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SIMULATED SEISMIC PERFORMANCE OF PILE EMBEDMENT LENGTH ON PILE-TO-PILE CAP CONNECTIONS

Mochamad Taguh

ABSTRACT: Potential plastic hinges on pile-to-pile cap connection may not be avoidable when strong ground motion due to a moderate earthquake shakes structural elements. Consequently, severely crack damage propagates a critical zone at the interface between pile head and pile cap producing a plastic hinge at the connection. The pile head damage is essentially dependent on the performance of pile embedment length, pile embedment model, confinement ratio, and headed reinforcement ratio. This paper presents an investigation on seismic behavior of the pile-to-pile cap connection developing a variety of pile embedment lengths. Advanced finite element models (FEM) of the pile embedment length of prestressed concrete pile-to-pile cap connections were carried out taking into account sophisticated constitutive material models, material plasticity, bond-slip and perfect bond between reinforcing steels and concrete in both the pre- and post-yield ranges. The result shows that the FEM analyses can capture the load and deformation relationship and load carrying capacity of the octagonal-preserved concrete pile-to-pile cap connection satisfactorily. The FEM accurately performs crack damage propagation at the certain area of plastic hinges providing better understanding on the seismic behavior of pile embedment length in the connection.

Keywords: Finite element model, pile embedment length, prestressed concrete pile, seismic behavior, plastic hinge, constitutive material model

INTRODUCTION

Pile foundation systems, in general, will be subjected to large lateral loads in addition to the normal gravity loads from the superstructure during severe earthquake attack. Ground movement as response of earthquake events leads to plastic hinges forming in the piles near their connections to the pile cap and along the pile length. This produces more conservative in the design of pile foundation systems, especially where prestressed concrete piles connected to the reinforced concrete pile cap are concerned. The most critical regions of the pile are at the interface of the pile-to-pile cap connection because of the fricty of the connection and pile regions embedded in the soil, where interfaces between hard and soft soil exist.

Under lateral seismic loads, the fricty at the pile-to-pile cap connection induces a large curvature demand at the pile head, with potential for severe damage or failure of the pile. For a laterally loaded fixed-head pile, serial yielding of the pile occurs until a plastic mechanism is fully developed (Figure 1).

It is obvious that difficulties associated with the repair of foundation damage make it desirable to design the piles to remain undamaged during severe earthquakes. A recent international design practice recommends joint details that are heavily reinforced. Such reinforcement produces in congested steel in the pile cap and is extremely difficult to construct.

The foundation system should be provided with sufficient strength to ensure that as far as possible it remains in the elastic range, while energy dissipation occurs elsewhere at the chosen yielding locations during a severe earthquake. However, uncertainties exist with regard to soil-structure interaction and the resulting actual pile behavior during a severe earthquake, and it would appear to be essential to detail piles that are capable of a reasonable degree of ductile behavior (Park et al. 1984). The ductile design approach is commonly used for bridge piers to provide sufficient resistance during earthquake. In this approach, seismic energy is dissipated by the formation of plastic hinges at the chosen regions of the piers and the ductility capacity of the plastic hinges in the piers should always be greater than the ductility demand for the chosen design earthquake spectrum (Pam 1988). The piles should also possess adequate ductility.

Fixity at the pile-to-pile cap connections (Figure 1) is more desirable in order to provide proper

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transfer of forces from the bridge piers to the piles. This fixity produces a large ductility demand at the connection during seismic attack. Careful detailing of the connection must be constructed and transverse reinforcement should be provided to meet the strength and ductility demand. The parameters investigated in this study include material and reinforcement details, pile embedment models, and pile embedment length. The region of the pile cap around the connection needs special attention, because damage in the connection may penetrate into the pile cap. In addition to the regions sensitive to seismic loading, the top and tip of piles also require sufficient confinement ratio since during driving high impact loading is applied to the pile and the pile tip needs to penetrate into layers of soil and often hard layers or boulders are encountered.

![Figure 1. Deformed pile at each limit state and bending moment distribution due to a laterally loaded fixed-head pile (Song et al. 2005).](image)

This paper presents current studies in the development of seismic performance on prestressed concrete pile-to-pile cap connections focusing on the variability of pile embedment lengths applied to headed embedment models (HEM) of the pile-to-pile cap connection. This study simulates seismic performance of the connection by providing sufficient strength and ductility. The seismic performance of two pile-to-pile cap connection (PPC) models was investigated incorporating various pile embedment lengths and compressive strengths of concrete. Model analyses were carried out utilizing an advanced finite element program.

