the other hand, in experimental research, typical damage observations are such as those presented in Table 4.2, as obtained from Specimen UW-1.

The first column in Table 4.2 refers to the maximum drift at which damage was observed, the
second column describes the damage, and the third column classifies the damage as none, minimal, moderate, severe, and near collapse. This damage classification will be further explained in Section 4.2.1. Additional information about Specimen UW-1 can be found in Chapter 3.

Table 4.2 - Typical damage observations (Specimen UW-1)

<table>
<thead>
<tr>
<th>Drift (%)</th>
<th>Observations</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Vertical cracks opened (west and east pile faces).</td>
<td>None</td>
</tr>
<tr>
<td>0.016</td>
<td>First horizontal cracking of the pile. By the end of this series horizontal</td>
<td>Minimal</td>
</tr>
<tr>
<td></td>
<td>cracks had formed at approximately 4 inches.</td>
<td></td>
</tr>
<tr>
<td>0.201</td>
<td>Vertical cracks opened (north and south pile faces).</td>
<td></td>
</tr>
<tr>
<td>0.244</td>
<td>An additional horizontal crack formed at approximately 36 inches from the</td>
<td></td>
</tr>
<tr>
<td></td>
<td>pile to the deck interface, spanning roughly 75% of the pile circumference.</td>
<td></td>
</tr>
<tr>
<td>0.483</td>
<td>Minimal pile spalling.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>By the end of the series flakes of concrete had spalled on both the north</td>
<td></td>
</tr>
<tr>
<td></td>
<td>and south pile faces.</td>
<td></td>
</tr>
<tr>
<td>0.945</td>
<td>Pile spalling became moderate.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>By the end of the series 11 pile spalling covered roughly from the pile/deck</td>
<td></td>
</tr>
<tr>
<td></td>
<td>interface to approximately 3 inches above the interface.</td>
<td>Moderate</td>
</tr>
<tr>
<td>1.194</td>
<td>Substantial pile spalling was reached.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>By the end of this series the cover had completely spalled on both the north</td>
<td></td>
</tr>
<tr>
<td></td>
<td>and south pile faces, exposing spiral reinforcement to approximately 8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>inches above the interface, with shallower spalling reaching several inches</td>
<td></td>
</tr>
<tr>
<td></td>
<td>higher on the pile.</td>
<td>Severe</td>
</tr>
<tr>
<td>2.455</td>
<td>Pile spalling continued to deepen.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>By the end of the series several prestressing strands and steel ducts were</td>
<td></td>
</tr>
<tr>
<td></td>
<td>visible on both the north and south pile faces.</td>
<td></td>
</tr>
<tr>
<td>3.933</td>
<td>Bar buckling occurred.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>During this series pile spalling continued to deepen, until it penetrated</td>
<td></td>
</tr>
<tr>
<td></td>
<td>slightly into the concrete core.</td>
<td></td>
</tr>
<tr>
<td>5.514</td>
<td>Bar fracture.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>The north-most bar fractured, as seen by a sharp drop in horizontal</td>
<td></td>
</tr>
<tr>
<td></td>
<td>resistance, during the first cycle of the series (drift=1.4%).</td>
<td></td>
</tr>
<tr>
<td>7.021</td>
<td>During the second cycle two additional bars fractured on the north-side of</td>
<td></td>
</tr>
<tr>
<td></td>
<td>the pile (drifts: 1.6% and 5.1%).</td>
<td></td>
</tr>
<tr>
<td>8.212</td>
<td>Near Collapse</td>
<td></td>
</tr>
</tbody>
</table>

Source: Jellin (2008)

These damage levels are more visible in a graph such as the force vs. displacement plot.
shown in Figure 4.1. In the figure, the non-damage zone is near displacement equal to zero. Then, it is followed, to the left and right, by the minimal, moderate, severe and near collapse zones.

Figure 4.1 - Levels of damage in a graph force vs. displacement (Specimen UW-1)

(1kN=0.22 kip, 1mm=0.04 in.)

According to the description of damage in Table 4.2, MOTEMS Level 1 of damage can be related to “moderate” observed damage, since relatively minor concrete spalling would hardly interrupt port operations. MOTEMS Level 2 of damage can be related to initial “severe” observed damage, since substantial cover spalling and even initial cracking in the core concrete, as well as exposure of several rebars, can still be repaired. This type of damage shows also that the structure has up to this point avoided bar buckling, bar fracture, and collapse.
MOTEMS expresses its two levels of damage in terms of crucial strains. For shallow embedment pile-deck connections of the type studied in this dissertation, and also for in-ground hinges, MOTEMS recommends the following values:

Table 4.3 - Crucial strains recommended in MOTEMS

<table>
<thead>
<tr>
<th>At the pile head (reinforced concrete capacity)</th>
<th>Level 1</th>
<th>Level 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extreme concrete fiber compression strain</td>
<td>$\varepsilon_c \leq 0.004$</td>
<td>$\varepsilon_c \leq 0.025$</td>
</tr>
<tr>
<td>Extreme tensile strain in dowels</td>
<td>$\varepsilon_s \leq 0.010$</td>
<td>$\varepsilon_s \leq 0.050$</td>
</tr>
<tr>
<td>In-ground hinge (prestressed concrete capacity)</td>
<td>Level 1</td>
<td>Level 2</td>
</tr>
<tr>
<td>Extreme concrete fiber compression strain</td>
<td>$\varepsilon_c \leq 0.004$</td>
<td>$\varepsilon_c \leq 0.008$</td>
</tr>
<tr>
<td>Prestressing strain in strands</td>
<td>$\varepsilon_s \leq 0.005$ (incremental)</td>
<td>$\varepsilon_s \leq 0.025$ (total strain)</td>
</tr>
</tbody>
</table>

Table 4.3 shows crucial strains for both concrete and steel at the two MOTEMS levels of damage. The strains are measured at the extreme fibers under compression or tension. For damage Level 1, the extreme concrete fiber corresponds to a cover concrete perimetral fiber. For damage Level 2, the extreme concrete fiber corresponds to a core concrete perimetral fiber. In the case of the steel, the extreme steel fiber corresponds to the dowel with higher tension strains. Crucial strains for in-ground hinges have also been included in Table 4.3, since Chapter 5 will include the pile-soil interaction.

It is worth noting that, at the pile-deck connection, the structural behavior has little to do with the prestressing steel, since the only working elements crossing the interface are the dowels. That changes into the body of the pile. At damage Level 1, the prestressed steel crucial strain is obtained as an increment to the initial strain. At damage Level 2, the prestressed steel crucial
strain includes the initial strain. Crucial strains at the in-ground hinge are more conservative than in the pile-deck connection. This should guarantee that damage occurs first at the pile-deck connection. The analytical model developed in this dissertation calculates these crucial strains at points on the force-displacement curve, similar to those obtained experimentally. As an example, Figure 4.2 shows UW-1 analytical results, including the position of MOTEMS Levels of damage 1 and 2. This characteristic has also been explained in Chapter 3, but this time the relationship with observed damage is highlighted.

Figure 4.2 - MOTEMS Level 1 and Level 2 of damage for Specimen UW-1

(1 kN= 0.22 kip, 1mm=.04 in.)
Figure 4.2 shows the positive analytical displacements and forces obtained for Specimen UW-1. The experimental observed damage zones are drawn as different type of shades on top of this graph. The points along the analytical curve at which MOTEMS damage Levels 1 and 2 occur are also depicted (with triangles). Figure 4.2 indicates that the analytical values obtained using MOTEMS damage level recommendations generally match with the observed damage.

4.2 Damage assessment based on damage indexes

As explained at the beginning of this chapter, an initial review of the damage scales, as well as the damage indexes themselves, is needed. Then, six of them are selected and applied to a typical pile-deck connection. Further damage analysis is undertaken using the Park and Ang damage index, since its damage estimation agrees well with the observed damage in University of Washington and University of California at San Diego specimens.

4.2.1 Visual damage states

Visual damage states are related to the visual identification of significant degrees of structural damage. Traditionally, four damage states are considered to describe the damage to reinforced concrete frame structures. These are Minor, Moderate, Severe, and Collapse. These damage states have been identified with simple visual signs of damage, as for instance in Park, Ang and Wen (1987).

4.2.1.1 Park, Ang and Wen damage states

These author have identified the following damage states:

- None: localized minor cracking at worst.
- Minor: minor cracking throughout
- Moderate: severe cracking and localized spalling
- Severe: crushing of concrete and exposure of reinforcing bars
Their simplicity helps these visual damage states to be applied both in experimental research and in post-earthquake inspections, but they lack more rigorous definition. For example, it is not clear the amount of cracking that can or should be defined as “minor”. Attempts have been made to refine these definitions and to associate Minor Damage, for instance, with repairs that only need epoxy injections; in other words, cracks with widths in the range of 0.5mm – 0.8mm. The lower bound of Moderate Damage can then be related to cracks with widths greater than 0.8 mm, which require a V-cut before repair. However, the type of crack (i.e., flexural, shear, or bond slip) could be more important than simply the amount or size of the cracks. On the other hand, these definitions focus only on reparability, leaving out other consequences (loss of human life and economic losses due to loss of use of the facility).

It should be noted that the persistent lack of precision in the definition of the observed damage states gives room for uncertainty in the measures of damage. This partially explains the important role played by “expert opinion” in damage estimation.

4.2.1.2 Strengthening of the visual damage states

A review of the observed structural behavior in pile-deck connections is further needed to appropriately strengthen the applicable visual damage states. Note also that there is a good similarity between the structural behavior reported for pile-deck connections and that reported for reinforced concrete bridge columns, if their failure is controlled by flexure. In this dissertation, data from structures such as bridge columns connected to footings was used to better understand the structural behavior of pile-deck connections.

Usually, the most notable visual observations are, in sequence of first occurrence, concrete cracking, longitudinal reinforcement yielding, initial spalling of the concrete cover,
complete spalling of the concrete cover, spiral fracture, longitudinal reinforcement buckling, and longitudinal reinforcement fracture. These damage states are described briefly, with help of the observed damage in Specimen UW-1 (see Table 4.2), in the following paragraphs:

4.2.1.2.1 Cracking and yielding

Initial damage is in the form of horizontal cracks, with crack spacing equal to approximately half the pile diameter (In Specimen UW-1, two initial cracks were observed at approximately 100 mm, 4 in., and 457 mm, 18 in., above the pile deck interface). In general, new cracks are not observed at subsequent displacement cycles to a given displacement level; however, at increasing levels of displacement demand, new cracks form (for instance, in Specimen UW-1, a third crack formed at approximately 1.5D from the pile to deck interface). However, cracking is concentrated during all the loading history at the pile-deck connection and do not distribute noticeably along the pile. After the initial horizontal cracks (two in the UW-1 case), additional vertical cracks at the tension/compression sides of the pile are observed. These are indicators of initial steel yielding of the longitudinal reinforcement. In UW-1, steel yielding was measured using a strain gauge that was attached to the extreme longitudinal bar immediately above the pile-deck interface. The geometry of the octagonal-cross-section pile is such that initial yielding is restricted to one or two longitudinal bars; therefore, softening of the load-displacement response occurs only gradually as yielding spreads to adjacent bars around the pile circumference. In Specimen UW-1, this damage state went up to a drift equal to 0.031%.

4.2.1.2.2 Initial spalling

Minimal pile spalling, consisting of flakes of concrete spalled on the tension/compression sides of the pile at approximately D/8 (D=diameter of the section) above the interface, follows yielding of the longitudinal reinforcement. The extent of concrete spalling increases with
growing displacement amplitude but remains essentially constant for cycles at constant amplitude. In Specimen UW-1, this damage state corresponded to drifts between 0.031% and 1.415%.

4.2.1.2.3 Moderate spalling

Additional spalling, which in the case of Specimen UW-1 was termed “moderate”, then occurs but is still contained in the concrete cover, with no spiral visible, and covers roughly from the pile/deck interface to D/3 above the interface. In Specimen UW-1, this damage state corresponded to drifts between 1.415% and 2.455%.

4.2.1.2.4 Substantial spalling

Cover spalls off completely on both tension/compression pile sides, exposing spiral reinforcement to approximately D/3 above the interface, with minor spalling reaching several inches higher on the pile. Pile spalling continues to deepen, in such a way that several prestressing strands and steel ducts are visible on the tension/compression pile sides. In Specimen UW-1, this damage state corresponded to drifts between 2.455% and 7.021%.

4.2.1.2.5 Bar buckling, bar fracture, and loss of lateral load-carrying capacity

Once the cover concrete spalls off completely and the spiral and longitudinal reinforcement are exposed, longitudinal bar buckling is observed within the next displacement cycle. Pile spalling also continues to deepen, until it penetrates slightly into the concrete core. The lateral deformation of the buckled bar increases during subsequent displacement cycles at a given displacement level, and the spirals located within the buckled length distort noticeably (the data available for UW-1 consisted of pictures that apparently confirm higher deformations in the spirals).

No spiral fracture was reported in UW-1. The length of the buckled region spanned
approximately two spiral spacing and was just above the pile interface with the deck. Longitudinal bar fracture could be estimated indirectly as sharp drops in horizontal resistance. A first bar failed at the initial cycle to 8.2% drift, and two additional bars failed during the second cycle of the same series. In both cases, they did not fail at the maximum displacement of the series. The information above was used to adjust the descriptions of visual damage states as applied to pile-deck connections (see Table 4.4).

Table 4.4 - Standard and enriched descriptions of pile visual damage states (VDS)

<table>
<thead>
<tr>
<th>VDS</th>
<th>Standard Description</th>
<th>Enriched Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
<td>Localized minor cracking at worst.</td>
<td>Localized minor cracking at worst. Small vertical cracks open.</td>
</tr>
<tr>
<td>Minor</td>
<td>Minor cracking throughout.</td>
<td>Minor cracking throughout. Minimal spalling (initial concrete flaking).</td>
</tr>
<tr>
<td>Moderate</td>
<td>Severe cracking and localized spalling.</td>
<td>Severe cracking and localized spalling. The spalling does not allow seeing the spiral.</td>
</tr>
<tr>
<td>Severe</td>
<td>Crushing of concrete and exposure of reinforcing bars.</td>
<td>Crushing of concrete and exposure of reinforcing bars. Substantial spalling that penetrates slightly into the concrete core. Initial bar buckling.</td>
</tr>
<tr>
<td>Near Collapse</td>
<td>-</td>
<td>Bar buckling increases significantly. Spiral fracture and or bar fracture.</td>
</tr>
</tbody>
</table>

It is important to observe that the definition of the standard visual damage state “Collapse” has been modified to better give an idea of a range of damage, in a similar way as to the other damage states. And, in this respect, the name for this visual damage state has been changed to “Near Collapse”. This change also explains that there is no standard description for it.

4.2.2 Other damage levels

The descriptions of damage explained above can be applied, enriched by additional considerations, not only for local (structural elements) but also for global (frames, structures in general) damage estimation.
In 1994, the Earthquake Engineering Research Institute (EERI 1994) adopted a scale that includes consideration of non-structural damage, the likely duration of loss of function, and risk of casualties to facility occupants:

A  None
B  Slight – minor damage to non-structural elements; facility reopened in less than 1 week
C  Moderate – mainly non-structural damage, little or no structural damage; facility closed for up to 3 months; minor risk of loss of life
D  Extensive – widespread structural damage; long term closure and possibly demolition required; high risk of loss of life.
E  Complete – collapse or very expensive, irreparable damage; very high risk of loss of life

This scale is essentially similar to that proposed by Park, Ang and Wen (1987), and already introduced in section 4.2.1.1, except that levels B and C take into account different minor damages, including those in non-structural elements.

The Vision 2000 document (1995) proposed, based on structural performance levels, the following damage levels:

Level 1: Fully operational. Facility continues in operation with negligible damage.
Level 2: Operational. Facility continues in operation with minor damage and minor disruption in non-essential services.
Level 3: Life safe. Life safety is essentially protected, damage is moderate to extensive.
Level 4: Near collapse: Life safety is at risk, damage is severe, structural collapse is prevented.
Note the extraordinary similitude here with the Park, Ang and Wen damage scale (see section 4.2.1.1), in the sense that the same words are used to describe and separate damage into different categories.

Vision 2000 can be considered an initial step to more advanced performance-based approaches. In particular, it can be argued that a "damage control" performance level, related to moderate damage, needs to be included. For example, Priestley et al. (2007) remarked that, although the performance in the 1995 Kobe earthquake of some reinforced concrete buildings satisfied the “life safety” performance level, the cost of repairing the many locations of limited damage was often excessive and uneconomical.

According to these ideas, Priestley et al. (2007) recommended the following structure limit states:

Serviceability Limit State: This corresponds to the “fully operational” seismic performance level of Vision 2000. No spalling of cover concrete should occur, and although yield of reinforcement should be acceptable at this limit state, residual crack widths should be sufficiently small so that injection grouting is not needed. This limit state can be directly related to strain limits in the extreme compression fibers of the concrete, and in the extreme tension reinforcement. Potential for non-structural damage must also be considered when determining whether or not the serviceability limit state has been exceeded.

Damage-Control Limit State: This is the basis for most current seismic design strategies. At this limit state, a certain amount of repairable damage is acceptable, but the cost should be significantly less than the cost of replacement. Damage may include spalling of cover concrete (requiring cover replacement), and the formation of wide residual flexural cracks requiring injection grouting to avoid later corrosion. Fracture of transverse or longitudinal reinforcement,
or buckling of longitudinal reinforcement should not occur, and the core concrete in plastic hinge regions should not need replacement. Again, non-structural limits must be considered to keep damage to an acceptable level.

Survival Limit State: Extensive damage may have to be accepted, to the extent that it may not be economically or technically feasible to repair the structure. Protection against loss of life is the prime concern.

Several attempts, besides MOTEMS, have been specifically made to relate visual damage states and performance levels in pile-deck connections. For instance, Stringer (2010) proposed three damage states based on the involvement and cost of repairing the damage. The first damage state is defined as initial cracking of the pile and spalling into the pile or the deck, but does not expose any reinforcing steel. In this case, the repair process is described as follows: remove spalled concrete, epoxy inject cracks, apply shotcrete, and coat repair. The second damage state is defined as pile or deck spalling that exposes reinforcing steel. The repair process then is: remove spalled concrete behind bars, paint rebar with zinc silicate, epoxy inject cracks, apply shotcrete, and coat repair. Finally, the third damage state is defined as spalling into the core concrete of the pile, or fractured reinforcing bars. The repair process for this is: use capital repair technique or replace with new pile adjacent to broken pile.

4.2.3 Brief background on damage indexes

Damage indexes provide a means to quantify the damage sustained by concrete structures during earthquakes. The earliest and simplest measures of damage were based on ductility and interstorey drift. These simple damage indicators, however, consider neither degradation in the stiffness of the member or structure, nor energy dissipation under cyclic loadings. Some of these simpler damage models are presented in Table 4.5.
Table 4.5 - Non-Cumulative Damage Indexes

<table>
<thead>
<tr>
<th>Model</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Based on deformations</strong></td>
<td></td>
</tr>
<tr>
<td>$D_{NR} = \frac{x_{\text{max}}}{x_y}$</td>
<td>Newmark and Rosenblueth (1974)</td>
</tr>
<tr>
<td>$D_1 = \frac{\phi_{\text{max}}}{\phi_y}$</td>
<td>Various authors (around 1980)</td>
</tr>
<tr>
<td>$D_{Br} = \frac{\phi_{\text{max, residual}}}{\phi_{u,\text{residual}}} = \frac{\phi_{\text{max}} - \frac{M_{\text{max}}}{(EI)<em>{\text{max, unloading}}}}{\phi_u - \frac{M_u}{(EI)</em>{u,\text{unloading}}}}$</td>
<td>Bracci et al. (1989)</td>
</tr>
<tr>
<td><strong>Based on actions</strong></td>
<td></td>
</tr>
<tr>
<td>$D_H = \frac{A_{\text{max}} - A_y}{A_u - A_y}$</td>
<td>Hasselman et al. (1980)</td>
</tr>
<tr>
<td><strong>Based on stiffness/flexibility loss</strong></td>
<td></td>
</tr>
<tr>
<td>$DR = \frac{(EI)<em>{0}}{(EI)</em>{\text{max}}}$</td>
<td>Lybas and Sozen (1977)</td>
</tr>
<tr>
<td>$FDR = \frac{(EI)<em>{\text{flexure}}}{(EI)</em>{\text{max}}}$</td>
<td>Banon et al. (1981)</td>
</tr>
<tr>
<td>$MFDR = \frac{\phi_{\text{max}}}{M_{\text{max}}} - \frac{\phi_y}{M_y}$</td>
<td>Roufaiel and Meyer (1987)</td>
</tr>
</tbody>
</table>

The content of Table 4.5 is ordered taking into account the parameter on which the damage model is based: displacement $x$, curvature $\phi$, actions (the literal A at $D_H$ represents shear and/or...
flexural moments), and stiffness/flexibility approaches. The subindices $y$ and $u$ represent actions and/or deformations at yielding and at ultimate, respectively. In addition, the subindex $max$ represents maximum deformations or actions at a monotonic load history. For instance, the damage index by Newmark and Rosenblueth is simply the ratio between maximum and yield displacement. In the special case of the damage index $D_{Br}$, it represents a ratio between curvatures. These are defined in the graph of moment vs. curvature as the ratio between the curvature at maximum and the curvature calculated using the slope of the descending branch at this point. The $DR$ damage index is the ratio between elastic and secant (at maximum curvature) slopes, again from a moment vs. curvature graph. Finally, the $FDR$ damage index is a modification of the previous damage index. The only difference is that $FDR$ does not include shear in the calculation of the elastic slope.

The importance of definitions for “yield” and “ultimate” is clear from the damage index descriptions above. They appear straightforward when the structural behavior is idealized as elastic-plastic and an equal displacement approximation is employed. However, there have been different interpretations about the appropriate definitions of yield and ultimate points when applied to realistic structural modeling. According to Priestley et al. (2007), with respect to force vs. displacements graphs, the yield displacement has variously been defined as the intersection of the line through the origin with initial stiffness and the nominal strength, the displacement at first yield, and the intersection of the line through the origin with secant stiffness through first yield and the nominal strength, amongst other possibilities. Ultimate displacement “also has had a number of definitions, including displacement at peak strength, displacement corresponding to 20% or 50% (or some other percentage) degradation from peak (or nominal) strength, and displacement at initial fracture of transverse reinforcement, implying imminent failure.” The
definitions employed in this dissertation will be explained when applied to the Park and Ang
damage index model.

Similarly, Tables 4.6a, 4.6b, and 4.6c show Cumulative Damage Indexes. They are
thought to be a better way, compared with the previous indexes, to represent damage produced
by cyclic or seismic load histories. They use the accumulation of force and displacement, as well
as potential and/or hysteretic energies, as parameters of damage. However, it should be noted
that the basic phenomena that is being represented here is that of low cyclic fatigue.