**STRENGTH AND DUCTILITY OF PRESTRESSED CONCRETE PILE**

In the analytical analysis, it was assumed that the pile-to-pile cap connection was experiencing a severe earthquake. The seismic action to simulate the structure was employed static axial and monotonic, and cyclic lateral loads. The strength and ductility of each pile were computed based on its pile section. In the nonlinear sectional analysis of pile, the stress-strain relationship for both unconfined and confined concrete was used to calculate moment-curvature relationships for the prestressed concrete piles with variations of cross-section and reinforcement ratio producing the strength and curvature ductility of the piles proportionately.

The Modified Compression Field Theory (MCFT) developed by Vecchio and Collins (1986) was adopted in this analysis. In this model, the cracked concrete is treated as a new material with its own stress-strain characteristics. Equilibrium, compatibility, and stress-strain relationships are formulated in terms of average stresses and average strains. While the original compression field theory ignored tension in the cracked concrete, this model takes into account tensile stresses in the concrete between the cracks, and employs experimentally verified average stress and average strain relationships for the cracked concrete.

![Figure 2. PPC Models (plain and headed models) description](image)

To investigate the seismic behavior of the proposed pile-to-pile cap connections, the effect of spiral confinement on the strength and ductility of the pile was studied to analyze the stress-strain relation and moment-curvature relation. In an attempt to increase the strength and curvature ductility, three different spiral confinement pitches, namely, L1, L2, and L3, were applied in each type of pile section and provided a variety of curvature ductility (Teguh 2010). Each pile was reinforced with eight 7 mm wire tendons and confined with 4 mm spiral reinforcement and the spiral pitch was varied from 45 mm to 100 mm along the pile length (Figure 2). To
cater for a significant change in the moment-curvature relationships for each pile. Axial loads of 0.1 \( A_{Lj} \) were used at the first step and continued with 0.2 \( A_{Lj} \) at the second step and an incremented moment at the x axis \( (M_{ax}) \). Instead of the specified loads, an initial prestress of 1.395 kN was also applied to each strand. In fact, the application of the two axial load levels presented in this study was due to the curvature ductility capacity and was strongly influenced by the presence of axial loads.

ADVANCED FINITE ELEMENT ANALYSIS

A single prestressed concrete pile connected to a cast in place (CIP) reinforced concrete pile cap presented in Figure 2 was modeled utilizing an advanced finite element method. Two significant aspects of the finite element model proposed herein were its capability of modeling of two and three-dimensional effects through the concrete constitutive law while simultaneously taking into account the gradual deterioration of the bond-slip between the reinforcing steel and concrete using the steel and bond-slip constitutive law. Due to a nonlinear structural analysis was performed using a small step-size loading scheme and iterative convergence control; consequently longer computing procedure was required to produce reliable results. Finally, to assess the seismic performance of the pile-to-pile cap connections, the proposed models (Teguh 2006) were tested against the effect of soil-pile interaction (SPI). The effect of SPI integrated in the complex finite element analysis is, however, beyond the scope of this study. It will be published in an elsewhere paper.

Model Discretization

A four-node quadrilateral isoparametric plane-stress element, namely, the QU4 Q8MEM element, was used to discretize the pile and pile cap concrete in the two-dimensional (2-D) finite element analysis of pile-to-pile cap connections. It is based on linear interpolation and Gauss integration. The polynomial for displacements \( u \) and \( v \) was expressed in terms of third and second parametric coordinates (\( \zeta \) and \( \eta \)) yielding a strain \( \varepsilon_{xy} \), which was constant in the \( y \) direction and varied linearly in the \( x \) direction. Applying a stabilization procedure of integration avoided a zero-energy mode; and assuming constant shear, the Q8MEM element provided a constant shear strain \( \gamma_{xy} \) over the element area. The procedure relied on the addition of a small least square contribution to the element energy norm as initially proposed by Roddeman (1994). A two-node line truss element (BE2 L4TRU) was selected to model bonded reinforcing steels for longitudinal reinforcement. A type of embedded reinforcement element was used to model spiral confinement and stirrups. These elements do not have degrees of freedom of their own and the strains are computed from the displacement field of the mother element. This implies perfect bond between the reinforcement and the surrounding material. In addition, a two-node interface element of IL22 L8IF was chosen to model over the interface elements.