Table 4.6a - Cumulative damage indexes

<table>
<thead>
<tr>
<th>Model</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Based on cumulative deformations</td>
<td></td>
</tr>
<tr>
<td>[ D_{Ba2} = \sum \left</td>
<td>\theta_{i,\text{max}} - \theta_{y,i} \right</td>
</tr>
<tr>
<td>[ D_{SY} = \sum \left( \frac{x_{pp,i}}{x_{u,\text{mono}}} \right)^{1.8} ]</td>
<td>Stephens and Yao (1987)</td>
</tr>
<tr>
<td>Based on potential energy at different cycles</td>
<td></td>
</tr>
<tr>
<td>[ D_{Go1} = \sum_{i=1}^{n} \frac{F_{i}x_{i}}{F_{y}x_{y}} ]</td>
<td>Gosain et al. (1977)</td>
</tr>
<tr>
<td>[ D_{Go2} = D_{Go1} \left( 1 - \frac{d_{\text{core}}}{a} \right) \left( 1 + \frac{0.0005N}{A_{\text{core}}} \right) ]</td>
<td>Gosain et al. (1977)</td>
</tr>
<tr>
<td>[ D_{Ha} = 1 - \frac{E_{p,i}}{E_{p}^{0}} ]</td>
<td>Hanganu (1997)</td>
</tr>
<tr>
<td>[ D_{HSe} = \frac{(A_{y} - A_{n})}{A_{y}} ]</td>
<td>Hindi and Sexsmith (2001)</td>
</tr>
</tbody>
</table>
In Table 4.6, $D_{Ba2}$ accumulates plastic rotations, as the difference between maximum rotations at cycle $i$ and yielding, normalized with respect to the yield rotation. On the other hand, $D_{SY}$ accumulates plastic displacements. In the simplified form shown here, Stephens and Yao compared a residual displacement $x_{rp,i}$, defined as the displacement left when the force is zero in a cyclic load history, to the ultimate displacement $x_{u,mono}$ in a monotonic load history. This is an extension to reinforced concrete of similar and successful damage indexes used for steel structures. To apply this type of damage index, the hysteretic cycles should be relatively uniform and stable, which is the case for reinforced concrete elements controlled by flexural failure. $D_{Go1}$ accumulates potential energy, and $D_{Go2}$ modifies this previous index by taking into account additional factors like height of the section core $d_{core}$, area of the section core $A_{core}$, shear length $\alpha$, and axial force $N$. $D_{Ha}$ uses potential energy obtained from pushover analyses, $E_{p,i}$ is the potential energy obtained in a pushover analysis run after cycle $i$ (beginning with the residual displacement left at cycle $i$), and $E_p^0$ is the potential energy in the traditional monotonic analysis. The same idea, but with a different format, is given in $D_{HSe}$, which will be further explained later.
Table 4.6b - Cumulative damage indexes

<table>
<thead>
<tr>
<th>Model</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Based on cumulative hysteretic energy</td>
<td></td>
</tr>
<tr>
<td>$D_{Ba3} = \frac{\int_0^t M(\tau)\theta(d\tau)}{M_y \theta_y} / 2$</td>
<td>Banon et al. (1981)</td>
</tr>
<tr>
<td>$D_{HS} = \sum \Delta E_h K_n \Delta n^2 \quad \frac{\Delta n^2}{K_y \Delta_y^2}$</td>
<td>Hwang and Scribner (1984)</td>
</tr>
<tr>
<td>$D_{Dar} = \frac{E_h}{0.5F_y x_y \left[ 1 + \left( \frac{A'_x}{A_y} \right)^2 \right]}$</td>
<td>Darwin et al. (1986)</td>
</tr>
<tr>
<td>$D_{Oil} = \frac{E_h}{E_{h\text{max}}}$</td>
<td>Oller (1988)</td>
</tr>
<tr>
<td>$D_{Br2} = \frac{S_{SD} \int dE_h}{x_y \Delta F}; \quad S_{SD} = \frac{\Delta M\phi_y}{\int dE_h}$</td>
<td>Bracci et al. (1989)</td>
</tr>
</tbody>
</table>

In Table 4.6b, the numerator of $D_{Ba3}$, equal to $\int_0^t M(\tau)\theta(d\tau)$, is the energy dissipated up to time $t$, and the denominator, $\frac{M_y \theta_y}{2}$, is the maximum energy that can be stored elastically. In $D_{HS}$, $\Delta E_h$ is the energy dissipated during the $i$th cycle, $K_n$ is secant stiffness at displacement $\Delta_n$, and $K_y$ is yield stiffness at yield displacement $\Delta_y$. It compares potential energy at cycle $i$ with yield potential energy. However, this ratio is weighted by the hysteretic energy at each cycle. $D_{Dar}$ is a normalized dissipated energy index, where $E_h$ is the total dissipated energy, and the normalizing factors besides the maximum elastic energy are $A'_x$ and $A_x$, the areas of
compression and tension steel, respectively. $D_{oli}$ is simply the ratio between dissipated energy in a cyclic load history $E_h$ and the energy $E_{h_{max}}$ dissipated in a monotonic load history. In $D_{br2}$, $S_{SD}$ is a factor of strength deterioration, where $\Delta M$ is moment decay at first cycle, $\phi_y$ is yield curvature, and $\int dE_h$ is dissipated energy. Additionally, $x_y$ is yield displacement and $\Delta F$ is force decay at each cycle.

Table 4.6c - Cumulative damage indexes

<table>
<thead>
<tr>
<th>Model</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Based explicitly on number of cycles to failure</td>
<td></td>
</tr>
<tr>
<td>$D_{ChMS} = \sum_i \left( \alpha_i^+ \frac{n_i^+}{N_i^+} + \alpha_i^- \frac{n_i^-}{N_i^-} \right)$</td>
<td>Chung, Meyer and Shinozuka (1988)</td>
</tr>
<tr>
<td>$D_{fatigue} = \sum_i \frac{1}{(2N_f)_i}$</td>
<td></td>
</tr>
<tr>
<td>$(2N_f)<em>i = \left[ \frac{2aL_p}{(x</em>{max} - x_y) - d} \left( L - \frac{L_p}{2} \right) \right]^{\frac{1}{b}}$</td>
<td></td>
</tr>
<tr>
<td>$a, b = 0.08, 0.5 (Mander); 0.065, 0.435 (Kunnath)$</td>
<td>Chang and Mander (1994)</td>
</tr>
<tr>
<td>$L_p = 0.08L + 0.15 f_y db_l \ (f_y \text{ in ksi})$</td>
<td>Kunnath et al. (1997)</td>
</tr>
<tr>
<td>$(2N_f)<em>i = \left( \frac{8.25}{\frac{x</em>{max}}{x_y}} \right)^{4.0}$</td>
<td>Kunnath and Chai (2004)</td>
</tr>
</tbody>
</table>

The damage indexes that express low-cycle fatigue as number of load cycles to failure deserve special attention, since this is a direct approach to the definition of fatigue as the progressive
accumulation of damage up to the point of failure under repeated load applications. As an important example, in Table 4.6c, $D_{ChMS}$ modifies Miner’s hypothesis (damage equal to the summation of the following ratio: number of cycles actually applied at curvature level $i$, $n_i$ over number of cycles to cause failure at curvature level $i$, $N_i$) taking into account the load path (parameter $\alpha$) and the typically different response of a reinforced concrete element to positive and negative moments (where + and – are indicators of loading sense). Kunnath et al. developed an application of Miner’s hypothesis to reinforced concrete elements using standard analytical procedures, which include the calculation of the plastic hinge length, $L_p$, following the recommendations from Priestley et al. (2007); additionally, $d'$ is distance between extreme steel bars. In this fatigue damage index, $2N_f$ refers to a “complete” cycle (positive and then negative reversals). Further, Kunnath and Chai described $2N_f$ as a function of displacement ductility.

Among cumulative damage indexes, that from Hwang and Scribner (1984) has attracted relatively recent attention of researchers like Prakash and Belarbi (2010), and so it will be further studied here. The fatigue damage will also be compared, to estimate its applicability to describe damage in a pile-deck connection.

Finally Table 4.7 shows Cumulative Damage Indexes that are based on a combination of factors, such as ductility and inelastic energy.
Table 4.7 - Cumulative Damage Indexes based on a combination of factors

<table>
<thead>
<tr>
<th>Model</th>
<th>Author(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{BV} = \sqrt{d_1^* + d_2^*}$</td>
<td>Banon and Veneziano (1982) (ductility and inelastic energy)</td>
</tr>
<tr>
<td>$d_1^* = d_1 - 1; d_2^* = 1.1d_2^{0.38}; d_1 = \frac{x_{max}}{x_y}; d_2 = \frac{E_h}{E_s}$</td>
<td></td>
</tr>
<tr>
<td>$D_{PA} = \frac{x_n}{x_{\text{u,mono}}} + \beta \frac{E_{hn}}{Q_yx_{\text{u,mono}}}$</td>
<td>Park and Ang (1985)</td>
</tr>
<tr>
<td>$D_{Kun} = \frac{\phi_{max} - \phi_y}{\phi_{u,mono} - \phi_y} + \beta \frac{\int dE}{M_y\phi_{u,mono}}$</td>
<td>Kunnath et al. (1991) (Modified Park and Ang) (curvatures)</td>
</tr>
<tr>
<td>$D_{Daal} = \frac{\mu_i}{x_{\text{u,mono}}} + \beta \left( \sum (\mu_i - 1) \right)$</td>
<td>Daali (1995) (Modified Park and Ang) (residual plastic displacement)</td>
</tr>
<tr>
<td>$\mu_i = \frac{x_{ipi}}{x_{\text{y,mono}}} + 1$</td>
<td></td>
</tr>
<tr>
<td>$\mu_{u,mono} = \frac{x_{\text{u,mono}}}{x_{\text{y,mono}}}$</td>
<td></td>
</tr>
<tr>
<td>$D_{RV} = \frac{x_{\text{max}} - x_y}{x_{\text{u,mono}} - x_y} + 0.25 \frac{E_h}{(x_{\text{u,mono}} - x_y)F_y}$</td>
<td>Reinhorn and Valles (1995) (Modified Park and Ang) (fatigue formulation)</td>
</tr>
<tr>
<td>$D_{JE} = \frac{x_i}{x_{\text{u,mono}}} + 1 - \frac{x_i}{x_{\text{u,mono}}} \frac{F_{\text{a,mono}} - F_i}{F_{\text{a,mono}} - F_{\text{failure}}} \quad x_i \leq x_{\text{u,mono}}$</td>
<td>Jeong and Elnashai (2006)</td>
</tr>
<tr>
<td>$D_{JE} = \frac{x_i}{x_{\text{u,mono}}} \quad x_i &gt; x_{\text{u,mono}}$</td>
<td></td>
</tr>
</tbody>
</table>

In Table 4.7, the importance of the Park and Ang (1985) model should be highlighted, since almost all other indexes are modifications of this fundamental model. $D_{BV}$ is the exception. It compares two modified damage parameters $d_1^*$ and $d_2^*$, where $d_1$ is the ratio of the maximum displacement to the displacement at the elastic limit, and $d_2$ is the ratio of plastic dissipated...
energy to absorbed energy at the elastic limit. $D_{le}$, is based on the same ideas as the Park and Ang method too, but observing that since damage has two components, the second component is the complement of the first. However, it replaces the $\beta$ factor by assuming that the decay of the force between the monotonic $F_{o,mono}$ and the cyclic behavior $F_i$ has a limit $F_{failure}$ defined using a straight line between the origin and the force at monotonic ultimate displacement.

Among these damage indexes based on a combination of factors, the Daali (1995) model will be further studied, since its use of a fundamental parameter like residual plastic displacement looks promising for use in performance-based design. The Jeong and Elnashai model will also be studied, since it avoids calculation of the $\beta$ factor in the Park and Ang method, which could be considered relatively cumbersome for practitioners.

So, in summary then, Tables 4.5, 4.6a, 4.6b, 4.6c, and 4.7 give an idea of the various parameters related by researchers to damage. They also give an idea of the evolution of the damage indexes.

**4.2.4 Calibration of damage indexes against visual damage states**

Damage indexes, as explained in Section 4.2.3, use different parameters, such as plastic displacements, energies (especially hysteretic), and/or number of cycles to failure, to quantify damage in a structure. However, their outputs are actually just numbers whose relationship to damage still needs to be established. In mathematical terminology, this is to say that the “damage functional” still needs to be defined, although the normalized form of the damage functional would assume a value of 1, in the case of failure, and a value of 0, in the absence of any plastic damage. What is really needed, then, is its calibration for intermediate degrees of damage. An additional but important practical requirement is that it should be easily used and understood by practicing engineers. By those measures, the most successful damage index is that of Park and
Ang (DPA). Its authors developed a really simple form to relate the numerical values obtained with their index to observed damage. They supported their results with an extensive statistical database, which included 142 monotonic and 261 cyclic test specimens of beams and columns working under flexural conditions (Park and Ang 1985). The values that they and Wen obtained in their damage scale can be seen in Table 4.8.

### Table 4.8 – Park, Ang and Wen damage scale

<table>
<thead>
<tr>
<th>DPA</th>
<th>Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.1</td>
<td>No damage, or localized minor cracking</td>
</tr>
<tr>
<td>0.1≤DPA&lt;0.25</td>
<td>Minor damage: light cracking throughout</td>
</tr>
<tr>
<td>0.25≤DPA&lt;0.40</td>
<td>Moderate damage: severe cracking, localized spalling</td>
</tr>
<tr>
<td>0.4≤DPA&lt;0.8</td>
<td>Severe damage: concrete crushing, reinforcement exposed</td>
</tr>
<tr>
<td>DPA≥0.8</td>
<td>Collapse</td>
</tr>
</tbody>
</table>

#### 4.2.5 Comparison between Park, Ang and Wen vs. MOTEMS damage scales

The relationship between the Park, Ang and Wen damage scale and that from MOTEMS will be examined in this section. The first clear difference is the year that these damage scales were proposed: a difference of 20 years. Obviously, damage definitions have been substantially enriched during this period of time, to for instance include observations related to cracking, spalling, and crushing of concrete, as well as of yielding, buckling, and fracture of the steel reinforcement. Perhaps just as important, attention to non-structural damage, the cost of repair, and probable loss of life and operation time have now sometimes been included as well.

However, in the specific case of wharves supported on piles, non-structural elements such as walls, electrical or sanitary installations, and the like, do not exist or have very limited importance. Moreover, relatively few people work on these port facilities and for relatively limited amounts of time. These considerations leave structural damage as the most important
factor to be taken into account, which is exactly the case for the Park, Ang and Wen damage scale.

Based on the description of damage reviewed in previous paragraphs, it can be considered that MOTEMS Level 1 of structural performance is equivalent to the limit between Minor and Moderate Damage in the Park, Ang and Wen damage scale. In the case of MOTEMS Level 2 of structural performance, it can be considered to be inside the Severe Damage region defined by Park, Ang and Wen (see Figure 4.3).

![Figure 4.3 - MOTEMS vs. Park, Ang and Wen damage levels](image)

4.2.6 Park and Ang model

The Park and Ang model represents the culmination of research developed in the United States for the last three decades of the twentieth century. As explained previously, the initial estimators of damage were based on the ratio between the maximum plastic excursion and a reference value, usually related to steel yielding. This is logical, taking into account that the early experimental data was obtained from monotonic load histories. In a second stage, however, the researchers began to use cyclic tests. Therefore, their estimators of damage evolved, to the following form:

\[
\frac{\Delta_{\text{max, cyclic}}}{\Delta_{\text{u, monotonic}}} = \frac{X_{\text{max, cyclic}} - X_y}{X_{\text{u, monotonic}} - X_y} = \frac{\mu_{\text{cyclic}}^{-1}}{\mu_{\text{u, monotonic}}^{-1}}
\]  

(4.1)
Here, \( X \) can refer to displacement, drift, curvature, or rotation; \( \mu \) to ductility; the subindex \( \text{max}_c, \text{cyclic} \) refers to the maximum plastic deformation in a certain cycle; the subindex \( \text{u}, \text{monotonic} \) refers to the “ultimate” (maximum) value of deformation in a monotonic loading history, and finally, the subindex \( \gamma \) refers to yielding. The form of the equation above explains its “ductility” ratio name.

These initial damage indexes can be generalized in the following form:

\[
D = 0 \quad \text{with} \quad X_{\text{max}_c, \text{cyclic}} \leq X_\gamma
\]

\[
D = \alpha_1 \left( \frac{\Delta_{\text{max}_c, \text{cyclic}}}{\Delta_{\text{u}, \text{monotonic}}} \right)^\alpha_3 \quad X_\gamma < X_{\text{max}_c, \text{cyclic}} \leq X_{\text{u}, \text{monotonic}}
\]

However, the results of cyclic compared to monotonic tests revealed that structural elements tend to fail earlier in a cyclic than in a monotonic test. Structural damage, therefore, is being accumulated in each cycle. This phenomena is known as low-cyclic fatigue and is characterized by means of the hysteretic energy. This type of damage index can be represented in general as follows:

\[
D_F = \alpha_2 \left( \frac{E_h}{E_{h,u}} \right)^\alpha_4
\]

where, \( E_h \) is the dissipated hysteretic energy up a certain point in the load history and \( E_{h,u} \) is a reference or normalizing hysteretic energy at a certain point of the load history. Note that \( E_{h,u} \), if used as normalizing energy, is supposed to be fixed and not dependent on the load history, which has been proved experimentally to be not true. Moreover, it has been found that this energy term alone cannot adequately represent the full history of damage, since this is controlled by displacements at its onset.

One way to deal with this limitation was to assume that damage can be represented as the linear combination of energy and ductility terms:
\[ D = \alpha_1 \left( \frac{\Delta_{\text{max}}}{\Delta_{\text{u,monotonic}}} \right)^{\alpha_3} + \alpha_2 \left( \frac{E_h}{E_{\text{h,u}}} \right)^{\alpha_4} \]  

(4.4)

This equation is cumbersome and difficult to apply. Therefore, many attempts have been made to simplify it, and at the same time preserve its relative accuracy. The most successful damage index of this type is that of Park and Ang.

In this damage model, the structural damage index \((D_{PA})\) is represented as:

\[ D_{PA} = \frac{x_n}{x_{u,\text{mono}}} + \beta \frac{E_{hn}}{Q_y x_{u,\text{mono}}} \]  

(4.5)

in which \(x_n\) = maximum deformation at cycle \(n\); \(x_u\) = ultimate deformation under monotonic loading; \(\beta\) = parameter evaluated so that \(D_{PA}\) is equal to one at the point of cyclic failure; \(E_{hn}\) = absorbed hysteretic energy up to cycle \(n\); \(Q_y\) = calculated monotonic yield strength (if the strength at cyclic failure, \(Q_u\), is smaller than \(Q_y\), \(Q_y\) is replaced by \(Q_u\)). The points at monotonic and cyclic failure are evident if failure occurs suddenly, but in the case of a gradual failure, 20% reduction in the flexural monotonic and cyclic moment, respectively, defines these points.

It should be remarked that the definition of \(\beta\), as a parameter that is evaluated so that \(D_{PA}\) is equal to one at the point of cyclic failure, makes obligatory the evaluation of the Park and Ang parameters at the cyclic failure point. Note that this point is defined arbitrarily as corresponding to a 20% reduction in the flexural moment, as explained previously, if failure is gradual. One way to overcome this difficulty is to use preliminary recommended values that vary between 0.05 and 0.15. Park and Ang proved that this factor \(\beta\) is not arbitrary, and that it has statistical significance. As explained in a previous paragraph, a total of 142 monotonic and 261 cyclic test specimens (Park and Ang 1985) were evaluated to estimate the parameters in Equation
(4.5). With respect to $\beta$, they found that it has weak positive correlations with the shear span ratio, the longitudinal steel ratio, and the axial stress, as well as a negative correlation with the confinement ratio.

Taking into account the recommendations explained above, $D_{pa}$ is a relatively easy to use damage index, since it simplifies the calculations of both ductility and energy terms. Note also, that the reference hysteretic energy has been related to the yield strength, with the idea to operate algebraically with the energy term in a similar way as with the ductility term.

The Park and Ang model has shown extraordinary resilience over time and is still considered one of the most realistic measures of structural damage (Datta et al. 2008). Among its other advantages, it is a very flexible index which, with minor modifications, has been defined for an element, for a story, and even for overall buildings (Kunnath et al. 1992, William et al. 1995, Hindi et al. 2001).

As a final point, the use of the yield strength in the $D_{pa}$ equation forces one to clarify the definition of the yielding point: it is the intersection of the line through the origin, with secant stiffness through first yield, and the nominal strength. First yield is defined as the stage at which the steel closest to the tension face of the section begins yielding, or where the concrete at the compressed perimeter of the section reaches its peak unconfined strength, whichever occurs first.

This definition of the yielding point is commonly used since this assumes a bi-linear approximation to force-displacement (and to moment-curvature) response, and therefore enables direct relationships to be established between the displacement ductility and force-reduction factors. Also, once cracking occurs, the line from the origin to first yield provides the best estimate of elastic stiffness at levels close to yield.

According to the above definition, calculated yield strength is equal to nominal strength.
The nominal strength, in turn, is defined by an extreme fiber compression strain of 0.004 or an extreme tension reinforcement bar strain of 0.015, whichever occurs first. These strain limits are closely related to minor damage such as initial spalling consisting of flakes of concrete spalled off on the tension/compression sides of the structural element. The extreme concrete compression fiber refers to the fiber at maximum distance from the neutral axis. Similarly, the extreme tension reinforcement refers to the rebar furthest from the neutral axis.

4.2.7 Other damage indexes

The Park and Ang model is fundamental for the study of damage indexes in this dissertation. However, a better idea about its applicability to real structural damage progression could be given by comparing its results with those of other models.

The Hwang-Scribner and Hindi-Sexsmith damage indexes, correspond to methods based on cumulative energy. Hwang-Scribner is particularly important since it produces similar maximum amounts of normalized energy near failure for different load histories. This characteristic could be used to develop a scale of damage based on energy amounts. Hindi-Sexsmith pushover curves could potentially combine energy terms with ductility and low-cycle fatigue criteria. The third and fourth damage indexes for comparison, from Jeong-Elnashai and Daali, are grounded on combinations of factors, and they have promising characteristics that could be applied for performance-based design. Jeong-Elnashai extends traditional uniaxial flexural criteria to bidirectional and torsional responses. Daali damage index uses residual plastic displacements as one of its parameters, which is a relatively common parameter in performance-based design. Finally, a fifth damage index, Kunnath damage index, is important since it takes into account low-cycle fatigue criteria. It should be remembered that the analytical model developed in Chapter 3 used these criteria to accurately represent the structural behavior of the
steel material.

The first other model analyzed is that proposed by Hwang and Scribner, which is an energy approach as explained previously (see Table 4.6, and Equation (4.6)).

\[ D_{HS} = \sum \Delta E_h \frac{K_n}{K_y} \frac{\Delta_n^2}{\Delta_y^2} \]  

(4.6)

Where \( \Delta E_h \) is the hysteretic energy dissipated in cycle \( i \), \( K_n \) is the flexural secant stiffness in cycle \( i \), \( K_y \) is the flexural stiffness in the elastic range, \( \Delta_n \) is the maximum deformation in cycle \( i \), and \( \Delta_y \) is the deformation at first steel yield (see Figure 4.4 for additional clarification of these definitions of stiffness).

Figure 4.4 - Hwang and Scribner definitions for \( K_y, K_n, \Delta_n, \Delta_y \)
In its original form, the cycles that should be taken into account are those with strengths over 75% of the yield strength. This definition provided the authors a basis for consistent evaluation of the number of cycles to be taken into account from various experimental tests.

A detailed analysis of Eq. 4.6 shows that the fractional ratio represents the potential energy in each cycle normalized by the elastic potential energy; hence, it is really an energy index. Note that it could represent behaviors different than flexural response. In fact, this index was obtained from experimental work in which testing variables included not only cyclic displacement but also shear stress.

Clearly its principal disadvantage is that its range of values is not bounded. And furthermore, it depends too much on the cross-sectional property of the member and the loading history (Prakash et al. 2010).

However, the importance of the Hwang and Scribner index is that the hysteretic energy is normalized in such a way that different load histories produce similar maximum amounts of normalized energy near failure. So, in this dissertation, the Hwang and Scribner index will be further compared to its own value at failure (defined as 20% reduction in moment capacity). Note that, in this way, this particular damage index will have values between 0 and 1, and can also be compared to the Park and Ang damage index.

The second damage index examined in this section was proposed by Hindi and Sexsmith (2001), to model reinforced concrete bridge columns under cyclic loading. They suggested that, to measure the damage after any cycle, one can use the monotonic response to failure following that cycle. They also used an initial monotonic load-displacement curve up to failure as the reference curve. Their damage index is the following:

\[
D_{HSe} = \frac{(A_0 - A_n)}{A_0} \tag{4.7}
\]
where $D_{HSE}$ is the Hindi and Sexsmith damage index at cycle $n$ of the displacement history, $A_0$ is the area under the monotonic load-displacement curve up to failure, and $A_n$ is the area under the monotonic load-displacement history from the end of the last cycle $n$ to failure. It is worth to mention that the end point of a cycle is defined as the point at which the force is equal to zero, even though there still are residual displacements. At the beginning of the cyclic history, $A_n$ is equal to $A_0$; therefore, $D_{HSE}$ is zero. Eventually, after a number of cycles, $A_n$ would become zero, and then $D_{HSE}$ reaches its maximum value of one.

This method is conceptually simple, accumulative, and can potentially combine energy, ductility, and low-cycle fatigue. However, compared to the Park and Ang method, this damage index requires much more computational work, since for each loading cycle a pushover analysis is required. In addition, a very stable and mature analytical model is needed to avoid convergence problems over the wide range of analytical conditions arising at the various cycles. Last but not least, the main disadvantage is that it has not been calibrated extensively with experimental results.

The third damage index examined in this section is a low-cycle fatigue index. Since seismic loads induce several inelastic cycles at relatively large ductilities, the concept of using low-cycle fatigue theories to model damage is logical. Initially, Miner’s rule gives a relationship between damage and the sum of the ratio of number of cycles at a certain level of displacement to number of cycles to steel bar fracture at this level of displacement:

$$D_{fatigue} = \sum_i \frac{n_i}{(2N_F)_i}$$

(4.8)

where $D_{fatigue}$ is fatigue damage, $(2N_F)_i$ is the number of complete (push and pull) cycles to steel failure at a level of displacements $i$, $n_i$ number of cycles at a level of displacements $i$, $t$ is the total number of cycles in the load history. $D_{fatigue}$ is equal to zero at the first cycle, but
eventually grows up to one in successive cycles.

The number of cycles to failure at a level of displacements \( i \) is obtained from the Manson (1953)-Coffin (1954) relationship between fatigue life and strain amplitude \( \frac{\Delta \varepsilon}{2} \):

\[
\frac{\Delta \varepsilon}{2} = \frac{\sigma'_{p}}{E} (2N_{F})^{b} + \varepsilon'_{F}(2N_{F})^{c}
\]

(4.9)

where \( \frac{\Delta \varepsilon}{2} \) is strain amplitude, \( \sigma'_{p} \) is the steel stress corresponding to fracture in one cycle, \( E \) is the modulus of elasticity of the steel, \( \varepsilon'_{F} \) is the steel strain corresponding to fracture in one cycle, and \( b \) and \( c \) are constants to be obtained experimentally.

Kunnath et al. (1997), following a procedure developed by Chang and Mander (1994), obtained an experimental fit to the Manson-Coffin equation:

\[
\varepsilon_{p} = 0.065(2N_{F})^{-0.435}
\]

(4.10)

On the other hand, the plastic strain in the longitudinal reinforcement of a circular section \( \varepsilon_{p} \) can be related to the plastic curvature \( \varphi_{p} \) as:

\[
\varepsilon_{p} = \varphi_{p} \frac{d'_{p}}{2}
\]

(4.11)

where \( d'_{p} \) is the distance (center to center) of bars at opposite sides of the structural element.

Moreover, if the plastic curvature is assumed to be constant over a plastic hinge length \( L_{p} \), then the lateral plastic deflection at the end of a structural element in cantilever is:

\[
\Delta_{p} = \varphi_{p} L_{p} \left( L - \frac{L_{p}}{2} \right)
\]

(4.12)

where \( \Delta_{p} \) is plastic deformation (equal to max. displacement \( x_{\text{max}} \) minus yield displacement \( x_{y} \)), \( \varphi_{p} \) is plastic curvature, \( L_{p} \) is length of the plastic hinge, and \( L \) is the cantilever length. \( L_{p} \) is obtained from Priestley et al. (2003) as:

\[
L_{p} = 0.08L + 0.15 f_{y} l d_{l}
\]

(4.13)
where \( f_{yt} \) is the yield stress of the longitudinal reinforcement, and \( d b_t \) is the diameter of the longitudinal bars (the units are in the English unit system).

A relationship between the plastic deformation \( \Delta_p \) and the number of complete cycles to failure \( 2N_F \) can be found using these last four equations. Then, the initial damage index is transformed to the so called Kunnath et al. damage index \( D_K \):

\[
D_K = \sum_{i=1}^{n} \frac{1}{\left[\frac{2+0.065\Delta_p}{(X_{max}-X_y)-d\left(L-rac{L_p}{2}\right)}\right]^{0.435}}
\]

(4.14)

where all the terms in this equation have been previously defined.

This damage index, as stated by Kunnath et al., adequately represents the fatigue damage due to a high number of cycles with relatively small amplitudes. However, it is not adequate if there are a small number of cycles at relatively high ductilities, which is typical of earthquakes. Nonetheless, an alternative expression for \( 2N_F \) as a function of ductility, valid for circular columns, has also been proposed (Kunnath and Chai 1995):

\[
(2N_f)^{4.0} = \frac{8.25}{X_{max}^{X_y}}
\]

(4.15)

A fourth damage index is examined in the next paragraphs. Relatively recent studies have highlighted the importance of residual deformations in damage estimation. In fact, after an earthquake, these residual deformations are visible and can be used as an objective estimate of the structural behavior. Therefore, an additional effort will be made to check a damage index that takes residual deformations into account, specifically Daali’s damage index (see Table 4.6).
\[
D_{\text{Daa}} = \frac{x_m}{x_{u,\text{mono}}} + \beta \frac{\sum (\mu_i - 1)}{\mu_{u,\text{mono}}}
\]

(4.16)

\[
\mu_i = \frac{x_{rpi_i}}{x_{y,\text{mono}}} + 1 ; \quad \mu_{u,\text{mono}} = \frac{x_{u,\text{mono}}}{x_{y,\text{mono}}}
\]

The Daali model is a modification of the Park and Ang model. Its first term is the same as in the Park and Ang model, but in the second term the hysteretic energy parameters are replaced by residual plastic displacements \((x_{rpi_i} \text{ is the plastic residual displacement in cycle } i)\).

Finally, the Jeong and Elnashai damage index is examined. Total damage is defined as a combination of damage due to in-plane monotonic displacement and strength drop-off from the backbone envelope:

\[
D_{\text{JE}} = \frac{x_i}{x_{u,\text{mono}}} + \left(1 - \frac{x_i}{x_{u,\text{mono}}} \right) \frac{F_{o,\text{mono}} - F_i}{F_{o,\text{mono}} - F_{\text{failure}}} \quad x_i \leq x_{u,\text{mono}}
\]

\[
D_{\text{JE}} = \frac{x_i}{x_{u,\text{mono}}} \quad x_i > x_{u,\text{mono}}
\]

(4.17)

where \(x_i\) and \(x_{u,\text{mono}}\) are displacement at a peak response and the ultimate monotonic displacement, respectively. At a given displacement \(x_i\), \(F_{o,\text{mono}}\) and \(F_i\) are the forces on the monotonic and the cyclic response curves, respectively. \((F_{o,\text{mono}} - F_i)\), represents cyclic strength degradation, and \(F_{\text{failure}}\) is the corresponding failure strength. The failure strength is assumed to be a linear function of displacement that connects the origin and the ultimate monotonic displacement.

It can be seen that this damage index has a principal term related to displacements and a second term that is the complement of the first term. This last term is also weighted by a force factor. It can be concluded that the Jeong and Elnashai damage index is similar to the Park and Ang damage index, but it has avoided the calculation of the parameter \(\beta\).
4.2.8 Analytical comparison of damage indexes

Figure 4.5 shows the values obtained, using specimen UW-1, for the different damage indexes (examined at the pile-deck connection). As expected, DPA, DDaa and DJE are very similar, since they are based on the same basic ideas (combining displacements and energies). However, it should be noted that the energy term is typically very small compared to the displacement term. DHS and DHSe are also similar between them, since they are based on energy principles. Note that DHS shows an increment of damage at the same level of displacement, which may not be experimentally observed. DHSe, on the other hand, shows a gradual increment of damage. Finally, DK shows smaller values than the other damage indexes.

Figure 4.5 - Damage indexes as applied in Specimen UW-1
Figure 4.5 clearly shows these three groups of damage indexes. DPA, DDaa and DJE are the first group, DHS and DHSe are the second group, and finally DK is the only damage index in the third group. The most conservative of all of them is the first group, since its damage indexes are higher than the others at the same displacements. Note that the observed damage agrees more closely with this and the second groups. On the other hand, DK does not agree well with the observed damage and will not be considered further.

Similar results are expected for other specimens such as UW-2. Table 4.9 shows numerical based results for Specimen UW-2 using the Park and Ang and Daali (both representing damage indexes in the first group explained above), as well as the Hwang and Scribner (representing the second group) damage indexes. This table also shows the experimental series and the observed damage (in columns 1 and 2). They will help to confirm the similarity between the damage indexes in the first group, and any difference with the damage indexes in the second group.

**Table 4.9 - DPA, DDaa, and DHS damage indexes for Specimen UW-2**

<table>
<thead>
<tr>
<th>Series</th>
<th>Observed Damage</th>
<th>DPA Analytical</th>
<th>DDaa Analytical</th>
<th>DHS Analytical</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>None</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>3</td>
<td>None</td>
<td>0.00</td>
<td>0.00</td>
<td>0.000</td>
</tr>
<tr>
<td>4</td>
<td>None</td>
<td>0.01</td>
<td>0.01</td>
<td>0.000</td>
</tr>
<tr>
<td>5</td>
<td>Minor</td>
<td>0.05</td>
<td>0.03</td>
<td>0.001</td>
</tr>
<tr>
<td>6</td>
<td>Minor</td>
<td>0.08</td>
<td>0.06</td>
<td>0.002</td>
</tr>
<tr>
<td>7</td>
<td>Minor</td>
<td>0.11</td>
<td>0.08</td>
<td>0.01</td>
</tr>
<tr>
<td>8</td>
<td>Minor</td>
<td>0.13</td>
<td>0.11</td>
<td>0.01</td>
</tr>
<tr>
<td>9</td>
<td>Minor</td>
<td>0.16</td>
<td>0.14</td>
<td>0.02</td>
</tr>
<tr>
<td>10</td>
<td>Minor</td>
<td>0.19</td>
<td>0.16</td>
<td>0.03</td>
</tr>
<tr>
<td>11</td>
<td>Moderate</td>
<td>0.32</td>
<td>0.28</td>
<td>0.07</td>
</tr>
<tr>
<td>12</td>
<td>Moderate</td>
<td>0.50</td>
<td>0.44</td>
<td>0.15</td>
</tr>
<tr>
<td>13</td>
<td>Moderate</td>
<td>0.69</td>
<td>0.62</td>
<td>0.31</td>
</tr>
<tr>
<td>14</td>
<td>Severe</td>
<td>0.88</td>
<td>0.79</td>
<td>0.57</td>
</tr>
<tr>
<td>15</td>
<td>Severe</td>
<td>1.07</td>
<td>0.97</td>
<td>0.95</td>
</tr>
</tbody>
</table>
Table 4.9 confirms that DPA and DDaa are very similar. It also shows that DHS values at first are smaller than the other damage index values, but then they grow very fast. Note also that the results are higher than 0.8 (equivalent to theoretical collapse) for the last series in all three cases. This shows the degree of conservatism in the collapse definition used for these indexes (20% descent in moment capacity).

Table 4.10 is similar to Table 4.9, but shows the damage index results for Specimen UCSD-1.

<table>
<thead>
<tr>
<th>Series</th>
<th>Observed Damage</th>
<th>DPA Analytical</th>
<th>DDaa Analytical</th>
<th>DHS Analytical</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>None</td>
<td>0.07</td>
<td>0.06</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>None</td>
<td>0.10</td>
<td>0.09</td>
<td>0.04</td>
</tr>
<tr>
<td>3</td>
<td>None</td>
<td>0.14</td>
<td>0.12</td>
<td>0.08</td>
</tr>
<tr>
<td>4</td>
<td>Minor</td>
<td>0.21</td>
<td>0.19</td>
<td>0.18</td>
</tr>
<tr>
<td>5</td>
<td>Moderate</td>
<td>0.28</td>
<td>0.26</td>
<td>0.32</td>
</tr>
<tr>
<td>6</td>
<td>Severe</td>
<td>0.42</td>
<td>0.40</td>
<td>0.62</td>
</tr>
<tr>
<td>7</td>
<td>Severe</td>
<td>0.77</td>
<td>0.79</td>
<td>0.95</td>
</tr>
</tbody>
</table>

This table shows the good similarity between DPA and DDaa, as well as smaller values in the first series and then a big increment of these values in DHS. In this case, DHS even predicts collapse at the last series.

Table 4.11 is similar to Table 4.9 and Table 4.10, but shows the damage index results for Specimen UCSD-2. This table again shows the same damage behavior trends as described in Tables 4.9 and 4.10 for DPA, DDaa and DHS. In Figure 4.5, Table 4.9, Table 4.10, and Table 4.11, the Park and Ang damage index values vary quite remarkably, but their corresponding damage scale values still have a good agreement with the observed damage. Table 4.12 highlights this good agreement between observed damage and the Park, Ang and Wen damage.
Table 4.1 - DPA, DDaa, and DHS damage indexes for Specimen UCSD-2

<table>
<thead>
<tr>
<th>Series</th>
<th>Observed Damage</th>
<th>DPA Analytical</th>
<th>DDaa Analytical</th>
<th>DHS Analytical</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
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<tr>
<td>3</td>
<td>None</td>
<td>0.03</td>
<td>0.03</td>
<td>0.001</td>
</tr>
<tr>
<td>4</td>
<td>None</td>
<td>0.05</td>
<td>0.05</td>
<td>0.002</td>
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</tr>
<tr>
<td>6</td>
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<td>0.12</td>
<td>0.10</td>
<td>0.02</td>
</tr>
<tr>
<td>7</td>
<td>Minor</td>
<td>0.15</td>
<td>0.14</td>
<td>0.03</td>
</tr>
<tr>
<td>8</td>
<td>Moderate</td>
<td>0.23</td>
<td>0.21</td>
<td>0.07</td>
</tr>
<tr>
<td>9</td>
<td>Moderate</td>
<td>0.30</td>
<td>0.29</td>
<td>0.13</td>
</tr>
<tr>
<td>10</td>
<td>Severe</td>
<td>0.46</td>
<td>0.44</td>
<td>0.27</td>
</tr>
<tr>
<td>11</td>
<td>Severe</td>
<td>0.61</td>
<td>0.61</td>
<td>0.45</td>
</tr>
<tr>
<td>12</td>
<td>Severe</td>
<td>0.95</td>
<td>0.92</td>
<td>0.85</td>
</tr>
<tr>
<td>13</td>
<td>Collapse</td>
<td>1.21</td>
<td>1.26</td>
<td>1.00</td>
</tr>
</tbody>
</table>

In Table 4.12 observed and analytical Park, Ang and Wen damage scales are compared using specimens from the University of Washington (UW) and the University of California, San Diego (UCSD). These specimens are used because they constitute representative cases of some important differences in the pile-deck connection detailing, as well as of seismic and nonseismic detailing. They have been explained in detail in Chapter 3. Specimens UW-1 and UW-2 are different from UCSD-1 and UCSD-2 in the sense that minor damage is observed on these specimens before the analytical damage scale detects it. However, the end of the “minor” damage zone and beginning of the “moderate” damage zone coincides on both the observed damage and the analytical damage scale for UW-1, UW-2 and UCSD-1. In the case of UCSD-2, “moderate” damage is observed just before it is detected on the analytical damage scale.

In addition, in three of these specimens the analytical damage scale calculates “severe” damage before it is really observed. “Collapse” is also analytically calculated before it happens in three of these specimens. From these results, it is concluded that the analytical damage scale
and the observed damage coincide very well for the “moderate” damage region. They coincide fairly well for the “minor”, “severe”, and “collapse” damage ranges.

Table 4.12 - Observed damage and damage according to Park, Ang and Wen (Specimens from University of Washington and University of California at San Diego)

<table>
<thead>
<tr>
<th>UW-1</th>
<th></th>
<th>Park and Ang</th>
</tr>
</thead>
<tbody>
<tr>
<td>Series</td>
<td>Observed Damage</td>
<td></td>
</tr>
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</tr>
<tr>
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<tr>
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</tr>
<tr>
<td>4</td>
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<td>0.12</td>
</tr>
<tr>
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<td>Minor</td>
<td>0.15</td>
</tr>
<tr>
<td>6</td>
<td>Minor</td>
<td>0.18</td>
</tr>
<tr>
<td>7</td>
<td>Minor</td>
<td>0.21</td>
</tr>
<tr>
<td>8</td>
<td>Moderate</td>
<td>0.28</td>
</tr>
<tr>
<td>9</td>
<td>Severe</td>
<td>0.42</td>
</tr>
<tr>
<td>10</td>
<td>Severe</td>
<td>0.77</td>
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</table>

<table>
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<tr>
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<th>Park and Ang</th>
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</thead>
<tbody>
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<td>Observed Damage</td>
<td></td>
</tr>
<tr>
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<td>None</td>
<td>0.00</td>
</tr>
<tr>
<td>2</td>
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<td>0.01</td>
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<tr>
<td>3</td>
<td>None</td>
<td>0.08</td>
</tr>
<tr>
<td>4</td>
<td>None</td>
<td>0.12</td>
</tr>
<tr>
<td>5</td>
<td>Minor</td>
<td>0.15</td>
</tr>
<tr>
<td>6</td>
<td>Minor</td>
<td>0.18</td>
</tr>
<tr>
<td>7</td>
<td>Minor</td>
<td>0.21</td>
</tr>
<tr>
<td>8</td>
<td>Moderate</td>
<td>0.28</td>
</tr>
<tr>
<td>9</td>
<td>Severe</td>
<td>0.42</td>
</tr>
<tr>
<td>10</td>
<td>Severe</td>
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</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>UCSD-1</th>
<th></th>
<th>Park and Ang</th>
</tr>
</thead>
<tbody>
<tr>
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<td>Observed Damage</td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>None</td>
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<tr>
<td>2</td>
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<tr>
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<td>None</td>
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<td>5</td>
<td>Moderate</td>
<td>0.28</td>
</tr>
<tr>
<td>6</td>
<td>Severe</td>
<td>0.42</td>
</tr>
<tr>
<td>7</td>
<td>Severe</td>
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</table>

<table>
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<td>5</td>
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</tr>
<tr>
<td>6</td>
<td>None</td>
<td>0.12</td>
</tr>
<tr>
<td>7</td>
<td>Minor</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>Moderate</td>
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</tr>
<tr>
<td>9</td>
<td>Moderate</td>
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</tr>
<tr>
<td>11</td>
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<td>0.61</td>
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<tr>
<td>12</td>
<td>Severe</td>
<td>0.95</td>
</tr>
<tr>
<td>13</td>
<td>Collapse</td>
<td>1.21</td>
</tr>
</tbody>
</table>
The good agreement between observed damage and Park and Ang damage scale is definitively the main reason why this damage index is preferred over other relatively similar methods in this dissertation.

4.3 MOTEMS vs. Park and Ang damage values

In the previous paragraphs, it was observed that both MOTEMS and Park and Ang damage values are capable of describing different damage states in a pile-deck connection. After all, and even though they use different parameters, both of them try to describe the same damage behavior. Obviously they do not coincide (see Figure 4.6), but naturally a relationship between them can be explored.

![Displacement vs. Lateral Force at the pile-deck connection](image)

Figure 4.6 - MOTEMS and Park and Ang damage values for Specimen UW-1
As an example, Figure 4.6 shows the Park and Ang displacement-force points represented by circles, corresponding to the damage scale reference values: 0.1, 0.25, 0.4, and 0.8 (see also Table 4.8), as well as the displacement-force points obtained at MOTEMS damage levels 1 and 2, for Specimen UW-1. These last points are represented with diamonds. According to Figure 4.6, MOTEMS damage level 1 should correspond to a Park and Ang damage scale reference value between 0.1 and 0.25. Likewise, MOTEMS damage level 2 should correspond to a Park and Ang damage scale reference value between 0.25 and 0.4.

The comparison between MOTEMS and Park and Ang damage index shows that they are similar. A relationship between them can be determined as follows. First of all, the Park and Ang damage index is calculated at different displacements. In a cyclic load history, these displacements can coincide with the maximum displacement of each load cycle. The damage index – displacement points that are obtained will follow a relatively straight line, since the energy term in the Park and Ang damage index equation is very small. The equation of this straight line can be easily determined. Then, the displacements at which MOTEMS damage level 1 and 2 occur, are calculated. For example, the displacement at damage level 2 corresponds to the concrete or steel stress-strain curve reaching a strain equal to 0.025 or 0.05, respectively, whatever occurs first. Finally, the Park and Ang damage index that corresponds to these damage levels can be calculated using the damage index – displacement equation described above. Since, it is expected that different specimens will produce similar but not equal damage indexes for each MOTEMS damage level, an additional statistical analysis can be undertaken. This analysis consists of obtaining a damage index value with at least one standard deviation from the mean of all specimens. This is a conservative assumption, since in this way, the majority of cases will have the same damage but with higher damage indexes. The process to match MOTEMS
and Park and Ang damage index is described in detail in Figure 4.7.

Figure 4.7 (a) - Matching MOTEAMS and Park and Ang damage values

Figure 4.7(a) shows first how the Park and Ang damage index is determined for each series in a cyclic load history. Then, maximum displacement at a series vs. Park and Ang damage index points are fitted into a linear equation. Note that the format used in Figure 4.7 is suitable for use with programs such as MATLAB.

Figure 4.7(b) shows the determination of the analysis step at which MOTEAMS levels of damage 1 (L1) and 2 (L2), defined in terms of strains in both steel and concrete, are reached.
Figure 4.7 (b) - Matching MOTEMS and Park and Ang damage values

In Figure 4.7 (b), the steel strain is measured at the dowel supporting the highest stresses among all the dowels, and the concrete strain is measured at the point of highest stress along the perimeter of the pile (for damage level 1), and at the perimeter of the core concrete (for damage level 2). Once this analysis step is calculated, the corresponding crucial displacements, \( d(\text{criticalstep1}) \) and \( d(\text{criticalstep2}) \) or correspondingly \( \text{dMOTEMS L1} \) and \( \text{dMOTEMS L2} \), are easily obtained. Finally, using the linear equation considered in the previous paragraph, the Park and Ang damage indexes corresponding to MOTEMS damage levels 1 and 2 (\( D\text{Index Level 1} \) and \( D\text{Index Level 2} \))
and $D_{\text{Index Level 2}}$) are calculated.

This procedure is repeated for each specimen being analyzed. Then, as it can be seen in Figure 4.7(c), the general Park and Ang damage indexes, $D_{\text{Index L1}}$ and $D_{\text{Index L2}}$, appropriate to describe MOTEMS damage levels 1 and 2, are statistically calculated as the mean value of all specimens plus one standard deviation.

![Diagram](image)

*Figure 4.7 (c) - Matching MOTEMS and Park and Ang damage values*

### 4.4 Park and Ang damage index and simplified analytical models

In the specific case of the pile-deck connection design and rehabilitation, spectral modal analysis and/or earthquake time histories should be applied to first check the seismic demand on the pile-deck connection. For instance, guidelines such as MOTEMS recommend two levels of seismic demand (Operational and Contingency Level earthquakes). Once the force-displacement demand curve at the pile-deck connection is obtained, DPA can be calculated. Note that if the energy term is relatively small, for instance, 5% of the total DPA value, it can be neglected, and therefore the calculations simplified. The structure is considered to have an acceptable
performance under the operational level earthquake if DPA is less than 0.25. In the case of the contingency level earthquake, the same is true if DPA is between 0.25 and 0.4.

In case of seismic rehabilitation, DPA values less than 0.25 will confirm that the structural damage is minor (initial cracking and spalling of the cover concrete). Therefore, the repair process is similar to the one suggested by Stringer (2010). In the case of DPA values between 0.25 and 0.4, the cost of repairs should be significantly less than the cost of replacement. Finally, if DPA is higher than 0.4 and the structure apparently is still supporting loading, additional more detailed analysis (for instance, using fiber-based models) is recommended before any rehabilitation attempt.

The purpose of using the Park and Ang damage index, as an alternative to the description of limit states for a performance-based design, is to give elements of a relatively simple to apply damage assessment methodology for wharves supported on piles. Currently, in many commercial programs, the structural nonlinear behavior is modeled assuming concentrated plastic hinges at the ends of elastic elements, such as columns and beams. In Chapter 7, it will be shown that the Park and Ang damage index calculated using these simplified nonlinear models is still a good approximation for describing the damage states.
CHAPTER 5: PILE SUPPORTED WHARF MODEL

In this chapter, the pile-deck connection developed in Chapter 3 is used in an analytical model that includes the full length of the piles and therefore soil-interaction and in-ground hinge effects.

Wharf structures are currently being designed by performance based criteria: acceptable displacements are based on amount of damage under expected seismic load levels. The assessment of the pile-deck connection lateral displacement is crucial in this type of design, but it could be influenced by the existence of an in-ground hinge and the resulting damage sequence at the head and body of the pile. It should also be remembered that the maximum axial force in the pile is controlled by the capacity at the pile head. This is so because, under a seismic load, even though the moment distribution at the deck will be always linear, its magnitude varies and hence the axial force induced at the piles. Since the moments at the head of the piles cannot go beyond their moment capacity, the maximum axial force is also limited. However, the varying axial forces could have an effect on the pile capacity. For this reason, it was deemed necessary to include in the analysis the full length of at least two piles where damage was expected to be more severe. Note also, that this model will be used, in the next chapter, in a detailed parametric study of crucial elements in the pile supported wharf structure. A schematic illustrating the structural behavior explained above will be presented in Chapter 6, too.

Performance-based design intends to give a more uniform capacity/demand ratio under different forces that ultimately could cause the wharf to collapse. An initial knowledge of these forces is necessary and therefore will be addressed at the first pages of this chapter. Then the key elements of the performance-based design of pile-supported wharves will be presented. In addition, crucial structural elements will be identified based on the literature review. Built on this
information, a simplified analytical model will be presented and described. Finally results such as moment versus displacement curves at the pile-deck connection of typical wharves will be shown.

5.1 Analysis and design of wharf structures

Strong lateral earthquake forces could produce plastic hinging in the piles supporting the wharf structures. These hinges are typically localized in the pile-deck connection zone, however they could potentially develop in ground. These forces could also trigger other phenomena such as settlement of the wharf, lateral ground deformation, slope instability, and most important, liquefaction of the underlying and/or retained soil (Borg, 2007). This last phenomenon once triggered by seismic loads, activates in turn settlement, deformation and soil instability. Figure 5.1 shows the probability of failure modes for wharf structures, including if there are pile caps, steel piles and retaining walls.
In Figure 5.1, the probability of failure of the pile-deck hinge is around 10%, being overpassed only by “settlement of the structure” (12%) and “seaward displacement” (13%). Note that settlement of the structure and seaward displacement are commonly triggered by soil liquefaction. There is nothing in this dissertation that really directly considers liquefaction, but rather things which simply improve the connection region performance in general will most likely be helpful in cases of liquefaction as well. On the other hand, the probability of failure of the pile-deck hinge is 2.5 times higher than the probability of failure of the in-ground hinge and around 1.5 higher than deck cracks. Although Figure 5.1 shows clearly the importance of the

**Figure 5.1 - Probability of failure of wharf structures (source: Borg 2007)**
pile-deck connection in a wharf supported on piles, other studies (Goel 2010-a, 2010-b) stress on the importance of in-ground hinges. Hence there is a debate about the importance and interaction between the plastic zone at the pile head and the in-ground plastic hinge. Thus, this dissertation will focus on clarifying these interactions too.