The three-dimensional (3-D) finite element models were mainly used to analyze the nonlinear pile-to-pile cap connections assuming all reinforcing steels were perfectly bonded to the embedded reinforcements. A regular brick element, namely, HE8 HX24L, was utilized to model overall the pile and pile cap concrete. The meshing was employed for both 2-D and 3-D pile-to-pile cap connections. As shown in Table 1, the only PPC Model-1 has been completely detailed with the nonlinear analyses having three different pile embedment lengths. Whilst the PPC Model-2 was simply used for the sake of comparison particularly for the pile embedment length of 1.0 pile width and the reduction of compressive strength of concrete in the pile cap.

Material Models

The geometry, reinforcement detail and material properties of the pile and pile cap were generally unchanged in each model as illustrated in Figure 2. The model was varied with different cross-sections, starting with a square pile model depicting current practice and continuing with circular and octagonal pile models. The constitutive relationships used for the finite element material models and the model variables of pile-to-pile cap connections are listed in Table 1.

In this case, the concrete crack model is combined with a plasticity model, which includes the crushing of the concrete. The concrete utilizes Drucker-Prager yield criterion, which is applicable for quasi-brittle structures. The Hordijk model was used for the concrete tensile behavior (de Witte and Kikstra 2002). It consists of an elastic response to the tensile capacity followed by a nonlinear unloading branch. The concrete compressive model is obtained from cylinder tests. Cracking is modeled using both multiple fixed cracks and rotating crack formulations. The reinforcement uses Von Mises yield-hardening criteria with constitutive models matching the behavior determined from testing. The results presented in this paper are limited to the fixed crack model.

In order to define the strain softening branch of the tensile stress-strain relationship of concrete
through fracture mechanics concepts, three important parameters need to be defined: the tensile strength of concrete at which a fracture zone initiates; the area under the stress strain curve; and the shape of the descending branch (Reinhardt 1986). Among these parameters, the first two-parameter can be considered as material constants, while the shape of the descending branch varies with the models, as proposed by Bazant and Oh (1983).

### Table 1. Model variables of pile-to-pile cap connections

<table>
<thead>
<tr>
<th>Pile unit number</th>
<th>Model-1</th>
<th>Model-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial prestressing steel (constant)</td>
<td>1.394x10^7 N/m^2</td>
<td>1.394x10^7 N/m^2</td>
</tr>
<tr>
<td>Applied axial load (increment); assumed as pressure load over the pile surface</td>
<td>4.260x10^7 N/m^2</td>
<td>4.260x10^7 N/m^2</td>
</tr>
<tr>
<td>Maximum lateral loads (monotonic &amp; cyclic)</td>
<td>1.112x10^7 N</td>
<td>9.600x10^6 N</td>
</tr>
<tr>
<td>Pile reinforcement:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Low relaxation strand tendon</td>
<td>12.5 mm</td>
<td>12.5 mm</td>
</tr>
<tr>
<td>2. Wire plain spiral</td>
<td>7 mm (R12)</td>
<td>7 mm (R12)</td>
</tr>
<tr>
<td>Pile cap reinforcement:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1. Main</td>
<td>22 mm (D22)</td>
<td>22 mm (D22)</td>
</tr>
<tr>
<td>2. Transverse</td>
<td>D10 - 152 mm</td>
<td>D10 - 152 mm</td>
</tr>
<tr>
<td>Pile dimension (B)</td>
<td>0.45 x 0.45 m</td>
<td>0.45 x 0.45 m</td>
</tr>
<tr>
<td>Pile cap dimension</td>
<td>2.14 x 0.92 x 2.14 m</td>
<td>2.14 x 0.92 x 2.14 m</td>
</tr>
<tr>
<td>Pile embedment length</td>
<td>0.5 - 1.5 B</td>
<td>1.0 B</td>
</tr>
</tbody>
</table>

The uniaxial stress-strain relation for concrete under tension is assumed to be linear elastic up to its tensile strength, $f_t$. The descending branch after cracking is represented by a tri-linear tension-softening curve with fracture energy. The fracture energy of concrete, $G_f$ (N/mm²), is determined using the formula proposed by Oh-oka et al. (2000):

$$ G_f = \frac{0.23 \cdot f_t + 136}{1000} $$  \hspace{0.5cm} (1)

$\varepsilon' = W / L$, the compressive strength of concrete (MPa). To minimize localization of the fracture, the crack strain, $\varepsilon''$, is defined by dividing the crack width, $W$, by the characteristic element length, $L_\varepsilon$, for regularization: $\varepsilon'' = W / L_\varepsilon$. The characteristic length for a two-dimensional problem is defined as the diameter of a circle with an equivalent area to the element area, $A$, and $L_\varepsilon$ for the three-dimensional problem is defined as the diameter of a sphere with an equivalent volume to the element volume, $V$. Note that the origin-oriented secant stiffness is assumed for unloading and reloading.