It is understood that the behavior of the pile-deck connection, which is the principal study goal of this dissertation, could be affected by in-ground phenomena. Soil properties are the most important factors that govern these phenomena. However, they are inevitably subjected to some uncertainty. This fact coupled with liquefaction probability makes the soil-structure model demanding. The approach followed in this dissertation to deal with the soil-structure interaction is to use nonlinear p-y soil springs. The sensibility of the results to the soil characteristics is checked using maximum and minimum soil values recommended by MOTEMS (2007).

Figure 5.2 shows wharf layout and cross-section of Berth 100 in POLA, which was built in 2004.
a) wharf layout
b) wharf cross-section

Figure 5.2 - Pile-supported wharf layout and cross-section

It can be seen in the wharf layout in Figure 5.2 (a) that in order to support the crane leg vertical reactions, spacing between piles in rows A and F is reduced to one third of the regular spacing used elsewhere. The cross-section in Figure 5.2 (b), on the other hand, helps in clarifying that the most rigid elements are the two rows of piles at the land side of the wharf. They would attract the majority of the forces applied on the wharf. To reinforce this zone, as can be observed again in the pile layout, the second row of piles (from the land side, at the bottom of the graph, Figure 5.1 (b)) has one additional pile in between the regular piles. The need to support the crane reactions explains also the use of comparatively close-spaced smaller diameter piles than fewer number of
larger diameter piles. In the seismic areas of the US west coast, 610 mm (24 in.) diameter octagonal prestressed piles are most common (Priestley et al., 2007). Nonetheless, in modern container wharves the overall wharf width is also dictated by details of the container cranes used to service the ships. Typically modern container cranes have a rail gauge of about 30 m (100 ft), and a capacity between 1500 and 2000 tones.

As an example of wharf design, Figure 5.3 shows initial and final design cross sections presented by Priestley et al. (2007).

Figure 5.3 - Initial and final design (Priestley et al. 2007)
A characteristic module length of 6m (19.6 ft) is used initially for all the wharf bays. However, in the final design, the row E is moved closer to the row F, and the clearance between the top of the dyke and the deck soffit is increased. The first change increased the seismic resistance of row E, and the second change increased considerably the limit-state displacements even though it reduced the lateral shear that can be resisted by row F.

The above example shows how the geometry of the wharf structure is dictated by performance-based criteria. Key additional elements that affect directly the wharf geometry, at the section and at the structure level, besides crane loading and soil-structure interaction, are:

- The frame built by the deck and the piles should be ductile and moment-resisting.
- Strong beam (deck), weak column (piles) proportioning should be used.
- Vertical piles (as opposed to inclined piles) should be employed.
- 3-D effects should be considered, since the wharf is highly eccentric.

It is expected that crucial structural points such as the pile-deck connections, once reached their highest capacity, should be capable of resisting considerable moments at higher rotations and displacements. Using energy terms, it is expected that the seismic energy would be dissipated by ductile yielding at plastic regions. Due to the thick deck needed to support the crane reactions and the container weights, the weak elements, at which damage will concentrate, are the piles. It is also recommended to only use vertical and not inclined piles, since this last type of structural element tends to attract excessive lateral loads. Finally, it should be remembered that in a wharf, the center of rigidity is significantly apart from the center of mass, and therefore, there are important torsional 3-D effects.

Figure 5.4 shows both seismic and non-seismic piles supporting wharf structure. As explained in Chapter 1, due to the rigidity of seismic piles, they sustain larger seismic forces;
hence their design requirements and restrictions are more severe than non-seismic piles.

![Figure 5.4 - Seismic and non-seismic piles](image)

Analytical models have shown also the sequence of expected hinge formation as can be seen in Figure 5.5.

![Fig. 5.5 - Sequence of expected hinge formation. Source: Arulmo (2008)](image)

In this particular example, the first two piles from left to right suffer extensive damage, both at
the head and at the body, before the third pile is subjected to damage too. The first hinges should occur at the head of the piles because it is visually observable and accessible for repairs, and the occurrence of the hinges should occur at different times showing the capacity of the structure to redistribute stresses.

5.1.1 Structural effect of in-ground hinges

Figure 5.6 shows the typical distribution of moments along the length of the pile at different maximum horizontal displacements at the pile head. Note that the ground level is at depth 0.0 m, and the soffit of the deck at +1.0 m. In this figure, clearly two zones are susceptible to the highest moments and also related flexural damage: the pile-deck connection and a region at a depth inside the soil approximately equal to 2.4 m. (94 in.).

Figure 5.6 - Distribution of moments along the length of the pile (1 kN=.22 kip; 1m=3.28 ft)

Note that the soil layers should have an impact on the shape of this curve (moment vs.
position along the length of the pile), since they have usually different stiffness. In this specific case, three types of soil layers have been considered, from top to bottom: rock fill (5 m), marine sand (8 m), and lagoonal clay (7 m). Their properties are summarized in Section 5.2.1. Note that the rock fill, even though apparently more rigid, is atop of the other soil layers, and therefore has less influence of the depth into its stiffness. That means that eventually the deeper soil layers would have similar stiffness than the rock fill. In the case of the stiffness differences between the marine sand and the lagoonal clay, it should be noted that significant moments occur entirely inside the zone of the marine sand and therefore they do not affect the shape of the moment curve. All these characteristics explain the relatively smooth change in moment along the length of the pile observed in Figure 5.6.

In any case, damage at the region inside the soil would produce an “in-ground” hinge. Concrete and prestressed steel would reach their peak stress values at around the same strain increment. Concrete stresses would descend thereafter, but prestressed steel stresses would remain relatively constant up to the point of failure. Therefore, heavy spiral reinforcement is used to improve the concrete stress-strain curve and to match the demand produced by the steel capacity. The distance between the pile deck connection and this probable damage zone in the body pile is usually less than 5 m. (16.4 ft.), small enough to make possible the use of the same pile-deck spiral reinforcement along this entire length.

Nevertheless, displacements at the pile head are controlled by the rotation capacity of the pile-deck connection or the in-ground zone of maximum moments, whichever is smaller. In a typical design, such as that in Berth 100 at POLA, the pile-deck connection has less rotation capacity than the in-ground zone, and thus damage there is accessible and repairable. The sequence of damage will be further explained in numeral 5.3.
Goel (2010-a, 2010-b) presented a simple model which could help to evaluate the interaction between the pile-deck connection and the in-ground hinge behavior. A comparison with the model developed in this dissertation will highlight eventual limitations of both models. Goel’s model represents the pile as a cantilever with a length such that the displacement at the pile head is equal to that of the real structure. The in-ground hinge is assumed to develop at the point of fixity. A rotational spring is used at the pile head to represent the expected nonlinear behavior, see Figure 5.7.

Figure 5.7 – Simplified model of the pile with connection to the deck
(source: Goel 2010-b)

The rotational spring is modeled using a rotation vs. moment curve. In this curve, the moment values are obtained through equilibrium of the stresses developed in a deformed pile section complying with the hypothesis that plane sections remain plane during loading. Concrete is assumed to have a constant stress equal to 1.7 $f_c$. Steel uses an elastic perfectly plastic strain-stress curve. Finally, rotations in this moment versus rotation curve are calculated as the ratio
between the elongation of the outermost dowel ($\Delta L_1$) and the distance between the neutral axis and the outermost dowel ($y_1$). $\Delta L_1$ is calculated as the strain at the outermost dowel times the length of the plastic hinge. The plastic hinge length is calculated following recommendation from Priestley et al. (2007). The rotation versus moment curve at the head of the pile can be further idealized as elastic-perfectly plastic.

The displacement at the head of the pile under lateral loads can be obtained adding the contributions from the different body segments of the pile, which are influenced by the moment capacity at the pile head. The contribution from the pile body is calculated using the curvature versus moment curve at the section level. It can be idealized as bilinear curve but taking into account hardening effects. The resulting displacement versus force curve at the head of the pile has an initial linear slope, until yielding of the steel at the pile-deck connection is reached. It continues with a softer linear slope until yielding of the steel in the hinge inside the pile, the in-ground hinge, is reached. After this point, the linear slope is much softer. This is the expected behavior. However, it is possible that the in-ground hinge reaches yielding before the pile-deck connection. In Goel et al. model, it depends on the length of the in-ground plastic hinge. Additional observations about this model can be found in section 5.3.

### 5.2 Analytical model

The principal goal of the analytical model is to capture the effects of in-ground hinges on the displacement capacity of the wharf, and the effect of varying axial loads on the pile head capacity. In addition, the analytical model should be as simple as possible. As another equally important goal, this analytical model must be useful for a parametric study of the pile-deck connection.

Taking into account the constraints described above, the wharf is modeled in 2D. It
consists of the substructure built by the three most land side piles, the soil around them, and the
derk at top of the three piles. The geometric dimensions of the model are such that the bay width
between the two initial piles is 4.0 m (13.12 ft.), since it is common to use reduced bay lengths
between these piles, and 6.8 m (22.3 ft.) for the additional bay length. The clearance between the
top of the dyke and the deck soffit is selected as 1.0 m (3.28 ft.). The section of the piles is
octagonal with 610 mm (24 in.) diameter. The sectional dimensions of the deck are 6.0x0.9
meters (19.7x3.0 ft.) between the first two piles, and 3.2x0.6 meters (10.5x2.0 ft.) beyond the
second pile. These values are estimated assuming that the deck behaves rigidly under lateral
loads. The length of the piles, 21 m (69 ft.), was found to be sufficient to obtain zero moments
and deformations at the pile base.

Figure 5.8 - General sketch of the crucial wharf substructure (1m=3.28 ft)

Figure 5.8 shows a general sketch of the analytical simplified model. Each pile has been divided
into elements with a length of 0.25 m (10 in.), to capture the material sectional variation and the
effect of the soil. The sections are typical of prestressed concrete piles. At the head of the pile,
however, care has been taken to model the presence of dowel bars and the development length of
the dowel bars and prestressed steel strands. At the top of each pile a rotational spring was
included to model the dowel slip. The zone between the soffit and the neutral axis of the deck
was modeled using a rigid link that connects the rotational spring to the deck. Finally, the deck is
modeled with a horizontal typical frame element at the neutral axis of the deck. It assumes that
the deck will behave linearly elastic and more like a rigid body due to its dimensions. This is also
the reason to represent the structural element connecting the soffit of the deck and its neutral axis
with a rigid link.

Figure 5.9 shows a sketch of the different materials that were used at the head of the
piles. This figure shows five columns. The first two columns are related to the characteristics of
dowels and prestressed steel strands. Columns four and five, on the other hand, are related to
cover and core concrete characteristics, which are the same for all structural elements. Column
three indicates the length of each element. The variation in the steel shading shows the degree at
which the steel materials are capable of working: a strong color represents more capacity;
conversely, a weak color represents less capacity. The capacity at each section depends on the
development length of dowel and prestressed steel. The crucial elements can be distinguished by
column 3 shading, too. The most crucial elements are the first two at the head of the pile, since
the development of a nonlinear region is expected there. Finally, note that dowels work only in
the first seven structural elements.
It is important to highlight again that the effect of slip is modeled independently with a rotational spring introduced at the head of the pile.

5.2.1 Soil-structure interaction

The soil has been represented in this model with nonlinear soil springs. Lateral soil resistance is represented using “p-y” springs, axial pile-soil friction through “t-z” springs and end bearing through a “Q-z” spring. The shape of the curves force-displacement obtained for these springs can be seen in Figure 5.10.
Fig. 5.10 – Shape of the curves force-displacement obtained for the springs “p-y”, “t-z”, and “Q-z”

The initial parameters of the p-y spring, namely ultimate capacity and displacement at which 50% of the ultimate capacity is mobilized in monotonic loading (d50), were modeled using the recommendations of the American Petroleum Institute (API) (1987) which also specifies a linear variation of initial stiffness with depth. A detailed description of the cyclic p-y behavior adopted in this work can be found in Boulanger et al. (1999). The t-z springs were defined based on the work of Mosher (1984) and Kulhawy (1991). The Q-z behavior was based on the work of Meyerhof (1976), Vijayvergiya (1977), and Kulhawy and Mayne (1990). These springs were implemented in OpenSees using the example prepared by Christopher McGann (2010) and available at OpenSeesWiki (www.opensees.org). Figure 5.11 shows a scheme of the
The nonlinear soil springs could represent very well the expected soil behavior, if not subjected to extreme cases like liquefaction. In addition, the distribution and type of soil layers vary depending on the site characteristics of each wharf. In an effort to take into account a realistic soil distribution, at least in one of the most important ports in the West Coast of the United States, the soil characteristics of Berth 100 in POLA were used in this work.
Figure 5.12 shows that three important types of soils can be recognized in Berth 100 and in this work. First quarry run fill, then marine sand and finally clay-silt (lagoonal clay), all of them on top of the Lakewood-San Pedro Formation constituted by sand to silty sand.

The parameters used to define the upper two soil types are presented in Table 5.1.
In addition to length of the sandy soil layers, unit weight and friction angle, Table 5.1 shows clearance between the deck and the ground. It should be emphasized again the general nature of these values, intended to be an approximation to realistic soil conditions in port sites.

Finally, the lagoonal clay layer has been considered to have a mean ultimate capacity of 2270 kN/m (155 kip/ft.) and a displacement d50 of 0.01 m (0.4 in.). In addition, the backbone of its p-y curve approximates Matlock (1970) soft clay behavior. Further, the shear modulus at the tip of the pile, needed for the Q-z spring, is considered to be equal to 150 MPa (21.8 ksi).

### 5.3 Structural behavior of typical cases

The two pile rows at the land side of a typical wharf have different number of piles than the other rows. This affects the stiffness of these two rows in a 2D model. In the case of Berth 100 in POLA, the stiffness of the landside pile was multiplied by three and the stiffness of the next pile by two.

Once the general geometry of the analytical model was defined, the materials required closer attention. Extreme values of different material parameters could be used to estimate the

---

**Table 5.1 - Initial parameters for sandy soil types**

<table>
<thead>
<tr>
<th></th>
<th>Length [m (ft.)]</th>
<th>Soil unit weight, [kN/m³ (kip/ft³)]</th>
<th>Friction angle [degrees]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pile 1</td>
<td>Pile 2</td>
<td>Pile 3</td>
</tr>
<tr>
<td>Clearance</td>
<td>1.0 (3.3)</td>
<td>1.0 (3.3)</td>
<td>5.5 (18.0)</td>
</tr>
<tr>
<td>Rock Fill</td>
<td>5.0 (16.4)</td>
<td>6.0 (19.7)</td>
<td>3.5 (11.5)</td>
</tr>
<tr>
<td>Marine Sand</td>
<td>8.0 (26.2)</td>
<td>4.0 (13.1)</td>
<td>5.0 (16.4)</td>
</tr>
</tbody>
</table>
behavior of the wharf. However, typical values are also needed to establish an initial base for comparison. The materials in Berth 100 in POLA, even though there are some data gaps, will be used to obtain such typical structural behavior.

The materials used for Berth 100 in POLA are presented in Table 5.2

Table 5.2 - Materials used in Berth 100

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. $f'c$</td>
<td>72000 kN/m² (10.4 ksi)</td>
<td>Expected concrete strength</td>
</tr>
<tr>
<td>2. $f_yh$</td>
<td>607000 kN/m² (88 ksi)</td>
<td>Expected yield stress of spiral strand</td>
</tr>
<tr>
<td>3. strand</td>
<td>16-0.6 in. (16 mm) diameter (3.47 in.²)</td>
<td>Prestressed steel</td>
</tr>
<tr>
<td>4. Steel ratio (=Asl/Ag)</td>
<td>0.02 (9 in.²)</td>
<td>Long. reinforcement (Dowels)</td>
</tr>
<tr>
<td>5. $P_s$ (=4<em>Ash/(Ds</em>s))</td>
<td>0.018</td>
<td>Spiral reinforcement ratio</td>
</tr>
</tbody>
</table>

Table 5.2 shows relatively high quantities of longitudinal and spiral reinforcement. Prestressed steel is also important and has been included, with the exception of the strand number, with the same details as in specimen UW-1 (Chapter 3). Since the expected concrete strength value was not available, a value of 72000 kN/m² (10.4 ksi) was selected similar to that used in UW-1.

In the initial vertical load calculation, self-weight D resulting from the deck and 1/3 of the pile weight between its head and a point at a depth inside the ground 3 times the pile diameter was considered. Temporal or live loads $L$ was assumed to be 100 psf. This value corresponds to current practice for container wharf piles as described by Arulmoli (2008). This is also consistent with POLA (2004) and POLB (2009) recommendations regarding additional live load to be considered as seismic mass: 10% of design live load, not to exceed 100 psf. Minimum
and maximum vertical loads were calculated using the following formulas (MOTEMS 2007):

Minimum vertical load = $(0.9 - \text{vertical seism}) \times D$ \hspace{2cm} (5.1a)

Maximum vertical load = $(1.2 + \text{vertical seism}) \times D + L$ \hspace{2cm} (5.1b)

In these equations, vertical seism is half the effective peak ground horizontal acceleration, expressed as a fraction of the gravity acceleration. In this work, a value of 0.25 was used for vertical seism since this is a typical value for the US West Coast.

Since the axial forces were relatively low for both minimum and maximum levels, only the maximum level was used in the following analytical results.

The envelope curves for moment versus displacement at the head and body of pile 1 (landside) and pile 2 (waterside) are presented in Figure 5.13.

![Figure 5.13 - Moment versus displacement envelope curves for a typical case](image)

*(500 kN-m = 369 kip-ft., 0.1m = 3.94 in.)*
The term “body”, in Figure 5.13, refers here to the zone into the body of the pile with highest moments, approximately at 3.75 m (12.3 ft.) below the deck soffit. The star symbol in these curves represents MOTEMS damage level 1. The circle symbol represents MOTEMS damage level 2. It can be noted that both levels of damage occur at similar displacements in both pile 1 and pile 2 heads. In the case of the pile body, only the first level of damage is reached in both piles. It should be noted that MOTEMS damage level 1 is related to moderate damage such as initial cracking and cover spalling. This is not the expected damage in a plastic hinge, meaning that at moderate damage only the hinge initial plastic zone of behavior is reached. On the other hand, the damage at the pile head reaches level 2. That means that the damage is relatively severe. Evidently the crucial structural zone is the pile-deck connection.

Note that pile 1 and 2 have varying axial forces. In pile 2, they are higher at positive displacements (displacements towards sea), and lower at negative displacements (displacements towards land). The opposite is true for pile 1. This is also illustrated in Figure 5.13, where the moment capacity of pile 2 is higher than pile 1. This is attributed to the higher axial force applied on pile 2 at positive displacements.

The model also shows a sudden descend in the pile 2 capacity at a displacement around 0.15 m (6 in). From an analytical point of view, this shows difficulties to reach equilibrium in the structural system. From a phenomenological point of view, this is related to severe damage and collapse.

To highlight some numerical details, selected moment vs. displacement data shown in Figure 5.13 is presented again in Table 5.3. Note that only positive displacements are taken into account to avoid an excessive amount of data.
The first column in Table 5.3 shows lateral displacements. The second and third columns show moments at the head and body of pile 1, respectively. The fourth and fifth columns in Table 5.3 show moments at the head and body of pile 2, respectively. Table 5.3 shows that pile 2 has higher capacity than pile 1 not only at the pile-deck connection but at the body zone also.

Table 5.4 shows curvatures at the head and body of pile 1 at different positive displacements.
<table>
<thead>
<tr>
<th>Displacement</th>
<th>Curvature1H</th>
<th>C1H/C1Hy</th>
<th>Curvature1B</th>
<th>C1B/C1By</th>
<th>H/B</th>
</tr>
</thead>
<tbody>
<tr>
<td>[m (in.)]</td>
<td>[1/m (1/in.)]</td>
<td>[-]</td>
<td>[1/m (1/in.)]</td>
<td>[-]</td>
<td>[-]</td>
</tr>
<tr>
<td>0.013</td>
<td>-0.0051</td>
<td>0.31</td>
<td>0.0012</td>
<td>0.16</td>
<td>1.93</td>
</tr>
<tr>
<td>0.026</td>
<td>-0.0081</td>
<td>0.49</td>
<td>0.0029</td>
<td>0.39</td>
<td>1.27</td>
</tr>
<tr>
<td>0.039</td>
<td>-0.0106</td>
<td>0.64</td>
<td>0.0046</td>
<td>0.61</td>
<td>1.05</td>
</tr>
<tr>
<td>0.052</td>
<td>-0.0146</td>
<td>0.88</td>
<td>0.0064</td>
<td>0.85</td>
<td>1.04</td>
</tr>
<tr>
<td>0.065</td>
<td>-0.0257</td>
<td>1.56</td>
<td>0.0081</td>
<td>1.08</td>
<td>1.44</td>
</tr>
<tr>
<td>0.098</td>
<td>-0.0605</td>
<td>3.67</td>
<td>0.014</td>
<td>1.87</td>
<td>1.96</td>
</tr>
<tr>
<td>0.130</td>
<td>-0.1195</td>
<td>7.24</td>
<td>0.0172</td>
<td>2.29</td>
<td>3.16</td>
</tr>
</tbody>
</table>

The first column in Table 5.4 shows lateral displacements. The second and third columns show curvature at pile 1 head (C1H), and the curvature/curvature at yield ratio (C1H/C1Hy), respectively. The fourth and fifth columns in Table 5.4 show curvature at pile 1 body (C1B), and the curvature/curvature at yield ratio (C1B/C1By), respectively. The last column in Table 5.4 compares head and body curvatures. The yield curvature at pile 1 head for positive
displacements is \(-0.0165 \, \text{m} \, (0.0004 \, \text{in})\), and is \(0.0075 \, \text{m} \, (0.0002 \, \text{in})\) at the body of the same pile. Table 5.4 shows that curvature at the pile head is higher than at the pile body. Therefore, the expected damage is more severe there.

Figure 5.14 shows displacement versus curvature relative to yielding at the head and body of pile 1 curves (columns 3 and 5 in Table 5.4).

\[\text{Curvature Ratio at the Head and Body of Pile 1}\]

Figure 5.14 - Displacement versus relative curvature for a typical case (0.1m = 3.94 in.)

Figure 5.14 shows that relative curvature demand at pile 1 head and body is similar up to a displacement around 0.05 m (2 in.). Then, the curvature demand at pile 1 head is much higher than at pile 1 body. It is interesting to note that MOTEMS damage level 1 at the pile head, Level1H, occurs just after its curvature demand begins to separate of that at the pile body. MOTEMS damage level 2 at the pile head, Level2H, occurs at a displacement around 0.13 m (5 in.) and a curvature ratio of 7. On the other hand, only MOTEMS damage level 1 is present at the
pile body, at a displacement around 0.10 m (3.9 in.) and a curvature ratio of 1.9.

In Figure 5.14, the levels of damage were calculated according to MOTEMS strain limits. A better intuitive idea of the strain levels reached at the head and body of pile 1 can be obtained comparing concrete and steel stress-strain curves at these pile critical zones, see Figure 5.15.

Figure 5.15 a) shows the cover concrete stress-strain curves at the head and body of pile 1, and the core concrete at the head of pile 1. The cover concrete curves illustrate that the strains are much higher at the head than at the body of the pile. To avoid an excessive number of curves in the figure, and since even the cover concrete at the body of the pile has not reached relatively high strains, only the core concrete curve at the pile head has been also shown in this figure. It illustrates that the extreme core concrete fibers at the pile head have reached residual stresses and strains, in other words, its damage is relatively severe. On the other hand, Figure 5.15 b) shows the stress-strain curves for a dowel bar at the pile head and for a prestressed steel strand at the pile body. The dowel bar works clearly in the nonlinear range in contrast with the prestressed steel strand which works basically in the linear range. These results confirm that the structural
damage is relatively moderate and severe at the pile head, and relatively minor at the pile body.

Finally, the structural behavior at the head and body of pile 1 is again checked, this time using a curve displacement at the pile head versus steel strain, see Figure 5.16:

![Figure 5.16 - Displacements at the head of the pile 1 vs. steel strains at the extreme dowel bar (pile head) and prestressed steel strand (pile body)](image)

Figure 5.16 shows two curves, one for steel strains in the extreme dowel bar at the pile head and the other for steel strains in the extreme prestressed steel strand at the pile body. These strain curves have been selected since they control MOTEMS damage level 1 and 2. As can be observed in Figure 5.16, damage level 1 at the pile head occurs around a displacement equal to 0.06 m, damage level 1 at the pile body (strain increment less or equal than 0.005) occurs around a displacement equal to 0.010 m, and finally, damage level 2 at the pile head occurs at a displacement approximately equal to 0.13 m. Figure 5.16 shows that the dowel strains at the pile
head are much higher than those at the prestressed steel strand at the pile body. These results further confirm that the structural damage is much more severe at the pile head than at the pile body.

Finally, note that the cyclic loading history has at least two cycles at the same level of displacement. The stiffness loss, between successive cycles at the same maximum displacement, explains the jumps in strain at the same level of displacement.

A simplified model such as Goel’s (2010-a, 2010-b) shows also that the crucial zone is the pile-deck connection. However, the length of the in-ground “hinge” should be assumed equal to 4% (damage level 1) and 6% (damage level 2) of the pile equivalent length. In this way, the results obtained using Goel’s model and the analytical model developed in this chapter are identical. Additional experimental studies are needed to validate these results.