The uniaxial stress-strain relation for concrete under compression up to the compressive strength, $f'_c$, is represented by a bilinear model with an intersection point at $f'/3$. The descending branch after the peak represents a linear compressive strain-softening model with compressive fracture energy, $G_{fc}$ (N/mm). The compressive fracture energy is determined according to the formula by Hordijk (1991) and Nakamura and Higa (1999):

$$ G_{fc} = 8.8 \sqrt{F_c} $$  \hspace{0.5cm} (2)

The plastic strain in the solid concrete, $\varepsilon_p$, is defined by dividing the plastic deformation, $\varepsilon_p$, by the characteristic element length, $L_\varepsilon$, for regularization. The definition of the characteristic element length for compression similar to that for tension, a compressive-strength reduction factor, $\lambda$, is introduced to take compressive softening of the cracked concrete in consideration, and referring to a current research undertaken by Tajima et al. (2004) $\lambda$ is assumed to be 0.85 in this study. The shear retention factor for the cracked concrete, $\beta$, is determined as a ratio of the shear stiffness of the cracked concrete, $G''$, to the elastic shear modulus of concrete, $G''$, according to the formula by Walraven and Kauser (1987):

$$ \beta = \frac{G''}{G''} = \frac{1}{1 + 4447 \cdot \varepsilon''} $$  \hspace{0.5cm} (3)

The reinforcing bar is treated as an elasto-plastic material and its constitutive law is derived on the basis of the Von-Mises yield criterion. The gradient after yielding is 1/100 of the initial stiffness, $E$. The recommended material model for cracked concrete, used in the finite element code in this study, is based on the nonlinear fracture mechanics approach. The material model presented here allows for high-strength concrete strain softening after cracking. The tension
stiffening has been used to simulate load transfer across cracks through the reinforcing bar, and a specific value for tension stiffening has been recommended for high-strength concrete. A constant shear retention factor has been introduced to consider the secondary mechanisms of shear resistance. An interface element model was performed between the pile head and pile cap to bond the two different concrete materials that were used in the pile and pile cap. The material arrangement tabulated in Table 2, and Table 3 was used to provide a more realistic behavior for the pile-to-pile cap connection, particularly when experiencing lateral seismic action. Modeling of the interaction between non-prestressing and prestressing steel and the surrounding concrete incorporating a bond-slip combined with interface element. The material constitutive models used in the concrete, bond-slip and interface element, and reinforcing steel are listed in Table 4.

Table 2. Mechanical properties of concrete

<table>
<thead>
<tr>
<th>Pile unit number</th>
<th>Compressive strength (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pile cap</td>
<td>Pile cap</td>
<td>Pile cap</td>
</tr>
<tr>
<td>Model-1</td>
<td>46.2</td>
<td>34.5</td>
<td>4.62</td>
</tr>
<tr>
<td>Model-2</td>
<td>46.2</td>
<td>20.7</td>
<td>4.62</td>
</tr>
</tbody>
</table>

Table 3. Mechanical properties of reinforcing bars

<table>
<thead>
<tr>
<th>Reinforcement</th>
<th>Yield stress (MPa)</th>
<th>Tensile strength (MPa)</th>
<th>Modulus of elasticity (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tendon</td>
<td>1791</td>
<td>1882</td>
<td>1.94 x 10^5</td>
</tr>
<tr>
<td>Wire</td>
<td>448</td>
<td>630</td>
<td>1.86 x 10^5</td>
</tr>
<tr>
<td>D22</td>
<td>275</td>
<td>464</td>
<td>2.00 x 10^5</td>
</tr>
<tr>
<td>D12</td>
<td>275</td>
<td>464</td>
<td>2.00 x 10^5</td>
</tr>
</tbody>
</table>

Table 4. Typical material characteristics for concrete, reinforcing steel, and bond-slip

<table>
<thead>
<tr>
<th>Material description</th>
<th>Pile cap</th>
<th>Structural Element</th>
<th>Pile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Poisson's ratio, σ</td>
<td>0.17</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td>Density, γ</td>
<td>2400 kg/m^3</td>
<td>2400 kg/m^3</td>
<td></td>
</tr>
<tr>
<td>Plasticity model</td>
<td>Drucker-Prager criterion</td>
<td>Drucker-Prager criterion</td>
<td></td>
</tr>
<tr>
<td>Hardening</td>
<td>Strain</td>
<td>Strain</td>
<td></td>
</tr>
<tr>
<td>Smearing cracking model</td>
<td>Constant stress cut-off</td>
<td>Constant stress cut-off</td>
<td></td>
</tr>
<tr>
<td>Tension softening</td>
<td>Multi-linear</td>
<td>Multi-linear</td>
<td></td>
</tr>
<tr>
<td>Shear retention, β</td>
<td>Constant, β = 0.999</td>
<td>Constant, β = 0.999</td>
<td></td>
</tr>
</tbody>
</table>

Constitutive model for concrete

Constitutive model for bond-slip

Constitutive model for reinforcing steel

Bond-Slip Model and Interface Element

Stresses in the interface between the reinforcing bar and concrete are generally transferred by a cohesive action, frictional action and a locking action between the reinforcing steel and concrete. Two basically different elements have been proposed, to date, to include the bond-slip effect in the finite
element analysis of reinforced concrete structures. The first is the bond link element by Ngo and Scordelis (1967). It consists of two orthogonal springs, which connect and transmit shear and normal forces between a reinforcing steel node and an adjacent concrete node. As the link element has no physical dimensions, the two connected nodes originally occupy the same location in the finite element mesh of the undeformed structure. The second bond-slip element, called the bond-zone element, is significantly different from the bond link element; the most important difference is its finite dimension. In the bond-zone element by de Groot et al. (1981), the contact surface between reinforcing steel and concrete, and the concrete in the immediate vicinity of the reinforcing bar are modeled by a material law, which represents the special properties of the bond-zone. In this model, the bond stress is the sum of the slip resistance and the stress due to mechanical interlocking. The following discussion on the bond-slip is based on an expansion of the first model. However, the second bond-slip element was not employed in the development of the source code of the finite element program (DIANA version 9.1, 2005).

In a simple model, bond behavior was induced by these interactions as a bond stress \( \tau_b \) - slip \( \delta \) relation. A trilinear bond model constructed by modifying the so-called ‘Kaku model’ has been frequently used in finite element analyses of beam-column joints by Tajima et al. (2004). The modified Kaku model and ‘Comité Euro-International du Beton’ (CEB) model (CEB-FIP 1993) are used as the \( \tau_b - \delta \) relation, and they are represented by the interface element. The bond stress-slip of the CEB model, associated with the bond-slip of the multi-linear model, was selected and consistently used in the overall finite element models of the pile-to-pile cap connections. In order to include the locking action of protrusions in the two and three-dimensional analyses used in this study, the bond behavior of the steel part between protrusions was represented by the \( \tau_b - \delta \) model for the round smooth bar provided by the CEB model code. The locking action was then modeled by the linkage element consisting of a set of inclined orthogonal springs. The inclined springs have compressive resistance but no tensile resistance. The axial compressive stiffness of the spring, \( K_u \), is determined according to the following equation by Fujii (1992):

\[
K_u = 21 \cdot \pi \cdot L \cdot E_c \cdot \cos \theta
\]

where \( d \) is the diameter of the reinforcing bar, \( L \) is the length covered by a set of springs, \( \theta \) is the length of the local deformation zone and assumed to be equal to \( d/20 \). Next, \( \theta \) is the angle between the bar axis and the spring direction and is assumed to be 450, and \( E_c \) is the Young’s modulus of concrete.

RESULTS AND DISCUSSION

Effects of various pile sections and headed embedment length ratios

The main factor affecting the ductility of prestressed concrete piles is highly dependent on the longitudinal reinforcing steel and confinement ratios, rate of loading, compressive strength of concrete, yield stress, and bar diameter to preserve the integrity of the core concrete against excessive compressive force resulted from significant deformation. Having sufficient transverse reinforcement prevents premature buckling of the tendon and provides adequate shear resistance.