As a final check, displacement vs. moment curves at pile 1 head are presented in Figure 5.17 taking into account mean, minimum (0.3*standard values), and maximum (2.0*standard values) soil stiffness properties.
As shown in the figure, the behaviors of all three cases are quite similar, even though MOTEMS damage levels occurs first at the pile 1 with stiffer soil. The stiffer soil curve also shows a small but abrupt descend at a displacement around 0.08 m (3.1 in). This descend is related to spalling in the cover concrete. Further analysis will be necessary to ensure that, in this specific case, the relative weakness of the cover concrete does not affect the overall behavior of the pile structure.

In summary, the 2D model of the critical wharf supported on piles substructure has shown that the influence of the changing axial forces on the structural behavior of the pile-deck connection is relatively moderate. It has also shown that the type and depth of the soil layers
used in this model produced a relatively smooth flexural moment curve along the length of the pile. Finally, changes in the soil stiffness between recommended minimum and maximum values do not show relatively important changes in the structural behavior of the piles. Since these soil characteristics were intended to represent standard conditions at ports in the West Coast of the U.S., other soil geometries and characteristics could have potentially a different impact on the pile-deck connection, and should be further investigated.
CHAPTER 6: PARAMETRIC STUDY OF PILE-DECK CONNECTIONS IN WHARF STRUCTURES

The interactions between the structural parameters controlling the rotational capacity of pile-deck connections at a moderate/severe damage level, and therefore the lateral displacements of the wharf supported on piles, are studied in this chapter. Rotational ductility is taken as the parameter that best describes this capacity. Then, the influence of the amount of spiral reinforcement, concrete strength, spiral yield stress, concrete cover, initial prestress, longitudinal reinforcement and axial force on the rotational ductility is checked using the model developed and described in Chapters 3 and 5. Since the accuracy of the results depends on the critical limiting concrete and steel strains, they are defined carefully. Initially, a factorial analysis shows the importance of spiral reinforcement, concrete strength, longitudinal reinforcement and axial force on pile-deck connection structural behavior. Two-dimensional (2D) and three-dimensional (3D) graphs of these structural parameters help to explain their effects and interactions.

This chapter begins with a brief explanation of the geometry of the model, which is necessary since there are minor differences with the model employed in Chapter 5 to describe the crucial wharf substructure. Next, the characteristics of the horizontal cyclic loading are explained in detail. ATC-24 (1992) is the reference guide for that.

Then, the parameters affecting the rotational ductility at moderate/severe damage level are presented. A total of seven parameters are taken into account in this study as explained above. This selection is based on a review of the relevant literature and on the results obtained in the validation process for the pile-deck connection analytical model (Chapters 2 and 3, respectively). Each of the initial parameters is considered with minimum and maximum values that bracket limits of their practical range of use in actual wharf structures.
The factorial analysis has been undertaken using two analytical alternatives and its results are confirmed using a regression analysis and comparison with the parameters used in code equations. Next, additional intermediate values of the most critical parameters found in the initial factorial analysis are included since minimum and maximum values, such as those used for the parameters in the factorial analysis, may not give enough information about the structural behavior at intermediate points. These detailed analyses focus on the parameters that the factorial analysis found to be the most important, and on their relationship with the PCI spiral reinforcement equations.

The work described above forms the basis for potential revisions to PCI and other code equations, where the amount of spiral reinforcement can now be prescribed as a function of rotational ductility. However, its current limitations are highlighted in the following paragraphs.

First of all, the geometry of the pile section is limited to an octagonal shape with diameter equal to 0.63 m (24 in.). Even though a range of longitudinal reinforcement amounts has been considered, the distribution and number of the steel bars is always considered as uniform, circular, and with 8 bars. Furthermore, the cover concrete range of values is valid only for the octagonal section with diameter equal to 0.63 m.

In addition, the 2D wharf substructure used in this parametric study is intended to be an example of typical wharf structures built on the West Coast of the United States. However, additional models with different bay lengths, pile heights above the ground level, deck thickness, pile diameter, as well as number, thickness, and type of soils, among other parameters, are needed for a comprehensive analysis of the pile-deck behavior. Based on these statements, this dissertation can be thought of as an initial, valuable contribution in terms of design code procedures, with more work still to be done on that particular topic.
6.1 Structural geometry

The geometry and loading conditions of the wharf substructure presented in Chapter 5 are considered here as being crucial to estimating the seismic behavior of the piles in a wharf structure. However, the results in Chapter 5 also indicated that the stiffness due to a different number of piles at the two pile rows at the land side of the wharf in the 2D model is not a significant factor in the structural behavior at the pile-deck connection. Therefore, it was decided to only use one pile, with the same size as the regular piles, at each support axis. This decision changed the dimensions of the beams representing the deck, as well as the initial vertical loading. Between piles 1 and 2, the beam section is now 2x0.9 m (6.6x3 ft); between piles 2 and 3, it is 1.2x0.6 m (4x2 ft). The numeration of the piles begins with the landside pile, and the initial vertical loading will be explained in detail in Section 6.3. All other structural characteristics remain the same as in Chapter 5.

Pile 1 is considered more crucial than pile 2. Indeed, it is subjected to changes in axial load due to horizontal displacements (see Figure 6.1). On the other hand, it is worth noting that the beam shear forces are capped by the flexural capacities of the pile-deck connections.

![Figure 6.1 - Additional axial loads in piles due to shear in the deck](image)

From the above paragraph and Figure 6.1, clearly the frame model developed in Chapter 5 is
preferred to a model that includes only one pile, since a frame model allows one to investigate
the effect of varying axial loads on the structural behavior of the piles. The model does not
include masses and other than hysteretic damping. This is further explained in the following
section.

6.2 Loading history

The loading history employed in this chapter is based on ATC-24 (1992) recommendations. That guideline is only concerned with “slow cyclic” loading; in other words, inertial effects and damping other than hysteretic are not taken into account. However, results from these loadings can be considered conservative for performance assessment (Krawinkler 1996), since they typically show a small decrease in strength and increase in the rate of deterioration compared with dynamic loading. In addition, since the model checks the structural behavior at particular crucial points of the moment-rotation curve, additional loading cycles are needed at the range where these crucial points occur. This is possible with the cyclic loading option, and so it was therefore selected for the parametric study described in this chapter.

Table 6.1 shows the ATC-24 recommendations. The cycles are identified with the letter “n” and a subindex. The peak deformation at each cycle is always referenced to the yield displacement “dy”. The number of cycles per peak deformation can be found under the column entitled “Description”. More cycles per step could be applied if the rate of deterioration turns out to be relatively high. In the specific case of the model developed in this dissertation, additional cycles were needed to cover in detail the cover concrete spalling, since the reduction in moment capacity was relatively important due to the loss of the relatively thick concrete cover.
### Table 6.1 - ATC-24 general recommendations

<table>
<thead>
<tr>
<th>Cycle identification</th>
<th>Peak Deformation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n_0$</td>
<td>$&lt;dy$</td>
<td>number of cycles with a peak deformation less than $dy$, should be at least 6.</td>
</tr>
<tr>
<td>$n_1$</td>
<td>$dy$</td>
<td>number of cycles with peak deformation equal to $dy$, should be at least 3.</td>
</tr>
<tr>
<td>$n_2$</td>
<td>$dy+\Delta$</td>
<td>number of cycles with peak deformation $dy+\Delta$, should be at least 3.</td>
</tr>
<tr>
<td>$n_3$</td>
<td>$dy+2\Delta$</td>
<td>number of cycles with peak deformation $dy+2\Delta$, should be at least 3.</td>
</tr>
<tr>
<td>$n_4$</td>
<td>$dy+3\Delta$</td>
<td>number of cycles with peak deformation $dy+3\Delta$, should be at least 2.</td>
</tr>
<tr>
<td>$n_m$</td>
<td>$dy+(m-1)*\Delta$</td>
<td>number of cycles with peak deformation $dy+(m-1)*\Delta$, should be at least 2.</td>
</tr>
</tbody>
</table>

The key element in the ATC-24 recommendations is the yield displacement, since other displacements are presented as functions of this parameter. The displacement “$\Delta$” depends on the detail required to describe the damage development. Finally, the factor “$m$” should be big enough so that the structure/specimen will reach collapse.

Table 6.2 shows the cyclic load history employed in this dissertation. In Table 6.2, the values of the reference displacement and the displacement increment, which will be explained later, are shown at its head. The same cyclic loading is employed for all of the parametric study cases (otherwise the results could be difficult to compare), even though the yield displacement can change a bit in each case. Fortunately, initial results shown that this variation is small. Additional series are also considered to cover the yield displacement variation. The displacement increment for each analysis step was chosen initially as one millimeter (0.039 in), since the millimeter is the smallest length unit below a centimeter for which the results have practical meaning. Each group of cycles in Table 6.2 are called “series” and identified with a number. The number of cycles per series is shown in the 2nd column. The next column is for the
ratio of peak to “yield” deformation. The following column is for the peak lateral displacement at the pile-deck connection per the particular series. Finally, the last column is the number of steps needed to reach the peak displacement in increments of one millimeter, except for series 4, 5, and 6 where the increment is 0.1 millimeter. This is necessary to avoid computational instabilities at these series.

Table 6.2 - Cyclic load history employed in this dissertation

<table>
<thead>
<tr>
<th>Series</th>
<th># of cycles</th>
<th>D/ dy</th>
<th>Displacement [m (in.)]</th>
<th># of steps</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2</td>
<td>0.20</td>
<td>0.013 (0.5)</td>
<td>13</td>
</tr>
<tr>
<td>2</td>
<td>2</td>
<td>0.40</td>
<td>0.026 (1.0)</td>
<td>26</td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>0.60</td>
<td>0.039 (1.5)</td>
<td>39</td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>0.80</td>
<td>0.052 (2.0)</td>
<td>520</td>
</tr>
<tr>
<td>5</td>
<td>3</td>
<td>1</td>
<td>0.065 (2.6)</td>
<td>650</td>
</tr>
<tr>
<td>6</td>
<td>3</td>
<td>1.5</td>
<td>0.098 (3.9)</td>
<td>970</td>
</tr>
<tr>
<td>7</td>
<td>3</td>
<td>2</td>
<td>0.130 (5.1)</td>
<td>130</td>
</tr>
<tr>
<td>8</td>
<td>3</td>
<td>3</td>
<td>0.195 (7.7)</td>
<td>195</td>
</tr>
<tr>
<td>9</td>
<td>2</td>
<td>5</td>
<td>0.325 (12.8)</td>
<td>325</td>
</tr>
<tr>
<td>10</td>
<td>1</td>
<td>7</td>
<td>0.455 (17.9)</td>
<td>455</td>
</tr>
</tbody>
</table>

The load history employed in this chapter has more series than the minimum recommended by ATC-24 for two reasons. First, it covers any possibility that for particular cases the yield displacement could be less/greater than the expected reference displacement value.
Second, this approach documents in detail the deterioration of the pile capacity, for instance after cover spalling, as explained above. Note that the last series has only one cycle, since it is expected to be beyond pile failure. The resulting cyclic load history is finally shown in Figure 6.2.

![Figure 6.2 - Cyclic load history (0.1 m=3.94 in.)](image)

Figure 6.2 shows the cycle-point to displacement graph of the cyclic loading employed in this dissertation. The cycle-points are those when the lateral displacement at the pile-deck connection is highest, lowest or zero.

### 6.3 Parameter list

The input parameters for the factorial analysis can be seen in Table 6.3. In this table, the concrete strength and spiral yield stress are estimates of expected or likely strength values. This
is necessary since, in seismic design, damage is anticipated. Consequently, use of traditional lower-bound estimates for material strengths, in combination with strength reduction factors, could result in higher than needed ductility demands. Priestley et al. (1996) recommend the following design material strength:

Concrete: \( f'_{ce} = 1.3 f'_{c} \)  
Steel: \( f_{ye} = 1.1 f_{y} \)  

(6.1)  
(6.2)

The expected concrete strength acknowledges the influence of conservative batching practice (average 28-day compression strength is typically 20% above the specified value) and the increase in strength after 28 days before the structure is subjected to the full design loading (Priestley et al. 2007). The expected steel strength is relatively similar to the specified yield stress value, due to the fairly high level of control in steel fabrication.

The relative importance of unconfined concrete can be highlighted through the cover value. The initial prestress in the concrete section, \( f_{pc} \), can indirectly affect the pile-deck connection, and it is an important characteristic of the pile body section. Another key parameter is the ratio of longitudinal reinforcement to concrete area, \( A_{sl} / A_{g} \), since dowel reinforcing steel is resisting the tension forces resulting from bending. The spiral reinforcement ratio, \( \rho_{s} \), is known to substantially affect the confined concrete strength.

Finally, the effect of axial force is also considered in the parameter list. Pile axial force has two components: an initial vertical axial load produced by permanent and temporal gravity loads, and any differential axial force produced as a reaction in the frame substructure when subjected to lateral horizontal displacements. The parameter considered here is the ratio of initial vertical axial load to the product of expected concrete strength times area of the pile section, \( P_{vertical} / (A_{g} * f'_{ce}) \). Any additional axial load due to the lateral horizontal displacement of the wharf
depends in part on the flexural capacity of the pile-deck connection and therefore is not an independent parameter.

Table 6.3 - Parameters used in factorial analysis

<table>
<thead>
<tr>
<th>Input Parameter</th>
<th>Range of Values</th>
<th>Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. $f_{ce}$</td>
<td>41.4-82.8 [MPa] 6-12 [ksi]</td>
<td>Expected concrete strength</td>
</tr>
<tr>
<td>2. $f_{ye}$</td>
<td>413.8-551.7 [MPa] 60-80 [ksi]</td>
<td>Expected yield stress of spiral</td>
</tr>
<tr>
<td>3. Cover</td>
<td>64-89 [mm] 2.5-3.5 [in.]</td>
<td>Concrete cover</td>
</tr>
<tr>
<td>4. $f_{pc}$</td>
<td>4.8-9.7 [MPa] 0.7-1.4 [ksi]</td>
<td>Average prestress in the concrete section</td>
</tr>
<tr>
<td>5. $A_{s}/A_{g}$</td>
<td>0.013-0.025</td>
<td>Longitudinal reinforcement ratio</td>
</tr>
<tr>
<td>6. $\rho_s=4<em>A_{st}/(D_s</em>s)$</td>
<td>0.007-0.018</td>
<td>Spiral reinforcement ratio</td>
</tr>
<tr>
<td>7. $P_{vertical}/(A_{g}*f_{c})$</td>
<td>Varies (see explanation below)</td>
<td>Normalized axial force</td>
</tr>
</tbody>
</table>

Table 6.3 also shows the output variable, which is the rotational ductility at moderate/severe
level of damage. This is defined as the rotation normalized with respect to the yield rotation when the strains at the steel and concrete reach values associated with moderate/severe damage. The strain values used in this dissertation will be explained later. Rotational ductility is considered in the technical literature as a representative engineering parameter of the pile-deck structural behavior; for example, see Brackmann (2009).

The range of values for each input parameter, as presented in the third column of Table 6.3, will be explained in the following paragraphs.

6.3.1 Range of values

The use of factorial analysis requires that initially two values for each input parameter are studied. These values, and their respective sources of recommendation, are presented in Table 6.4.

The values for parameter 1 are relatively higher than those recommended by the PCI guideline, since they are expected concrete strength values. They are also based on the experimental research carried on at the University of Washington and at the University of California, San Diego.

For spiral yield strength values, the maximum is slightly higher than that recommended by PCI, since it is the expected yield strength value. On the other hand, the minimum is based on PCI’s mention that the spiral yield strength typically is greater than 414 [MPa] (60 [ksi]).

The minimum cover corresponds to the nominal value used regularly in prestressed concrete piles. The maximum value considers that the actual cover varies, with some values higher than the nominal cover since the geometry of the pile section is octagonal.
| PARAMETER 1: |  
| $f'_{ce} = $ | Minimum (PCI (2004) Section 20.3.1.6) and experimental research  
| 41.4 (6) [MPa (ksi)] |  
| 82.8 (12) [MPa (ksi)] | Maximum (PCI (2004) Section 20.3.1.6) and experimental research  
| PARAMETER 2: |  
| $f_{yte}= $ | (PCI (2004) Section 20.5.3.5.2)  
| 413.8 (60) [MPa (ksi)] |  
| 551.7 (80) [MPa (ksi)] | (PCI (2004) Section 20.5.3.5.2)  
| PARAMETER 3: |  
| cover= | Nominal cover (typical practice)  
| 2.5 (63.5) [in. (mm)] |  
| 3.5 (88.9) [in. (mm)] | Variation due to the pile octagonal geometry  
| PARAMETER 4: |  
| $f_{pc}= $ | Low prestress value (PCI (2004) Section 20.5.3.3)  
| 4.8 (0.7) [MPa (ksi)] |  
| 9.6 (1.4) [MPa (ksi)] | High prestress value (PCI (2004) Section 20.5.3.3)  
| PARAMETER 5: |  
| $A_{sl}/A_{g}= $ | Based on values observed in current piles (see explanation below)  
| 0.013 |  
| 0.025 | Corresponds to 5% reduction of the pile cross section due to the dowel duct holes.  
| PARAMETER 6: |  
| $\rho_{s}= $ | (PCI (2004) Section 20.5.4.2.5.2-3)  
| 0.007 |  
| 0.018 | Value observed in current piles (typical practice)  
| PARAMETER 7: |  
| $P_{vertical}/(A_{g}*f'_{c})= $ | varies | See explanation below  

The minimum and maximum values for prestress in the concrete are those recommended by the PCI guideline. The minimum value is related to the prevention of cracking due to handling and installation. The maximum value is used in case of pile lengths in excess of 15 m.
The ratio of longitudinal reinforcement to concrete area is based on an assessment of the amount of longitudinal reinforcement in “seismic” piles of wharves already existing in ports at the Pacific coast of the U.S. The minimum value is obtained assuming 8 dowel bars with a diameter smaller by one-eighth of an inch (3 mm) than the value observed in US ports. The maximum value corresponds to a 5% reduction of the pile cross section due to the dowel duct holes.

The spiral volumetric ratio has minimum and maximum values found, as for the longitudinal reinforcement ratio, in “seismic” piles of wharves already constructed in the U.S. The minimum value is related to handling and installation of the piles. The maximum value corresponds to that existing near the head of a pile, immediately after the special spiral reinforcement needed for the pile installation (driving). Figure 6.3 shows not only the typical different spiral reinforcement along the length of a pile, but also the reason why the special pile installation reinforcement at the head of the pile is not used in this study. During installation, many times the pile finds enough soil resistance before the head reaches the deck level. Therefore, this short remnant at the end is cut off and its special reinforcement is lost.
Finally, the minimum and maximum values for initial vertical axial load are explained as follows. First of all, the various typical tributary areas for each pile are shown in Figure 6.4.
Since the tributary areas for piles 1 and 2 are different, the corresponding vertical axial loads are different, too. The minimum and maximum vertical loads are calculated with the following equations:

\[ Axial\ minimum = (1 - k) \times D \]  (6.3)

\[ Axial\ maximum = (1 + k) \times (D + L) \]  (6.4)

where D and L correspond to permanent and temporal vertical (gravity) loads, respectively. The effect of vertical earthquake loading has also been included through the factor k, which is a percentage of the effective horizontal peak ground acceleration. Including vertical seismic effects is important for the output of this work, since it allows studying a broader load range that is not usually taken into account in practice or even in research.
Permanent vertical load includes the weight of the deck, plus equipment equal to 1.4 kN/m² (0.035 kip/ft²), and also one-third of the pile length between the deck soffit and a point into the ground equal to 5 times the pile diameter. Note also that the thickness of the deck changes between axis F – E and E – D, from 0.9 m (3 ft.) to 0.6 m (2 ft.). In Figure 6.4, the crane rail is visible on axis F, so pile 1 has to include the temporal weight transmitted by one of the crane legs equal to 730 kN/m (50 kip/ft.) (MOTEMS 2007). On the other hand, pile 2 only has to take into account distributed temporal loading equal to 48 kN/m² (1 kip/ft²) (MOTEMS 2007). The factor k is calculated as 80% (Collier and Elnashai 2001) of the effective peak ground acceleration experienced by the wharf. For ports on the West Coast of the U.S., the maximum expected horizontal acceleration is 0.6g (Petersen et al. 2011); therefore, k is equal to 0.48. Since it is unlikely that a significant earthquake would occur at the same time as the worst temporal vertical loading conditions explained above, an additional global reduction factor equal to 0.67 is included in the final calculations. The temporal live load has been explained in Section 5.3.

For piles 1 and 2, the calculated minimum/maximum axial loads, as relative values to the product between expected concrete strength and area of the pile section, are then those shown in Table 6.5.

**Table 6.5 - Vertical Loads**

<table>
<thead>
<tr>
<th></th>
<th>P/(f′ce*A_g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Minimum Axial Load</td>
</tr>
<tr>
<td>Pile 1</td>
<td>0.015</td>
</tr>
<tr>
<td>Pile 2</td>
<td>0.030</td>
</tr>
</tbody>
</table>

Table 6.5 shows that the minimum axial load is higher at pile 2 than at pile 1, since it depends on the tributary area. However, the maximum axial load is higher at pile 1 than at pile 2, since the
weight of the crane is included at pile 1.

6.4 Parametric study methodology

As explained in Section 6.3, seven factors or parameters have been selected to check their effects on the rotational ductility of typical structural concrete pile-deck connections. Since high- and low-level values exist for each parameter, there are a total of $2^7 = 128$ cases, with each case corresponding to one computational run. The main “effect” of a factor is defined to be the difference between the mean response (mean rotational ductility) when the factor is at its high level and the mean response when the factor is at its low level. However, there are other “effects” due to the interactions between the factors. For instance, there are 14 possible two-way (two parameters at a time) “effects”. The overall “effect” estimates can be obtained from a “sign table”. In this table, the main “effects” are given the sign of + if the factor is at its high level, and the sign of − otherwise. For the interactions, the signs are computed by taking the product of the signs of the corresponding main “effects”. The importance of the main and interaction effects are then tested using an analysis of variance (ANOVA) (Johnson 2005).

The factorial analysis described above can highlight the most important parameters that affect the rotational ductility at a pile-deck connection, as described in more detail below.

6.4.1 Available statistical tools for parametric study

In a standard parametric study, it is usual to consider the effect of each parameter changing one at a time. In this way, the researcher is sure to know which factor has affected the output and which has not. However, there are available statistical tools which allow learning the parameter effects even while changing more than one parameter at a time. This is more efficient and has the additional benefit of acquiring information about the parameter interactions. This technique is known as factorial analysis (Johnson 2005). Although there are many specific
implementations of this technique, the main characteristic common to all of them is the simultaneous variation of the input parameters. Note also that factorial analysis is not a new approach. The statistician R.A. Fisher developed many of the fundamental ideas in the 1920s for applications in agriculture. They were introduced more generally in scientific experimentation by Fisher (1935) and Yates (1937). Factorial analysis has been widely used in process control and related industries since at least the 1950s.

To be acquainted with the terminology, Hinkelmann and Kempthorne (2008) will be followed. Assume \( n \) factors \( A_1, A_2, \ldots, A_n \), where factor \( A_i \) has \( m_i \) levels \( a_{i1}, a_{i2}, \ldots, a_{im_i} \) \((i=1,2,\ldots,n)\). A treatment combination is denoted by \((a_{i1}a_{2j}a_{3k} \ldots a_{nt})\) and there are \( \prod_{i=1}^{n} m_i \) such treatment combinations. The effect of a treatment combination is written as \( \tau_{ijk \ldots t} \) and defines the various main effects and interactions through an expansion of the type as illustrated for \( n=3 \):

\[
\tau_{ijk} = \mu + A_{1i} + A_{2j} + (A_1A_2)_{ij} + A_{3k} + (A_1A_3)_{ik} + (A_2A_3)_{jk} + (A_1A_2A_3)_{ijk} \quad (6.5)
\]

Where \( \mu \) represents the overall mean; \( A_{1i}, A_{2j}, A_{3k} \) represent the main effects associated with the factors \( A_1, A_2, A_3 \); \( (A_1A_2)_{ij}, (A_1A_3)_{ik}, (A_2A_3)_{jk} \) represent the two-factor interactions with those factors; and \( (A_1A_2A_3)_{ijk} \) represents the three-factor interaction.

One of the disadvantages, from a practical point of view, of factorial analysis is the fact that the number of treatment combinations increases rapidly as the number of factors and/or levels increases. One way out of this dilemma is to consider only a subset of all possible treatment combinations, a so-called fractional factorial. Another possibility is to restrict the number of levels of each factor to two. Those two levels may be chosen so that they cover, in some sense, the practical range of levels. Suppose there are \( n \) such factors. Then it is referred to as a \( 2^n \) factorial.
A $2^n$ factorial is most useful as an exploratory experiment, since it cannot examine trends other than linear ones between the two levels of study. However, once the most important factors/parameters have been disclosed, attention can be focused on them, studying them at intermediate levels but maintaining the other not important parameters constant.

One additional issue is to determine whether the various observed non-zero main and interaction effects are statistically significant. This is often addressed by using standard methods from analysis of variance (ANOVA) and linear regression (Walpole et al. 2007, Navidi 2008) to carry out the multiple comparisons in a meaningful way when it can be assumed that the noise terms are independent and identically distributed (i.i.d.) and normally distributed. Tests on main and interaction effects can be performed from a single replicate of the factorial analysis, or, even better, from multiple randomized-input replicates. Standard statistical F-tests or t-tests form the basis for hypothesis tests and calculation of probability values, commonly called P-values. Some of the tests available for checking assumptions are further summarized in Barker (1994).

### 6.4.2 Application of factorial analysis to find the most important parameters in an equation

A small, but insightful, example can show the principal ideas behind the factorial analysis. Assume that a certain phenomenon depends on three parameters ($x_1$, $x_2$, and $x_3$) according to the following equation:

$$ y = 2 + 3x_1 + 4x_2 + 0.05x_3 + 5x_1x_2 $$

(6.6)

Nonlinearity is introduced in this equation thanks to the combined effect between the parameters $x_1$ and $x_2$; otherwise the equation’s terms are linear. Note that $x_1$ and $x_2$ are more important than $x_3$, and that the independent (constant) term of 2 can be regarded as the “mean” value when the parameters vary between a minimum of -1 and a maximum of +1.

Now assume that equation (6.6) is not known, but that we use the following equation as
an initial estimate:

\[ y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \beta_3 x_3 + \beta_4 x_1 x_2 + \beta_5 x_1 x_3 + \beta_6 x_2 x_3 + \beta_7 x_1 x_2 x_3 \]  

(6.7)

It should be remarked that the coefficients \( \beta_i \) are not known in equation (6.7). Assume also that the values of “y” when each parameter has minimum (-1) and maximum (+1) values are known; that means \( 2^3 = 8 \) cases. Each case can be ordered to produce a sign table, which is the matrix in the middle of Figure 6.5. Note also that, in this example, the “y” values are calculated using Eq. (6.6).

Figure 6.5 shows not only the resultant sign table, but also the output, the mean value of the output, and the effect of each parameter on the output:

<table>
<thead>
<tr>
<th>x1</th>
<th>x2</th>
<th>x3</th>
<th>x1x2</th>
<th>x1x3</th>
<th>x2x3</th>
<th>x1x2x3</th>
<th>y</th>
</tr>
</thead>
<tbody>
<tr>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>-0.05</td>
</tr>
<tr>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>-4.05</td>
</tr>
<tr>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-2.05</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>13.95</td>
</tr>
<tr>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>0.05</td>
</tr>
<tr>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-3.95</td>
</tr>
<tr>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1.95</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>14.05</td>
</tr>
</tbody>
</table>

\[ \text{effect} = \frac{\text{contrast}}{2^{p-1}} \]  

(6.8)

Where \( p \) is the number of parameters and \( \text{contrast} \) is the result of adding the eight known “y” values, each of them multiplied by the appropriate cell of the sign table (same row than the “y” value and a different column for a different effect). In this way, seven effects can be calculated using Eq. 6.8, each of them corresponding to one column of the sign table. To determine the
importance of each effect, an analysis of variance is performed. But first, we need the sum of the variances for each case (corresponding to each row of the sign table) – see Table 6.6.

Table 6.6 - The “y” Values and Variances for Each Row of the Sign Table in Figure 6.5

<table>
<thead>
<tr>
<th>“y”</th>
<th>Variance</th>
</tr>
</thead>
<tbody>
<tr>
<td>-0.05</td>
<td>4.2025</td>
</tr>
<tr>
<td>-4.05</td>
<td>36.6025</td>
</tr>
<tr>
<td>-2.05</td>
<td>16.4025</td>
</tr>
<tr>
<td>13.95</td>
<td>142.8025</td>
</tr>
<tr>
<td>0.05</td>
<td>3.8025</td>
</tr>
<tr>
<td>-3.95</td>
<td>35.4025</td>
</tr>
<tr>
<td>-1.95</td>
<td>15.6025</td>
</tr>
<tr>
<td>14.05</td>
<td>145.2025</td>
</tr>
</tbody>
</table>

$\Sigma (y)/8 = 2.00 \quad \Sigma(\text{variances}) = 400.02$

An analysis of the variance about the mean value of 2 is then performed in Table 6.7.

Table 6.7 - Analysis of variance

<table>
<thead>
<tr>
<th>Source</th>
<th>Effect</th>
<th>Sum of Squares =S</th>
<th>df</th>
<th>Mean Square =S/df=MS</th>
<th>F(1,8) =MS/MSE</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td>X1</td>
<td>6</td>
<td>216</td>
<td>1</td>
<td>216</td>
<td>4.319784</td>
<td>0.0713</td>
</tr>
<tr>
<td>X2</td>
<td>8</td>
<td>384</td>
<td>1</td>
<td>384</td>
<td>7.679616</td>
<td>0.0242</td>
</tr>
<tr>
<td>X3</td>
<td>0.1</td>
<td>0.06</td>
<td>1</td>
<td>0.06</td>
<td>0.0012</td>
<td>0.9755</td>
</tr>
<tr>
<td>X1X2</td>
<td>10</td>
<td>600</td>
<td>1</td>
<td>600</td>
<td>11.9994</td>
<td>0.0085</td>
</tr>
<tr>
<td>X1X3</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>X2X3</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>X1X2X3</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>Error</td>
<td>400.02</td>
<td>8</td>
<td>50.0025</td>
<td></td>
<td>1</td>
<td></td>
</tr>
</tbody>
</table>

In Table 6.7, the column “Source” indicates the parameter or combination of parameters being
analyzed; “Effect” contains the effect of each parameter or combination of parameters; “Sum of Squares” (or “S”) is equal to the square of the contrast divided by 2 to a power equal to the number of parameters minus 1; “df” is the degrees of freedom, or number of unknowns, in the statistical process; and “Mean Square” is equal to the ratio between “S” and “df”. Note that the mean square of error then has a particular name, MSE. Finally, the statistic F is the ratio MS/MSE, with 1 “df” for MS and 8 “df” for MSE. The probability of the parameter or combination of parameters not being significant to change the mean value of 2 is finally given in the last column headed as “P”. The results of this last column show small values in the cells corresponding to X1, X2 and X1X2. This is interpreted as showing that these factors are the most important toward calculating “y” in Equation 6.7. Note that in this example, the usual limit for “P” (<0.05) has been relaxed to help clarify the concepts behind factorial analysis. However, the values of the β coefficients are still not known. Observe that the sign matrix A (including an additional column with ones, representing the independent term), the β coefficients, and the output form a solvable system of equations, as follows:

\[ [A][\beta] = [y] \]  \hspace{1cm} (6.9)

\[ \{\beta\} = [A]^{-1}[y] \]  \hspace{1cm} (6.10)

Figure 6.6 shows the β values, obtained solving Equation (6.10).

\[ [A]^{-1}[y] = \{\beta\} \]

<p>| | | | | | | | | |</p>
<table>
<thead>
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<td>0.125</td>
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<td>0.125</td>
<td>0.125</td>
<td></td>
</tr>
</tbody>
</table>

Figure 6.6 - Calculation of β Values
Note that the same $\beta$ values could be obtained from the “effects”, by simply dividing them by 2.

The final equation is then:

$$y = 2 + 3x_1 + 4x_2 + 0.05x_3 + 5x_1x_2$$  \hspace{1cm} (6.11)

The same initial equation, as stated in Eq. (6.6), has been again obtained following this factorial analysis. However, observe that the not-significant terms are now also known ($X_1$, $X_2$, and $X_1X_2$). The following equation, though simpler than the initial equation, is still accurate:

$$y = 2 + 3x_1 + 4x_2 + 5x_1x_2$$  \hspace{1cm} (6.12)

In short, once the “effect” of each parameter or combination of parameters has been determined, its importance can be assessed with an analysis of variance. Furthermore, a simplified equation can also be determined relating the most important “effects” with the output.

**6.5 Strains at moderate/severe damage level**

In the factorial analysis, the relative importance of each parameter to describe pile-deck connection structural behavior is evaluated using the rotational ductility at a moderate/severe level of damage. Initially, this damage level was defined using MOTEMS (2007) critical strains, see Chapter 4, Section 4.1, Table 4.3 (MOTEMS values for steel and concrete strains at moderate/severe damage level are 0.05 and 0.025, respectively). However, it was then realized that in cases with minimum spiral reinforcement and maximum axial load (which were controlled by the concrete strain limit), the MOTEMS limit was not appropriate. And furthermore, the definition of yielding also needed some further refinement. Therefore, the modifications presented in Table 6.8 are considered necessary. These modifications are based on an elastic-perfectly plastic analytical model and on Mander’s concrete model for the concrete strain values at the moderate/severe damage level.
Table 6.8 – Definition of strains at yielding and moderate/severe level of damage

<table>
<thead>
<tr>
<th></th>
<th>Yielding</th>
<th>Moderate/severe damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel</td>
<td>( \varepsilon_s = \varepsilon_y \frac{M_{\text{max}}}{M_{ys}} )</td>
<td>( \leq 0.05 )</td>
</tr>
<tr>
<td>Concrete</td>
<td>( \varepsilon_c \leq \varepsilon_c (f_c = f_c') \frac{M_{\text{max}}}{M_{yc}} )</td>
<td>Mander’s model ( \leq 0.025 )</td>
</tr>
</tbody>
</table>

In Table 6.8, \( \varepsilon_y \) is the steel yield stress, \( M_{\text{max}} \) is the maximum moment capacity, \( M_{ys} \) is the moment calculated when the steel closest to the tension face of the section is yielding, \( \varepsilon_c (f_c = f_c') \) is the concrete strain at the peak unconfined concrete strength and \( M_{yc} \) is its corresponding moment. The ratio \( \frac{M_{\text{max}}}{M_{ys,c}} \) assumes an elastic-perfectly plastic analytical model. Note that the term \( M_{ys,c} \) refers to either the steel or the concrete moments related to yielding. The steel limit at the moderate/severe damage level is a conservative one (Priestley et al. 2007), but it is maintained since calculations using damage indexes produced similar rotation values to those using this steel conservative limit. The confined concrete strain limit at the moderate/severe damage level is taken to occur when fracture of the transverse reinforcement confining the core occurs. This can be estimated by equating the increase in strain-energy absorbed by the concrete, above the value suitable for unconfined concrete to the strain-energy capacity of the confining steel (Mander et al. 1988). Yielding and moderate/severe damage levels are governed by whichever concrete or steel limiting condition occurs first. The values and equations suggested in Table 6.8 are consistent with those in seminal papers such as by Watson et al. (1994) and Paultre and Legeron (2008), which used them to develop spiral reinforcement equations. Note that several of the spiral reinforcement equations in both PCI and the Canadian code are based on these papers. Note also that this definition of yielding is slightly different than that presented in Chapter 4.
Figure 6.7 shows the differences between MOTEMS and the proposed modifications to crucial strains at the moderate/severe damage level. The data has been grouped in four groups depending on the amount of spiral reinforcement (S) and axial load (A).

Figure 6.7 – Concrete or steel controls the moderate/severe damage level

Figure 6.7 shows that group 1 (minimum spiral reinforcement and minimum axial load) and group 4 (maximum spiral reinforcement and maximum axial load) have mixed results, meaning that either steel or concrete can reach their crucial strains and therefore control the moderate/severe damage level. On the other hand, group 2 (maximum spiral reinforcement and minimum axial load) is almost always controlled by steel reaching its crucial strain, and group 3 (minimum spiral reinforcement and maximum axial load) is almost always controlled by concrete reaching its crucial strain. MOTEMS resulting critical rotations and/or displacements compared to those using the proposed modifications are the same if steel controls the
moderate/severe damage level, but different if concrete controls the damage.

Table 6.9 – Rotation using DPA equal to 0.4 to rotation calculated using proposed crucial strains

<table>
<thead>
<tr>
<th>Case</th>
<th>Ratio</th>
<th>Case</th>
<th>Ratio</th>
<th>Case</th>
<th>Ratio</th>
<th>Case</th>
<th>Ratio</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>0.279</td>
<td>33</td>
<td>0.834</td>
<td>65</td>
<td>0.373</td>
<td>97</td>
<td>0.914</td>
</tr>
<tr>
<td>2</td>
<td>0.447</td>
<td>34</td>
<td>0.469</td>
<td>66</td>
<td>0.314</td>
<td>98</td>
<td>0.317</td>
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<tr>
<td>3</td>
<td>0.938</td>
<td>35</td>
<td>0.849</td>
<td>67</td>
<td>0.506</td>
<td>99</td>
<td>0.897</td>
</tr>
<tr>
<td>4</td>
<td>0.446</td>
<td>36</td>
<td>0.834</td>
<td>68</td>
<td>0.358</td>
<td>100</td>
<td>0.326</td>
</tr>
<tr>
<td>5</td>
<td>0.897</td>
<td>37</td>
<td>0.863</td>
<td>69</td>
<td>0.576</td>
<td>101</td>
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<tr>
<td>6</td>
<td>0.472</td>
<td>38</td>
<td>0.797</td>
<td>70</td>
<td>0.372</td>
<td>102</td>
<td>0.472</td>
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<tr>
<td>7</td>
<td>0.868</td>
<td>39</td>
<td>0.937</td>
<td>71</td>
<td>0.594</td>
<td>103</td>
<td>0.775</td>
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<tr>
<td>8</td>
<td>0.477</td>
<td>40</td>
<td>0.796</td>
<td>72</td>
<td>0.398</td>
<td>104</td>
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<tr>
<td>9</td>
<td>0.750</td>
<td>41</td>
<td>0.805</td>
<td>73</td>
<td>0.391</td>
<td>105</td>
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<tr>
<td>10</td>
<td>0.681</td>
<td>42</td>
<td>0.791</td>
<td>74</td>
<td>0.376</td>
<td>106</td>
<td>0.332</td>
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<tr>
<td>11</td>
<td>0.871</td>
<td>43</td>
<td>0.744</td>
<td>75</td>
<td>0.435</td>
<td>107</td>
<td>0.849</td>
</tr>
<tr>
<td>12</td>
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<td>44</td>
<td>0.785</td>
<td>76</td>
<td>0.521</td>
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<td>0.339</td>
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<td>0.795</td>
<td>77</td>
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<td>109</td>
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<tr>
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<td>47</td>
<td>0.963</td>
<td>79</td>
<td>0.558</td>
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<td>0.589</td>
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<td>0.303</td>
<td>114</td>
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<tr>
<td>19</td>
<td>1.235</td>
<td>51</td>
<td>0.965</td>
<td>83</td>
<td>0.751</td>
<td>115</td>
<td>1.103</td>
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<tr>
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<td>0.429</td>
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<td>84</td>
<td>0.364</td>
<td>116</td>
<td>0.230</td>
</tr>
<tr>
<td>21</td>
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<td>1.084</td>
</tr>
<tr>
<td>22</td>
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<td>54</td>
<td>0.435</td>
<td>86</td>
<td>0.388</td>
<td>118</td>
<td>0.411</td>
</tr>
<tr>
<td>23</td>
<td>1.180</td>
<td>55</td>
<td>1.029</td>
<td>87</td>
<td>0.848</td>
<td>119</td>
<td>1.074</td>
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<tr>
<td>24</td>
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<td>56</td>
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<td>0.328</td>
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<td>123</td>
<td>1.094</td>
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<td>124</td>
<td>0.379</td>
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<td>1.155</td>
<td>61</td>
<td>0.923</td>
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<td>0.661</td>
<td>125</td>
<td>1.064</td>
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<td>0.494</td>
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<td>1.039</td>
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<td>0.443</td>
<td>96</td>
<td>0.401</td>
<td>128</td>
<td>0.451</td>
</tr>
</tbody>
</table>

In Chapter 4, use of the Park and Ang Damage Index (DPA) was proposed as an alternative to
setting critical strain limits for defining the moderate/severe damage level. Table 6.9 shows the ratio between the rotations at moderate/severe damage using DPA equal to 0.4 and those obtained using the proposed critical strain limits. Table 6.9 shows all 128 cases used in the parametric study, ordered into four groups depending on the amount of spiral reinforcement and axial force. In addition, odd and even cases correspond to the minimum and maximum concrete strength, respectively. Finally, minimum longitudinal reinforcement corresponds to the first 16 cases of each group shown in Table 6.9. Correspondingly, the last 16 cases of each group in Table 6.9 correspond to maximum longitudinal reinforcement. DPA rotation values are reasonably near the rotation values related to the proposed crucial strains in groups 1, 2 and 4, if concrete strength is a minimum. In group 3, “minimum spiral reinforcement and maximum axial force”, DPA almost always produces rotations that compared to the rotations obtained using the proposed crucial strains give ratios less than 0.7.

Figures 6.8 through 6.11 show typical moment-rotation curves at a pile-deck connection, along with the position of the yield and moderate/severe damage level points. Figure 6.8 shows the typical moment-rotation curve for the case of “minimum spiral reinforcement and minimum initial vertical load”.

189
Figure 6.8 - Typical moment-rotation curve for the case “minimum spiral reinforcement and minimum initial vertical load”

Figure 6.8 indicates that the moment at the yielding point, represented with a star, occurs after some initial nonlinear behavior, due to the shape of the cover and core concrete stress-strain curves, and at relatively near the maximum moment capacity of the pile section at the pile-deck connection. The moderate/severe damage level point, represented with a circle at approximately 0.032 rad, is shown to occur after spalling of the concrete cover (signaled by an abrupt moment capacity descent after the peak moment), and just before an additional important moment capacity descent. The curve does not go beyond a rotation approximately equal to 0.05 rad, due to convergence problems.

Figure 6.9 shows the typical moment-rotation curve for the case of “maximum spiral reinforcement and minimum initial vertical load”.
Figure 6.9 - Typical moment-rotation curve for the case of “maximum spiral reinforcement and minimum initial vertical load”

Figure 6.9 exhibits a smaller range of rotation values than Figure 6.8, and yielding and moderate/severe levels of damage occur at slightly smaller rotations than in the previous case. For instance, the rotation at the moderate/severe damage level is approximately 0.03 rad. The curve’s descent after peak moment is relatively minor, with the moment capacity increasing again at the moderate/severe damage level, but after a relatively small rotation increment, the curve is interrupted by convergence problems.

Figure 6.10 shows the typical moment-rotation curve for the case of “minimum spiral reinforcement and maximum initial vertical load”.

191
Figure 6.10 - Typical moment-rotation curve for the case of “minimum spiral reinforcement and maximum initial vertical load”

Figure 6.10 indicates a yielding point relatively near the peak moment capacity, higher than in previous cases. On the other hand, the moderate/severe damage level occurs after an abrupt moment descent following the peak moment, at a rotation approximately equal to 0.02 rad (and before the curve is interrupted by convergence problems). Compared with the curves in previous figures, this curve shows fewer cycles to reach the critical damage level.

Figure 6.11 shows the typical moment-rotation curve for the case of “maximum spiral reinforcement and maximum initial vertical load”.

192
Figure 6.11 - Typical moment-rotation curve for the case of “maximum spiral reinforcement and maximum initial vertical load”

Figure 6.11 shows higher yield and moderate/severe damage level rotations compared with the previous case, due to the increased spiral reinforcement. Yielding occurs relatively near the peak moment capacity. The moderate/severe damage level occurs after spalling of the cover concrete, and before additional cycles that exhibit moderate and then even severe reductions in moment capacity.

6.6 Factorial analysis and results

The fundamental purpose of the factorial analysis employed in this dissertation is to determine which factors or parameters have an important effect on the rotational ductility of
typical structural concrete pile-deck connections at a moderate/severe damage level. Once this has been determined, more analysis cases can be run, in which the factors previously found to be important can be further varied.

First, a checking of the basic assumption upon which factorial analysis is built, namely the normality of the residuals, is performed. Figure 6.12 shows a normal probability plot of the residuals, where a residual is the difference between the rotational ductility at each case and its mean value, calculated using all 128 cases. In this plot, the expected values of the residuals assuming a normal distribution are plotted against the observed residuals. Note that the residuals are ordered taking into account their size (Navidi 2008, Walpole et al. 2007).

![Normal probability plot for the residuals](image)

**Figure 6.12 – Normal probability plot of the residuals**
In Figure 6.12, there is no evidence of a strong departure from normality, since the points approximately follow a straight line.

Then, the factorial analysis is carried out. The first alternative (“all interactions”) follows the standard procedure for this type of analysis (see Navidi (2008), and the explanation in Section 6.4.1). The second alternative (“mean + lower interaction effects”) assumes that the six- and seven-order interactions (groups of six and seven parameters) are negligible. The sum of squares for those interactions can be added and treated like an error sum of squares (Navidi 2008), and then used to create the F statistic. The number of degrees of freedom of this residual error is equal to the sum of the degrees of freedom for the six- and seven-order interactions. The main effects and lower order interactions can then be tested. Table 6.10 shows the most important effects (p < 0.05) obtained using these two alternatives.

**Table 6.10 – The most important effects using factorial analysis**

<table>
<thead>
<tr>
<th>All interactions (effect and p-value)</th>
<th>Mean + lower interaction effects (effect and p-value)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Axial</strong></td>
<td><strong>Axial</strong></td>
</tr>
<tr>
<td>1.0E-7</td>
<td>0.00040</td>
</tr>
<tr>
<td><strong>ρ_s</strong></td>
<td><strong>ρ_s</strong></td>
</tr>
<tr>
<td>1.0E-6</td>
<td>0.01670</td>
</tr>
<tr>
<td><strong>f'_ce Axial</strong></td>
<td><strong>f'_ce Axial</strong></td>
</tr>
<tr>
<td>0.00005</td>
<td>0.04750</td>
</tr>
<tr>
<td><strong>ρ_s Axial</strong></td>
<td><strong>ρ_s Axial</strong></td>
</tr>
<tr>
<td>0.00010</td>
<td>0.04850</td>
</tr>
<tr>
<td><strong>A_{sl}</strong></td>
<td></td>
</tr>
<tr>
<td>0.00011</td>
<td></td>
</tr>
<tr>
<td><strong>f'_ce</strong></td>
<td></td>
</tr>
<tr>
<td>0.00338</td>
<td></td>
</tr>
<tr>
<td><strong>A_{sl}ρ_s</strong></td>
<td></td>
</tr>
<tr>
<td>0.00619</td>
<td></td>
</tr>
<tr>
<td><strong>A_{sl}Axial</strong></td>
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</tr>
<tr>
<td>0.01200</td>
<td></td>
</tr>
<tr>
<td><strong>f'<em>ce A</em>{sl}ρ_s</strong></td>
<td></td>
</tr>
<tr>
<td>0.01273</td>
<td></td>
</tr>
</tbody>
</table>
In this table, *Axial* is the initial vertical load, $\rho_s$ is the volumetric spiral reinforcement ratio, $f'_{ce}$ is the expected concrete strength, and $A_{sl}$ is the longitudinal reinforcement (as the ratio longitudinal steel to concrete area). Table 6.10 shows the effects in order of relative importance, with the initial vertical load being the most important. Both alternatives show that initial vertical load, spiral reinforcement, and the interactions between initial vertical load and expected concrete strength, as well as between initial vertical load and spiral reinforcement, are important effects. Note that longitudinal reinforcement and concrete strength are also important in the first alternative. On the other hand, it has been found that expected yield stress of the spiral, cover, and prestress in the concrete section are relatively unimportant effects.

To confirm the validity of these results, a regression analysis is performed. The rotational ductility is calculated again, but only using the terms that the factorial analysis has found to be important. The resulting equation is the following:

$$
\mu_\theta = 2.719 - 0.367Axial + 0.233\rho_s + 0.158A_{sl} - 0.190f'_{ce}Axial + 0.156\rho_sAxial
-0.116 * f'_{ce} + 0.108A_{sl}\rho_s - 0.099A_{sl}Axial - 0.088f'_{ce}\rho_s - 0.098f'_{ce}A_{sl}\rho_s
$$

(6.12)

This equation has the $R^2$ statistic equal to 0.847, showing a good capacity to estimate the rotational ductility $\mu_\theta$. If a separate equation developed with only the four terms in the second factorial analysis alternative is used, the $R^2$ statistic is equal to 0.632. If the longitudinal reinforcement term is included, then the $R^2$ statistic is equal to 0.696. Finally, if the concrete strength term is also included, the $R^2$ statistic becomes equal to 0.730.

An analysis of the association/correlation between the independent and dependent variables is summarized in Table 6.11. The first column indicates the independent variables, while the second and third columns show the zero-order and partial correlations, respectively.
Table 6.11 – Correlation between structural parameters at the pile-deck connection and the rotational ductility

<table>
<thead>
<tr>
<th>Factors</th>
<th>Correlations</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Zero-order</td>
<td>Partial</td>
<td></td>
</tr>
<tr>
<td>$f'_{ce}$</td>
<td>-0.187</td>
<td>-0.427</td>
<td></td>
</tr>
<tr>
<td>$A_{st}$</td>
<td>0.253</td>
<td>0.539</td>
<td></td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>0.375</td>
<td>0.687</td>
<td></td>
</tr>
<tr>
<td>Axial</td>
<td>-0.590</td>
<td>-0.830</td>
<td></td>
</tr>
<tr>
<td>$f'_{ce}\rho_s$</td>
<td>-0.141</td>
<td>-0.337</td>
<td></td>
</tr>
<tr>
<td>$f'_{ce}Axial$</td>
<td>-0.305</td>
<td>-0.610</td>
<td></td>
</tr>
<tr>
<td>$A_{st}\rho_s$</td>
<td>0.174</td>
<td>0.402</td>
<td></td>
</tr>
<tr>
<td>$\rho_sAxial$</td>
<td>0.251</td>
<td>0.535</td>
<td></td>
</tr>
<tr>
<td>$A_{st}Axial$</td>
<td>-0.159</td>
<td>-0.373</td>
<td></td>
</tr>
<tr>
<td>$f'<em>{ce}A</em>{st}\rho_s$</td>
<td>-0.158</td>
<td>-0.371</td>
<td></td>
</tr>
</tbody>
</table>

The zero-order correlation shows an association between pairs of independent and dependent variables that does not include a control (extra) variable. The partial correlation is the correlation between the residuals resulting from the linear regression of each independent variable with all other variables, and the dependent variable with all these other variables. The correlation values vary between -1 (perfect negative correlation) and +1 (perfect positive correlation). A correlation of zero means that there is no relationship between the two variables. Table 6.11 confirms the importance of the association between Axial, $\rho_s$, $f'_{ce}Axial$, and $\rho_sAxial$ with the rotational ductility.
Figure 6.13 shows a plot of rotational ductility obtained using Equation (6.12) versus the rotational ductility as obtained from the OpenSees model:

![Graph showing comparison of rotational ductility using regression equation vs. OpenSees model](image)

**Figure 6.13 – Comparison of rotational ductility using the regression equation vs. the OpenSees model**

Figure 6.13 shows that the dispersion of the results for rotational ductility, comparing OpenSees and the regression equation, is relatively moderate.

An additional confirmation of these results is based on the fact that the important parameters according to the factorial analysis are also important in typical equations used by codes or guidelines (i.e., PCI, IBC, and PIANC) to calculate the minimum required spiral reinforcement – namely, concrete strength, axial load, longitudinal reinforcement, and rotational ductility.
The factorial analysis also produces a rotational ductility linear equation using 128 terms, whose coefficients are calculated using a system of 128 equations, one for each case of the factorial analysis. However, this equation does have its limitations. Even though, at the maximum/minimum levels of each parameter value, the match between this equation and the OpenSees rotational ductilities is perfect, this is not necessarily the case for intermediate levels. Figure 6.14 shows the perfect match between OpenSees and the factorial analysis rotational ductility at the maximum/minimum levels.

![Graph: OpenSees vs. Factorial Analysis](image)

**Figure 6.14 – Factorial analysis vs. OpenSees rotational ductility at min./max. levels**

As an example, Figure 6.15 shows factorial analysis vs. OpenSees rotational ductility, but this time for intermediate levels of axial load, and spiral reinforcement (see Table 6.11).
Figure 6.15 – Factorial analysis vs. OpenSees rotational ductility at intermediate levels of axial load and spiral reinforcement

Figure 6.15 shows no clear relationship between OpenSees rotational ductilities and those calculated using the linear equation with 128 terms, obtained in the factorial analysis, this time for intermediate levels of the parameters axial load and spiral reinforcement.

However, factorial analysis has highlighted the relationships between rotational ductility, spiral reinforcement, axial force, longitudinal reinforcement, and concrete strength. Specifically, it is found that the most important interactions are between axial load and spiral reinforcement, as well as between axial load and concrete strength. Additional data at intermediate points is needed, though, to confirm these interactions.

Table 6.12 therefore shows additional rotational ductilities, calculated at various spiral
reinforcement ratios and axial loads for the intermediate/moderate damage level.

Table 6.12 – Rotational ductility at various levels of spiral reinforcement and axial force

<table>
<thead>
<tr>
<th>Axial \ ( \rho_s )</th>
<th>0.0070</th>
<th>0.0098</th>
<th>0.0125</th>
<th>0.0153</th>
<th>0.0180</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.015</td>
<td>3.36</td>
<td>3.29</td>
<td>3.31</td>
<td>3.34</td>
<td>3.36</td>
</tr>
<tr>
<td>0.049</td>
<td>3.12</td>
<td>3.30</td>
<td>3.28</td>
<td>3.30</td>
<td>3.30</td>
</tr>
<tr>
<td>0.083</td>
<td>2.23</td>
<td>2.63</td>
<td>2.63</td>
<td>2.74</td>
<td>2.74</td>
</tr>
<tr>
<td>0.116</td>
<td>2.04</td>
<td>2.48</td>
<td>2.87</td>
<td>2.88</td>
<td>2.90</td>
</tr>
<tr>
<td>0.150</td>
<td>2.04</td>
<td>2.13</td>
<td>2.46</td>
<td>2.80</td>
<td>2.90</td>
</tr>
</tbody>
</table>

Table 6.11 shows rotational ductility ordered in columns according to the amount of spiral reinforcement, and in rows according to the level of axial load. Note that the rotational ductility is higher if the axial load is low, while for higher axial loads, the rotational ductility is greater when more spiral reinforcement is provided.

Table 6.13 shows additional rotational ductilities, calculated at various levels of axial load and concrete strength at the intermediate/moderate damage level.

Table 6.13 – Rotational ductility at various levels of axial load and expected concrete strength

<table>
<thead>
<tr>
<th>Axial \ ( f_{ce}' )</th>
<th>42 [MPa]</th>
<th>52 [MPa]</th>
<th>62 [MPa]</th>
<th>69 [MPa]</th>
<th>83 [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.015</td>
<td>4.35</td>
<td>4.10</td>
<td>3.65</td>
<td>3.31</td>
<td>3.51</td>
</tr>
<tr>
<td>0.049</td>
<td>4.08</td>
<td>3.67</td>
<td>3.42</td>
<td>3.28</td>
<td>3.37</td>
</tr>
<tr>
<td>0.083</td>
<td>3.83</td>
<td>3.39</td>
<td>2.83</td>
<td>2.63</td>
<td>2.83</td>
</tr>
<tr>
<td>0.116</td>
<td>3.34</td>
<td>3.09</td>
<td>2.96</td>
<td>2.87</td>
<td>2.88</td>
</tr>
<tr>
<td>0.150</td>
<td>3.20</td>
<td>2.41</td>
<td>2.66</td>
<td>2.46</td>
<td>2.11</td>
</tr>
</tbody>
</table>
Table 6.13 shows rotational ductility ordered in columns with respect to expected concrete strength, and in rows according to the level of axial load. Note that the highest rotational ductility values are produced with low axial load and low concrete strength.

In Table 6.12, the expected strength of the concrete is taken equal to 69 MPa (10 ksi). In Table 6.13, the spiral reinforcement ratio is taken equal to 0.0125. All other parameters are constant and equal to the values observed in Specimen UW-1 (see Chapter 3), representing typical current construction practice. The longitudinal steel ratio is equal to 0.022, the expected yield stress of the spiral is equal to 552 MPa (80 ksi), the concrete cover is equal to 64 mm (2.5 in.), and the initial prestress in the concrete is equal to 9.7 MPa (1.4 ksi). Tables 6.11 and 6.12 are represented graphically in Figures 6.16 and 6.17.

![Figure 6.16](image-url)  
**Figure 6.16 – Rotational ductility at various levels of spiral reinforcement and axial force**
Figure 6.16 further shows that rotational ductility decreases as axial force increases, and that more spiral reinforcement helps reduce the loss of ductility with increased axial load.

Figure 6.17 also further shows how rotational ductility increases as concrete strength decreases, and again that higher axial load reduces the rotational ductility. In the following paragraphs a more detailed analysis of these results is then undertaken.

6.7 Analysis of the interactions of axial load – spiral Reinforcement and axial load – concrete strength on rotational ductility

The values shown in Tables 6.11 and 6.12, as well as in Figures 6.16 and 6.17, are analyzed in detail in the following set of figures. Figure 6.18 first shows the effect of the
interaction of axial load – spiral reinforcement on rotational ductility.

In Figure 6.18, the main input is normalized axial load, the secondary input is spiral reinforcement ratio, and the output is rotational ductility. The other parameters are taken as constant, as indicated by the values in the table inside the figure. The letter “c” indicates that the moderate/severe damage level is controlled by concrete reaching its limit strain; otherwise, longitudinal steel controls this damage level. As general trends, it can be observed that low axial force has higher rotational ductility, and correspondingly, relatively high axial force has lower rotational ductility. If spiral reinforcement is taken into account at any particular level of axial load, low spiral reinforcement has lower rotational ductility, even though the change due to
different amounts of spiral reinforcement is relatively small. Local changes in the slope of the lines representing levels of spiral reinforcement are also noticeable. The curve with highest slope changes is that with less spiral reinforcement (spiral=0.007), and therefore relatively weak concrete. Its abrupt change at axial load levels between 0.05 and 0.08 is related to concrete (cover and core) between their maximum and residual strengths. At axial load levels higher than 0.08, the rotational ductility is relatively constant. In this state, the cover has completely spalled and the core concrete, which is more stable than the cover, is in charge. It is also observed that the maximum rotational ductility is almost the same for all levels of spiral reinforcement for one specific case of axial loading. In this case, the axial load is its smallest, and all spiral levels are controlled by steel reaching its limit strain (0.05).

In summary, the rotational ductility change according to spiral reinforcement at a fixed axial load level is related to whether the concrete or steel strain limits are reached first. If concrete controls, then the ductility is lower. The curves’ ascent, at higher axial loads, can be associated with the strength characteristics of the core concrete. If the core concrete is strong, it can replace the strength loss produced by cover spalling.

Figure 6.19 again shows the effect of the interaction of axial load – spiral reinforcement on the rotational ductility, but this time with additional data points obtained using PCI Eq. 20.5.4.2.5.2-2 (PCI 2004).
In this figure, the additional data points are obtained using PCI Eq. 20.5.4.2.5.2-2. All the input parameters for this equation are known and constant, except for axial load. Therefore, spiral reinforcement can be calculated at different axial load levels, and then extrapolated between the lines axial load – rotational ductility obtained with different spiral reinforcements in Figure 6.19. Figure 6.19 shows that the rotational ductility associated with the spiral reinforcement obtained using the PCI equation is relatively higher at low axial forces, but that the differences with those values at higher axial forces are relatively small.

Figure 6.20 again shows the interaction of spiral reinforcement – axial load on the rotational ductility.
rotational ductility. However, this time the main input is spiral reinforcement, and the secondary input is axial load.

Figure 6.20 – Effect of the interaction of spiral reinforcement – axial load on rotational ductility

Figure 6.20 is similar to Figure 6.19, since both curves show the same parameters. As general trends, it is expected that at low spiral reinforcement the rotational ductility would be lower, and correspondingly, at high spiral reinforcement it would be higher. Figure 6.20 shows even more clearly that this really is the case. However, a detailed inspection shows, for example, that the line with axial level 0.015 is an exception, since it has a small increase in rotational ductility at low spiral reinforcement. In this particular case, the concrete capacity is not consumed by the small axial load, and so therefore the crucial material is steel. On the other hand, the curves
“Axial 0.15” and “Axial 0.116” have almost all their data points defined by concrete reaching its crucial limiting strain. Other points where concrete reaches its crucial strain are in curves “Axial 0.083” and “Axial 0.049”, at low levels of spiral reinforcement.

Figure 6.21 shows for one more time the effect of the interaction of spiral reinforcement – axial load on the rotational ductility, but with additional data points obtained using PCI Eq. 20.5.4.2.5.2-2.

![Figure 6.21 – Effect of the interaction of spiral reinforcement – axial load on rotational ductility + PCI Eq. 20.5.4.2.5.2-2 data points](image)

The additional data points in Figure 6.21 were obtained by varying the amount of spiral reinforcement in PCI Eq. 20.5.4.2.5.2-2 so that the axial load would coincide with the axial load...
lines in Figure 6.21. In this way, rotational ductility was also found. It can be observed that the PCI spiral reinforcement equation produces relatively similar rotational ductilities across different axial load levels. However, in the cases of relatively high axial loads, these results are very near (case Axial = 0.116) or inside (case Axial = 0.15) the descending branch of the curves of spiral reinforcement vs. rotational ductility.

Now, Figure 6.22 shows the effect of the interaction of axial load – concrete strength on the rotational ductility.

**Figure 6.22 – Effect of the interaction of axial load – concrete strength on rotational ductility**

In this figure, the main input is normalized axial load, the secondary input is concrete compressive strength, and the output is rotational ductility. The other parameters are taken as
constant, as indicated by the values in the table inside the figure. The letter “c” has the same meaning as in previous figures. The sign “c*” means that both the moderate/severe damage level and yielding are controlled by concrete reaching its crucial strains. As general trends, it is expected that low axial load has higher rotational ductility, and correspondingly, high axial load has lower rotational ductility. This is not always the case since concrete at 42 MPa (6 ksi) and 52 MPa (7.5 ksi) have different stress-strain descending branch characteristics than for the other cases. Both of them use a factor $R$ (ratio of increase in strain and stress at peak stress due to confinement) equal to 5, while for the others, $R$ is taken equal to 3. The different shape of the concrete stress-strain curves, coupled with the interaction with the steel, produces a small increase in ductility at an axial load level equal to 0.115, for concrete with 62, 69 and 83 MPa (9, 10 and 12 ksi). Note also that at the highest axial load shown in the figure, the moderate/severe damage level is controlled by concrete, no matter what is the concrete strength. However, only concrete strengths equal to or higher than 62 MPa (9 ksi) are still controlled by concrete reaching its crucial strain, if the axial load ratio is reduced to a level around 10%.

Figure 6.23 again shows the effect of the interaction of axial load – concrete strength on the rotational ductility, but this time with additional data points obtained using PCI Eq. 20.5.4.2.5.2-2.
Figure 6.23 – Effect of the interaction of axial load – concrete strength on rotational ductility + PCI

Eq. 20.5.4.2.5.2-2 data points

The constant parameter values are presented in the table inside the figure. For each level of axial load, using the PCI spiral reinforcement equation, the concrete strength is varied to obtain a constant spiral reinforcement ratio of 0.0125. In this way, the rotational ductility associated with each combination of axial load, concrete strength, and spiral reinforcement, was also found. The rotational ductility values found are relatively similar to one another, and between 2.5 and 3.5. The entire axial load range is covered by the PCI equation data points, but these data points only cover the concrete strength range greater than or equal to 62 MPa (9 ksi), even though similar rotational ductilities can be found for lower concrete strengths.

The same parameters – concrete strength and axial load – are used in the next figure (Fig.
6.24). However, the main input is now concrete strength, and the secondary input is axial load. The output is, as always, rotational ductility. The other parameter values are constant, as indicated in the table inside the figure.

In Figure 6.24, the letter “c” and the sign “c*” have the same meanings as in previous figures. As general trends, it is expected that low concrete strength has higher rotational ductility, and correspondingly, high concrete strength has lower rotational ductility. This is not always the case, since it depends on the interactions between concrete and steel, as well as on the type of concrete model. At the highest axial load, as was observed previously, the moderate/severe damage level is controlled by concrete, no matter what is the concrete strength. In a similar way,
only concrete strengths equal to or higher than 62 MPa (9 ksi) are still controlled by concrete reaching its crucial strain if axial load is reduced to 0.116.

Figure 6.25 again shows the effect of the interaction of concrete strength – axial load on the rotational ductility, but this time with additional data points obtained using PCI Eq. 20.5.4.2.5.2-2.

![Figure 6.25](image_url)

In this figure, for each level of concrete strength, the axial load is varied to obtain a constant spiral reinforcement ratio of 0.0125. In this way, the rotational ductility associated with the different combinations of concrete strength, axial load, and spiral reinforcement was also found. Note that the range of axial load shown in the figure (0.015 – 0.15) was not enough to obtain the
constant spiral reinforcement ratio equal to 0.0125 if concrete strength is less than 62 MPa (9 ksi). Figure 6.25 shows that rotational ductility varies between 2.5 and 3.5 if the PCI spiral reinforcement equation is used. Similar and even higher ductilities could potentially be obtained at different axial loads, but with concrete strength less than 62 MPa (9 ksi).

The data presented, in this and previous sections, shows that the most important parameters affecting the structural behavior of the pile-deck connection are the spiral reinforcement, axial load, and concrete strength. Their interactions are reflected in the variation of the rotational ductility at the pile-deck connection. In addition, it has also been shown that PCI spiral reinforcement equation produced rotational ductilities between 2.5 and 3.5. These results will lead to important conclusions and recommendations presented in Chapter 8.

Finally, it should be noted that the factorial analysis described in this dissertation has shown the importance of the rotational ductility as a parameter to take into account when developing equations for spiral reinforcement amounts. Its explicit inclusion in this type of equation could help to widen the range of parameter combinations, for which the same amount of spiral reinforcement would produce acceptable structural performance levels.
CHAPTER 7: ENVELOPE CURVES FOR USE IN LUMPED PLASTICITY
(CONCENTRATED HINGE) MODELS

The Park and Ang damage index presented in Chapter 4 is used to replace crucial strain values representing damage states in pile-deck connections. An envelope curve is necessary to apply this damage index. In fact, the envelope curve is needed if commercial software only calculates inelastic time-history displacements and rotations, and not strains. This chapter is fundamentally devoted to explain and calculate that curve. The analytical cases used for this purpose are the same as those prepared for the factorial analysis in Chapter 6.

It should be emphasized that the envelope curves developed in this chapter are mainly applicable to the sectional and member structural characteristics described in Chapter 6. Their extension to different sectional and member geometries may still require additional detailed research.

7.1 Plastic hinges for use with the Park and Ang damage index

Plastic hinges model the flexural behavior of a structural element when subjected to actions beyond the elastic range, which occurs in crucial regions (usually at the ends of the element). This behavior is related with the moment capacity of the element at different rotations, and is described through a moment-rotation envelope curve.

There are different alternatives to calculate such an envelope curve. In this section, the envelope curve will follow the cyclic behavior of the structure obtained using fiber section models. To simplify this envelope curve, representative points such as yielding, maximum, and post-maximum capacity will be employed. The yielding point is obtained at first yield of the dowels, the maximum capacity point corresponds to the maximum moment, and the post-maximum capacity point corresponds to the final strong descent from the maximum moment.
The first two points define ascending branches of the strength envelope, and model elastic stiffness and strain-hardening stiffness behavior. The third point marks the end of accelerated strength degradation, such as from confinement failure or bar pullout. It is assumed that there is no residual strength beyond this last point. As an example, Figure 7.1 shows the envelope curve used for Specimen UW-1.

![Figure 7.1 - Envelope curve for Specimen UW-1.](image)

A set of additional rules is necessary to describe the cyclic behavior of this model. The Dowell et al. (1998) set of rules is regularly used in commercial programs such as SAP2000. First, it defines the primary pivot points in force vs. displacement graphs, which control the amount of softening expected with increasing displacement. Then, it defines pinching pivot points, which fix the degree of pinching following a load reversal. The pivot points are found following the slope of the elastic branch of the force-displacement curve, up until a force equal
to “α” times the yield force. The pinching points are found following the elastic branch of the force-displacement curve at a force equal to “β” times the yield force. The value of α is found at the intersection of the initial cracked stiffness and the softened unloading stiffness, defined by a line from the force at displacement ductility 6 through the displacement at zero force. The value of β is found following a load reversal at the intersection of the force-displacement path and the initial cracked stiffness. Preliminary results show good agreement between these two parameters calculated using the fiber-model developed in this dissertation, or with the charts shown in Figure 7.2.

Figure 7.2 - Hysteresis parameters for circular columns: (a) α parameter; and (b) β parameter

(source: Dowell et al. 1998)
The charts shown in Figure 7.2 relate axial load and longitudinal steel ratios with the parameters \( \alpha \) and \( \beta \). These charts were obtained from multiple analyses of circular reinforced concrete columns using a simplified fiber model (Dowell et al. 1998). The agreement between the fiber model and the chart values confirms that the behavior of columns controlled by bending is similar to the behavior of a pile-deck connection.

The hysteretic response follows the backbone curve as long as no displacement reversal occurs. Once the yield displacement has been exceeded, a subsequent strength envelope is developed. It is defined by lines joining the previous maximum displacement point and the pinching point described in a previous paragraph. The unloading branches are directed towards the pivot points, see Figure 7.3.

![Figure 7.3 - Pivot and pinching points in the concentrated hinge model (source: Dowell et al. 1998).](image)
Finally, and as an example, Specimen UW-1’s moment-rotation curve, obtained using the fiber model and the concentrated hinge model as described above (and as applied in SAP2000), is shown in Figure 7.4.

Figure 7.4 shows the moment-rotation curve using the concentrated hinge (“ch”) model for Specimen UW-1, as well as the curve using the analytical (“a”) model developed in this dissertation. Both curves compare quite well with respect to their shape and magnitude, as well as with respect to the position of the Park and Ang damage scale values 0.1, 0.25, 0.4, 0.8 that define levels of damage 1, 2, 3 and 4, respectively. Further analysis is required to generalize
these initial observations. An initial attempt to calculate the backbone curve for pile-deck connections will be explained in the following sections.

### 7.2 General lumped plasticity models

Even though continuum (finite element) models are the more advanced analytical methods since they seek to model the underlying physics of the materials and elements, in practice, they do require some phenomenological calibration to account for behavior not captured by their formulations. Accuracy, besides simplicity, can also be reached by phenomenological models such as the concentrated hinge model and the fiber model. Therefore, current practice for modeling relies more on these last two types. For example, ASCE/SEI 41 with its Supplement 1 (Elwood et al. 2007), which evolved from FEMA-273 (Building Seismic Safety Council 1997) and FEMA-356 (Building Seismic Safety Council 2000), uses a concentrated hinge model. In this type of model a fundamental component is the backbone (envelope) curve that relates normalized moment (flexural to yield moment ratio) to rotation in the concentrated hinge. In ASCE/SEI 41, this backbone curve is that obtained directly from experimental cyclic data incorporating cyclic strength and stiffness degradation. The key features of this backbone curve are illustrated in Fig. 7.5.

![Figure 7.5 - ASCE/SEI 41 backbone curve](image)
In Figure 7.5, linear response is displayed between point A (component with zero force) and an effective yield point B. The slope from point B to point C could represent phenomena such as strain hardening. Point C is a point at which fast lateral strength degradation begins. This is considered to occur when the shear capacity of the section falls 20% from its maximum capacity. Beyond point D, which indicates the end of quick lateral strength loss, the structural component continues responding with markedly reduced strength to point E. This last point marks off the maximum rotation/displacement that the structure can reach without collapse. Values for the parameters a, b and c can be obtained from tables. For column type and flexure controlled components, these parameters depend on the amount of transverse reinforcement and axial force. In addition, to avoid computational instability, a small slope (10 vertical to 1 horizontal) may be provided to the segment between points C and D. ASCE/SEI 41 also states that “representation of the load-deformation relation by points A, B and C only (rather than all points A-E), shall be permitted if the calculated response does not exceed point C.”

In addition, ASCE/SEI 41 selected point C to achieve a probability of failure less than 35% (the probability distribution is assumed to be log normal). Point E was selected to achieve a probability of failure less than 15%. These probabilities of failure were established by ASCE/SEI 41 based on judgment regarding the consequence of each failure mode, and do not consider the uncertainty in the ground motion or in structural analysis.

The ASCE/SEI 41 generalized force versus deformation curve could represent two types of ductile behavior:

Type 1 curve, illustrated in Figure 7.6, shows an elastic range (points 0 to 1), a plastic range (points 1 to 3), followed by loss of lateral-force-resisting capacity at point 3 and loss of vertical-force-resisting capacity at point 4. Point-2’s deformation should be greater than or equal
to two times deformation at point 1.

Type 2 curve, also illustrated in Figure 7.6, shows the same deformation ranges and points of Type 1 curve, but the deformation of points 2 and 3 coincide. Point-3’s deformation should be greater than or equal to two times deformation at point 1.

![Figure 7.6 - Types of ductile behavior (source: ASCE/SEI 41)](image)

7.2.1 Predecessors and ASCE/SEI 41 curves

When the ASCE/SEI 41 predecessor documents were developed, there were limited data available on the performance of existing structures, and reliability concepts were not evenly applied in the development of the criteria (ASCE/SEI 41). As a result, criteria specially related to deformation capacities, tend to err on the conservative side. Anecdotal reports from practicing engineers suggest that when the criteria have been applied to older reinforced concrete buildings, most do not pass the collapse prevention limits set out in these predecessor documents. As an example of the differences between ASCE/SEI 41 Supplement 1 and one of its predecessors (FEMA 356), Figure 7.7 shows their curves for columns under low axial force. FEMA 356 does not consider any difference if the spiral reinforcement ratio is low or high, but ASCE/SEI 41 does consider it.
In Figure 7.7, it is assumed that the ratio M/My at points C, C’, and/or C” is 1.13 and that points D, D’, D” and E, E’, E” coincide, respectively. The residual moment in all these curves is 20% of the yield moment.

Figure 7.7 shows that FEMA 356 curve is similar to ASCE/SEI 41 curve using a low level of spiral reinforcement, but differs markedly from the ASCE/SEI 41 curve using a high level of spiral reinforcement.

7.3 Description of the methodology employed to create new backbones moment-rotation

In Chapter 6, a total of 128 analytical cases were used to describe the structural behavior of the pile-deck connection. Seven structural parameters, with minimum and maximum values, were employed: spiral reinforcement, vertical load, concrete strength, longitudinal reinforcement, spiral yield stress, cover, and initial prestress. The first four of them were the
most important parameters to describe the pile-deck connection. Note that ASCE/SEI 41 also uses the first two parameters (spiral reinforcement and vertical load) in its moment-rotation envelope curves. In this chapter, the 128 analytical cases developed in Chapter 6 are taken as the data base to create new backbones moment-rotation. Initially, and following ASCE/SEI 41, the amount of spiral reinforcement and initial vertical load are the parameters that define these backbones.

Four groups (32 analytical cases in each group) are distinguished according to the level of spiral reinforcement and initial vertical load:

1. Minimum spiral reinforcement and minimum initial vertical load.
2. Maximum spiral reinforcement and minimum initial vertical load.
3. Minimum spiral reinforcement and maximum initial vertical load.
4. Maximum spiral reinforcement and maximum initial vertical load.

For each of these four groups, a simplified average curve will be obtained. This curve will follow three basic principles: represent accurately the structural behavior of the initial cases, be simple, and easy to apply. One of the most important points in this curve is yielding, since it identifies the end of the elastic and the beginning of the plastic zones. However, the concrete elastic behavior, which has a parabolic stress-strain shape, makes necessary the use of one additional point before yielding. In addition, the peak moment before cover spalling is clearly identified in many of the initial analytical cases, due to the relatively important reduction in moment capacity after cover spalling. Therefore, it will be included in the proposed backbone curve. Finally, a point that describes moderate/severe damage, for instance MOTEMS Damage Level 2, is also important to be included in the backbone curve, since it is fundamental in the performance based design of pile-deck connections.

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7.4 Numerical envelopes

Figures 7.8, 7.9, 7.10 and 7.11 will show the envelopes of the 32 original analytical cases of the four groups described in the previous section. These curves are shown with rotations up to the point were the first computational convergence problems occur in any of the 32 cases. The corresponding ASCE/SEI curve is shown in each figure too. This last curve is drawn as an elastic perfectly plastic curve. The proposed backbone curve is also shown in each of these figures. In this way, it can be compared with the ASCE/SEI and the original analytical curves. In addition, the MOTEMS Level 2 damage point is drawn on top of the proposed backbone curve (it has the shape of a diamond). The detailed description of the proposed backbone curve is deferred to section 7.5.

Figure 7.8 shows the backbone curve for the case “minimum spiral reinforcement and minimum initial vertical load”.
Figure 7.8 shows that the 32 initial envelope curves are quite similar in the elastic range, and differ moderately in the plastic range. It can also be seen that the ASCE/SEI curve is a conservative low limit of these curves. However, in the performance based design, as explained in Chapter 5, mean values are necessary since they avoid an excessively conservative design of the capacity protected structural elements. Note that the proposed backbone curve follows closely the group of 32 envelope curves drawn in Figure 7.8, since it uses mean values, and additional points such as those representing the initial elastic behavior and the peak moment before concrete spalling.

Figure 7.9 shows the backbone curve for the case “maximum spiral reinforcement and minimum initial vertical load”
minimum initial vertical load”.

Figure 7.9 – Backbone curves for the case “maximum spiral reinforcement and minimum initial vertical load”

Figure 7.9 shows similar behavior than that observed in the previous case. However, the last line segment in the proposed backbone curve has a smaller slope than in the previous case. In this case, the ASCE/SEI curve represents a conservative low limit too.

Figure 7.10 shows the backbone curve for the case “minimum spiral reinforcement and maximum initial vertical load”.

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Figure 7.10 - Backbone curves for the case “minimum spiral reinforcement and maximum initial vertical load”

Figure 7.10 illustrates that in this case, relatively high moments cannot be sustained if the rotation increases. It also illustrates that particular groups of envelope curves have different peak points. However, the peak points are conservatively captured by the ASCE/SEI 41 and the proposed backbone curve. At the ultimate point, nonetheless, ASCE/SEI 41 clearly is not a conservative limit. In this particular case, the proposed envelope curve constitutes a much better fit than the ASCE/SEI 41 curve. Note for instance that the point at ultimate rotation in the envelope curves, and in the proposed backbone, has a smaller moment capacity than in the ASCE/SEI curve.

Figure 7.11 shows the backbone curve for the case “maximum spiral reinforcement and
maximum initial vertical load”.

Figure 7.11 - Backbone curves for the case “maximum spiral reinforcement and maximum initial vertical load”

Note that at least 4 groups of envelope curves have been considered (four different ultimate rotations are clearly visible). These groups take into account not only spiral reinforcement and axial load, but the concrete strength and amount of longitudinal reinforcement. Even though they are not as visible as in Figure 7.11, they also had impact on the results presented in Figure 7.10. Note that they affect not only the ultimate rotation (some curves have much smaller ultimate rotation than others), but the peak moment before cover spalling (this point can be easily identified in some curves but not in others). More detailed curves are needed to capture accurately these two characteristics.
In Table 7.1, a summary is given of the point co-ordinates of the backbone curves described above.

Table 7.1 – Point co-ordinates rotation [rad] versus normalized moment [M/My] of the backbone curves

<table>
<thead>
<tr>
<th></th>
<th>Point 1</th>
<th>Point 2</th>
<th>Point 3</th>
<th>Point 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho_{min}P_{min}$</td>
<td>(0.0033,0.65)</td>
<td>(0.0090,1.00)</td>
<td>(0.0198,1.13)</td>
<td>(0.0311,1.01)</td>
</tr>
<tr>
<td>$\rho_{max}P_{min}$</td>
<td>(0.0032,0.64)</td>
<td>(0.0088,1.00)</td>
<td>(0.0209,1.13)</td>
<td>(0.0330,1.12)</td>
</tr>
<tr>
<td>$\rho_{min}P_{max}$</td>
<td>(0.0021,0.53)</td>
<td>(0.0104,1.00)</td>
<td>(0.0165,1.07)</td>
<td>(0.0231,0.81)</td>
</tr>
<tr>
<td>$\rho_{max}P_{max}$</td>
<td>(0.0021,0.53)</td>
<td>(0.0105,1.00)</td>
<td>(0.0160,1.08)</td>
<td>(0.0426,0.92)</td>
</tr>
</tbody>
</table>

Table 7.1 shows rotations and normalized moments at points defining the proposed backbone curve depending on the amount of spiral reinforcement and axial load (min/max). The rotation and moment at the first point (Point 1), the yield point (Point 2) and the maximum moment before spalling point (Point 3) are only substantially affected by the axial load and not by the spiral reinforcement. In contrast, the ultimate rotation point (Point 4) depends on both the axial load and the spiral reinforcement. In case of relatively high axial load, the moment capacity at this point is below the yielding moment. Note also that rotation at Point 4 is the highest in the case with maximum spiral reinforcement and axial force, even though its moment capacity is reduced compared to the case with low axial load.

More detailed normalized moment versus rotation curves are needed to represent accurately the structural behavior in case of maximum axial load and spiral reinforcement, as has already been noted in Figure 7.11. Two new parameters, concrete compressive strength and longitudinal reinforcement, which were also found to be important to describe the pile-deck
connection structural behavior, are taken into account in the following graphs.

Figure 7.12 shows the four backbone curves for the case “maximum spiral reinforcement, maximum initial vertical load” (S+A+) and the additional combinations of minimum (f’c-) and maximum (f’c+) concrete strength, as well as minimum (Asl-) and maximum (Asl+) amount of longitudinal reinforcement.

Figure 7.12 – Subcategories of the case “maximum spiral reinforcement, maximum initial vertical load” taking into account minimum/maximum concrete strength and longitudinal reinforcement

It is observed that the peak moment before cover spalling and the ultimate rotation are relatively well captured by the graphs considering different concrete strength and longitudinal reinforcement. MOTEMS damage level 2 has always been captured by the proposed backbone
curves. However, note that in the cases with high concrete strength MOTEMS damage level 2 is almost at the end of the proposed backbones. It is also important to highlight that in the cases with high concrete strength the ASCE/SEI curves do not constitute a conservative low limit.

In Table 7.2, a summary is given of the point co-ordinates of the backbone curves described above.

**Table 7.2 – Point co-ordinates rotation [rad] versus normalized moment [M/My] of the curves with maximum axial load and spiral reinforcement**

<table>
<thead>
<tr>
<th></th>
<th>Point 1</th>
<th>Point 2</th>
<th>Point 3</th>
<th>Point 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \rho_{max}, P_{max} )</td>
<td>( f_{c_{min}} \cdot A_{sl_{min}} )</td>
<td>( f_{c\ell_{max}} \cdot A_{sl_{min}} )</td>
<td>( f_{c_{min}} \cdot A_{sl_{max}} )</td>
<td>( f_{c\ell_{max}} \cdot A_{sl_{max}} )</td>
</tr>
<tr>
<td>( f_{c_{min}}, A_{sl_{min}} )</td>
<td>(0.0018,0.51)</td>
<td>(0.0104,1.00)</td>
<td>(0.0162,1.06)</td>
<td>(0.0480,0.99)</td>
</tr>
<tr>
<td>( f_{c\ell_{max}}, A_{sl_{min}} )</td>
<td>(0.0025,0.61)</td>
<td>(0.0127,1.00)</td>
<td>(0.0190,1.06)</td>
<td>(0.0340,0.74)</td>
</tr>
<tr>
<td>( f_{c_{min}}, A_{sl_{max}} )</td>
<td>(0.0017,0.43)</td>
<td>(0.0081,1.00)</td>
<td>(0.0124,1.11)</td>
<td>(0.0606,1.13)</td>
</tr>
<tr>
<td>( f_{c\ell_{max}}, A_{sl_{max}} )</td>
<td>(0.0023,0.54)</td>
<td>(0.0109,1.00)</td>
<td>(0.0165,1.08)</td>
<td>(0.0278,0.89)</td>
</tr>
</tbody>
</table>

Table 7.2 shows rotations and normalized moments at the proposed envelope curves depending on the min/max concrete strength and longitudinal reinforcement, with maximum amount of spiral reinforcement and axial load. The rotation at the elastic point (Point 1) varies depending on the concrete strength, but its moment varies slightly depending also on the longitudinal reinforcement. On the other hand, the yield point (Point 2) is affected fundamentally by the amount of longitudinal reinforcement. But it also changes moderately depending on concrete strength. It reaches higher rotations with maximum concrete strength and minimum longitudinal reinforcement. The point of peak moment before concrete spalling (Point 3) shows mixed results. Its rotation is the highest with maximum concrete strength and minimum longitudinal reinforcement. But its peak moment, compared to the yield moment, occurs at minimum concrete strength.
strength and maximum longitudinal reinforcement. Finally, it is well known that the rotation at the ultimate point (Point 4) depends heavily on the spiral reinforcement. Higher rotations are reached if the spiral reinforcement is at its maximum. Table 7.2 shows that this effect is more important with minimum concrete strength.

A comparison between these curves and their mean is shown in Figure 7.13. In this figure, four backbone curves, obtained with varying concrete strength and longitudinal reinforcement, are shown and compared with their mean.

![Figure 7.13 – Mean backbone curves for the case “maximum spiral reinforcement, maximum initial vertical load”](image)

In Figure 7.13, the most important difference is between backbones with high and low concrete strength. In the case of low concrete strength, the pile-deck connection has higher ultimate
rotation and moment capacity. Note also that longitudinal reinforcement has relatively limited influence on these rotation-normalized moment backbones.

7.5 Description of the model employed

Based on the methodology and results presented in previous sections, the proposed backbone is described here. The main idea of the ASCE/SEI 41 model, curves moment versus rotation with the moment normalized with respect to the yield moment, is conserved; but all other characteristics are modified to better capture the shape of the moment-rotation curves in pile-deck connections, and to take into account the computational limitations of the model. First of all, a point at 30% of the strain at peak concrete strength of unconfined concrete is used to represent the end of the initial elastic behavior. The second point is the yielding point, defined as the point at which steel closest to the tension face of the section yields or concrete reaches a strain equal to the strain at peak unconfined concrete strength, whichever occurs first. The third point is peak moment capacity before initial cover spalling. The fourth point is the maximum rotation that can be achieved numerically without convergence problems. It should be noted that this point always occurs at higher rotations than the moderate/severe damage level (MOTEMS damage level 2). Additional points, similar to D and E in ASCE/SEI 41 are not included due to convergence problems at relatively high rotations. The backbone points are calculated as the mean value of the corresponding points in all the curves rotation versus normalized moment being considered, except in the case of the point at maximum rotation, which is the smallest value without convergence problems.

This model has the advantage to accurately represent the mean structural behavior of the pile-deck connection with different amounts of spiral reinforcement, vertical load, and even, for certain particular cases, concrete strength, and amount of longitudinal reinforcement. For
simplicity, following ASCE/SEI 41, yielding has been maintained as the reference point of the 
elastic and plastic zones. Even though the proposed backbone has included two additional points 
(30% of the strain at peak concrete strength of unconfined concrete, and peak moment before 
cover spalling), it is still relatively simple to understand and apply. Finally, as explained 
previously, the point of maximum rotation can capture important levels of damage such as 
MOTEMS Damage Level 2.

Note that these envelope curves should be used to generate the rotation-moment backbone required in concentrated plastic hinge models. Practitioners need first to calculate the 
yield moment at the pile head of their particular pile-deck configuration. This can be attained in a 
moment-curvature analysis of the pile head section. Then, they should check the amount of 
longitudinal reinforcement that they are using in their piles, and interpolate this value between 
the maximum and minimum given in the developed envelope curves (cases S+A+). This 
calculation should be done twice, since there are two pairs of envelope curves depending on the 
amount of concrete strength. A similar process is now carried on using the axial load and the 
spiral reinforcement parameters. In this way, the moment relative to yielding can be obtained for 
each critical point defined in the envelope curves. Then, the moments in the backbone curve are 
obtained multiplying the interpolated relative moments by the calculated yield moment. Since 
this process is mechanical, it can be implemented in a spreadsheet.
CHAPTER 8: CONCLUSIONS AND RECOMMENDATIONS

This chapter summarizes the work developed in this dissertation, provides conclusions and recommendations regarding its findings, and finally suggests some areas for additional research.

8.1 Summary

This dissertation aimed to develop a simple but accurate analytical model of the pile-deck connection in a wharf supported on piles. Then, this model was used to examine alternative techniques to describe damage and to estimate the effect of different parameters on the rotation capacity of the pile-deck connection under cyclic lateral loading. In particular this dissertation studied the following key structural problems: (i) development and validation of optimal computational model for the pile-deck connection; (ii) alternative damage indexes to describe and characterize the pile-deck connection nonlinear behavior; (iii) importance of the interactions among structural parameters at the pile-deck connection.

The pile-deck connection was modeled using the fiber approach in the software OpenSees with carefully detailing of the pile sectional behavior including cover concrete, confinement of the core concrete, prestress, low-cycle fatigue and even buckling of the longitudinal reinforcement. The slip of the rebars phenomenon, especially important in this pile-deck connection, was shown independent of other flexural related behavior through a “rotational” spring. Specimens from the University of Washington and the University of California at San Diego validated the analytical results.

Since damage assessment is critical to estimate the structural behavior of the pile-deck connection, an effort was done to study this topic. In fact, damage indexes, such as Park and Ang (DPA), were used as an alternative to MOTEMS limit strains for damage assessment. Not only
DPA but other similar damage indexes give reasonable results in specimen UW-1 from University of Washington, and specimen UCSD-2 from University of California at San Diego. However, DPA was preferred due to its extended supporting data base. DPA and MOTEMS analytical results were compared; suggesting that MOTEMS strain limits can be replaced by DPA damage index. At the end of Chapter 4, a methodology to exclusively use DPA was also proposed.

A more refined model was deemed necessary to include important parameters that could have an impact on the member behavior of the pile-deck connection. In Chapter 5, a special substructure was developed to study the effect of in-ground hinges on the displacement capacity of the wharf, and also the effect of varying axial loads on the pile head capacity. It consisted of three piles, the soil around them, and the deck at top of the three piles. This substructure represented the initial land side portion of the wharf that usually concentrates the majority of the seismic damage. It was assigned geometry and loads similar to those of a typical wharf structure in the west coast of the United States (Berth 100 in POLA). Finally, the effect of the soil parameters on the structural behavior of the wharf substructure was estimated using minimum and maximum soil stiffness values according to recommendations from MOTEMS.

Since code equations vary considerably when calculating spiral reinforcement, a special study was deemed necessary to clarify its interactions with other structural parameters in pile-deck connections. Seven structural parameters were selected based on the literature review and experimental research results. Initially, the effect of each parameter and their relationships were studied using a factorial analysis. Then, the most important interactions between axial load and spiral reinforcement, as well as axial load and concrete strength were illustrated.

Finally, moment versus rotation envelope-curves were developed to help practitioners in
modeling the inelastic structural behavior of the pile-deck connection. The basic idea in the ASCE/SEI 41 format was maintained, meaning graphs moment normalized to the yield moment versus rotation for minimum/maximum spiral reinforcement and axial load. Additional points at the elastic region and at the peak moment before concrete spalling were drawn to obtain an accurate envelope curve. Due to computational convergence problems only points with rotations slightly larger than the rotation at moderate/severe damage level (MOTEMS damage level 2) were also drawn. The additional effect of parameters such as concrete strength and longitudinal reinforcement were considered in cases with high axial load.

8.2 Conclusions and recommendations

Pile-deck connections are crucial points that define the seismic behavior of port structures like wharves supported on piles. Therefore, an analytical model was developed that captures satisfactorily all important aspects of their structural behavior including peak strength, hysteretic energy, and progressive degradation in the strength even at high levels of damage. Close comparison between the analytical and experimental behaviors of four full-scale specimens obtained from previous research at the University of Washington (specimens UW-1 and UW-2) and University of California San Diego (specimens SD-1 and SD-2) revealed good agreement on the sectional level (moment vs. curvature) and the component level (moment vs. displacement).

Moreover, the analytical model developed in this dissertation can predict a variable dowel-slip contribution to the total rotation and displacement of the pile. The model showed that this contribution increases until cover spalling and initial damage in the core concrete occurs. Then, its relative importance decreases, as the typical flexural failure mechanism takes control of the general structural behavior at the end zone of the pile.

The good agreement between the experimental and analytical results suggests that the
proposed analytical modeling approach for pile-deck connections could be used in describing accurately the cyclic behavior and damage progression in the connections under large drift demands.

In this dissertation it was also shown that the crucial strains calculated in the model are indeed related to moderate and severe levels of observed damage. Note that usual damage scales were enriched with additional observations from the experimental research. This enriched scale was also compared to MOTEMS’ damage scale. It can be considered that MOTEMS’ level 1 of structural performance is equivalent to the limit between minor and moderate observed damage. MOTEMS’ level 2 of structural performance is at the beginning of the severe observed damage range.

The most appropriate damage index for pile-deck connections is determined as the Park and Ang model. It was compared to five additional indexes, to establish accurately its advantages and disadvantages. Park/Ang, Daali and Jeong/Elnashai damage indexes are similar between them, since they are based on the same ideas that combine displacement and energy parameters to calculate damage. Hwang/Scribner and Hindi/Sexsmith damage indexes are also similar between them, since they are based on energy principles. They gave lower damage indexes than the previous group. Finally, Kunnath damage index is based on low-cyclic fatigue and its values are the lowest compared to the other damage indexes.

The first group of damage indexes (Park/Ang, Daali and Jeong/Elnashai) had relatively good agreement with the observed damage, especially at the moderate damage range. However, Park and Ang damage index is relatively easy to apply, and it is the only model that has been extensively calibrated with observed damage. The good agreement between observed damage and Park and Ang damage index, its easiness to apply, and the extensive experimental data base,
in which it is supported, are definitively the main reasons that this damage index is preferred to other relatively similar methods.

Finally, the results obtained using MOTEMS damage levels and Park and Ang damage index were compared. MOTEMS’ damage level 1 can be situated between Park and Ang damage index 0.1 and 0.25. MOTEMS’ damage level 2, on the other hand, can be situated between Park and Ang damage index 0.25 and 0.4.

In the specific experimental cases studied in this dissertation, Park and Ang damage indexes equivalent to MOTEMS crucial strains could be used as a simplified measure of damage for performance based design of pile-deck connections. However, analytical results from the parametric analysis in Chapter 6 showed that traditional damage index limits are only comparable to MOTEMS strain limits in cases with regular concrete strength.

In the special substructure developed in this dissertation, under extensive cyclic loading, the two more rigid piles (landside piles) suffered MOTEMS’ damage level 1 and 2 at the pile-deck connection. In contrast, the pile zones inside the ground, and with high moments, only reach MOTEMS’ damage level 1. In fact, the moment capacity of the pile head was lower than that at the pile body. In addition, the curvature demand at the pile head was higher than that at the pile body. Finally, the ratio demand versus yield curvature at the pile head and body is similar up to a displacement around 0.05 m (2 in.), and then it is much higher at the pile head.

All these characteristics show that damage was concentrated at the pile head and that damage at the pile body was relatively minor. Finally, the effect of varying the soil stiffness values according to recommendations from MOTEMS was minor in the specific case developed in this dissertation.

In the factorial analysis, the moderate/severe damage level was defined using carefully
calculated crucial strains. The most important differences with regularly recommended MOTEMS strains are in the concrete strains. These differences are especially important for high-strength concrete.

The initial factorial analysis conducted on minimum and maximum values for seven structural parameters at the pile-deck connection and at the moderate/severe damage level revealed relationships among four of them, namely, concrete strength, axial force, longitudinal reinforcement and spiral reinforcement. These results were obtained using two alternatives for the factorial analysis, and confirmed with a regression analysis. Additional confirmation was based on spiral reinforcement code equations use of the same parameters. Finally, the two most important interactions between axial force and spiral reinforcement as well as axial force and concrete strength, revealed general trends such as increments in the rotational ductility if the spiral reinforcement is higher, or decrements if the concrete strength is higher. It also revealed local important variations in these trends, due to the effect of the axial load on the interaction between concrete and steel materials. Note also that the definitions of damage states as function of strains in the concrete or steel materials produced relatively abrupt changes when the control of the section changed from one material to the other.

The rotational ductility range of values obtained in this work is between 2.0 and 4.5, which is covers the range obtained using PCI Eq. 20.5.4.2.5.2-2. (2.5 to 3.5). However, similar rotational ductilities can be obtained using parameter values outside the range covered by the PCI spiral reinforcement equation. Re-ordering the parameters used in the PCI’s spiral equation, in a new equation, could help to cover other possibilities than those enclosed by the PCI’s equation.

The moment-rotation envelope curves were based on the methodology proposed by ASCE/SEI 41. The suggested additional points, at the elastic zone and at the peak moment
before spalling, accurately described the envelopes of the curves with different axial load and spiral reinforcement. Even though it was not possible to assess points at collapse of the structure due to computational convergence problems, rotations at damage levels up to slightly more than MOTEMS damage level 2 were still captured. Envelope curves for the cases of maximum/minimum spiral reinforcement and low axial force described accurately the behavior observed in the model. In the special cases with high axial load, better results were obtained if concrete strength and longitudinal reinforcement were added as parameters that define the envelope curves.

**8.3 Future work**

The work developed in this dissertation has aimed to answer a few key questions regarding the structural behavior of the pile-deck connection, to estimate its damage and to describe the relationships between its different parameters. However, many other questions have arisen that still need to be answered in future works. Some of these questions are summarized in the following paragraphs.

Damage in the common pile-deck connection described in this dissertation is controlled by bending and slip of the rebars. However, shear could play an important role in cases such as piles with reduced length-to-diameter ratio. In fiber models, shear could be introduced through an additional rotational spring, this time controlled by shear crucial conditions. Its structural behavior still needs to be analytically assessed and experimentally validated.

The use of damage indexes to estimate damage has revealed that the Park and Ang damage index is a good damage predictor. However, it showed acceptable values if concrete has regular strength but not for high strength concrete. Further experimental research is needed to validate possible modifications in the Park and Ang damage index, so it could be applied for
high-strength concrete.

The model developed in this dissertation does not show any effect due to in-ground hinge development. However, different wharf geometries and soil properties still need to be assessed regarding its effect on the in-ground hinge. Parameters such as bay length, pile height above the ground level, deck thickness, pile diameter, as well as number, thickness, and type of soils, among other parameters, are needed for a complete analysis of the pile-deck behavior.

Additional research is needed to propose and validate modifications to PCI Eq. 20.5.4.2.5.2-2 taking into account the rotational ductility. The work developed in this dissertation was limited to octagonal sections with a diameter of 0.62 m (24 in.) and with a fixed number of dowel bars (8) distributed regularly around the perimeter of the pile core concrete. Furthermore, the concrete cover values are applicable only to the specific pile section described above.

The applicability of the envelope curves developed in Chapter 7 is also limited to the sectional and member structural characteristics described in Chapters 5 and 6.

Finally, the ability to estimate accurately the structural damage at systems such as those described in this dissertation, could have potentially an impact on the cost estimation of the construction life cycle of port structures, and on the retrofit techniques recommended after seismic attacks. Methodologies such as those employed in this dissertation to describe the pile-deck connection structural behavior would eventually lead to optimal technical and economical solutions, and therefore will be increasingly accepted by the professional community.
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