Figure 3 shows a solid square prestressed concrete pile results in a substantial moment compared with the solid circular and octagonal pile sections; however, it results in a smaller curvature than other sections, where a longer curvature produces a higher curvature ductility of the pile. In other words, circular and octagonal pile sections have provided greater curvature ductility capacities. When the axial load was increased to 20% of the section capacity, the moment capacity at each section tended to be similar to previous curves, however the curvature consistently showed decreasing values. In contrast, the curvature ductility at each section has proportionally increased. Although each pile section was specified with the same area of the confined concrete, the greater area of the unconfined concrete produced by the square section contributed to a significant change in the moment capacity and curvature ductility. Based on the calculation of the moment-curvature relation, the curvature ductility of each pile section used in the pile-to-pile cap connections with different spiral confinement ratios and axial loads is further determined to recommend the best pile section in producing significant strength and curvature ductility.
The moment-curvature analysis was constructed to establish the significant moment-curvature relations for three common prestressed concrete piles having constant reinforcement details and confined concrete cores. Based on the moment-curvature analysis of various pile sections and static axial loads, as seen in Figure 4, it is obvious that the square pile section provides the highest moment capacity compared to other sections. Three models of pile-to-pile cap connections considering various pile sections were then investigated to establish the behavior of pile-to-pile cap connections subjected to simulated seismic actions. The main parametric studies included: a variety of pile sections and pile embedment lengths, a variety of material constitutive models, various bond-slip and interface elements and different pile embedment models. A comparison of structural responses for three different pile sections with constant axial load was undertaken by Teguh (2010), and resulted in significant displacement ductility and moment capacity on the square pile section. It can be concluded that the PPC model with a square pile section provides the highest structural response compared to the other two models. The circular and octagonal pile sections, however, seem to show similar trends to the square section, resulting in similar displacement ductility but less structural responses.

For this reason, three different headed-pile embedment lengths for the specified PPC Model-1 were studied in an attempt to investigate the proposed new development of pile-to-pile cap connections under simulated seismic loads. The pile embedment begins with a 0.5 times pile width or pile diameter and increases with multiple embedment lengths up to 1.5 times pile width. The 50 mm of pile seating as commonly used in practice was considered in the nonlinear inelastic pushover analysis and was excluded in the pile embedment length. Most importantly, this parametric study on the pile embedment ratio was selected in the range of 0.5-1.5 times pile width and thus, the behavior of headed pile-to-pile cap connections is extensively investigated. The worldwide standards (Australian Standard AS 2159 1995; Fuentes 2000; New Zealand Standard 1982) recommend that the pile embedment length should be approximately 1.0-2.0 times pile width.

![Figure 4. Fixed moment-drift ratio relationships for three different pile sections having constant static axial loads and 1.5 pile depth of embedment length.](image)

Nonsinear inelastic structural response

In the study of the new pile-to-pile cap connections herein, the bond-slip was applied to all main reinforcement of the pile cap and pile, while a perfect bond was applied to the transverse reinforcement of the pile cap and the spiral confinement of the pile. The structural response was performed for nonlinear inelastic pushover and reversed cyclic analyses. The resulting load-displacement response for the PPC Model-1 with headed embedment length of 1.5 pile widths is presented in Figure 5 showing a hysteresis loop that matches the envelope of the nonlinear pushover analysis. However, severity crack failures were occurred after achieving five cycles because of the reversed cyclic load behavior.

All PPC models were tested with a typical cyclic loading history, where the lateral deflection force was applied based on a sequence of numbers of load cycles. Although in the analytical development, the PPC models were similarly reinforced, the structural behavior, when subjected to the cyclic loading, was dependent on the overall PPC performance. For example, this is seen in the contribution of unconfined concrete due to unequal pile section areas producing a different moment capacity for the connection. For the PPC Model-1, the ultimate load was reached at about 5 cycles or 3344 load steps, resulting in an ultimate lateral displacement and load of 173 mm and 115 kN, respectively, as shown in Figure 5. For a comparison of these analyses, the first yield displacement as well as the maximum lateral load was reached and matched at the same value.
however the maximum lateral displacements were achieved at different values resulting in higher displacement ductility for the pushover analysis.

Figure 6 shows a comparison of the load-displacement responses of three different headed-pile embedment lengths. The strength as well as ductility increases significantly as the ratio of embedment length increases. As a result, the ductility displacements are 11.5, 15.72, and 25, corresponding with a maximum lateral load resistance of 87.6, 103, and 116 kN for the embedment ratios of 0.5, 1.0, and 1.5, respectively. Depending on how deep the pile embedment length is, and for which pile embedment model, the strength and ductility of the pile-to-pile cap connection, provides the desired structural performance in resisting simulated seismic actions. For practical reasons, it is not believed that embedment lengths shorter than 1.0 times the pile width should be specified. Similarly, embedment lengths longer than 1.5 to 2.0 pile widths are also impractical without significantly affecting the design and construction of the pile caps. It is recommended that the pile embedment length used in practice should be 1.5 times the pile width or pile diameter but should be not less than 1.0 times the pile width.

![Figure 5. A comparison of structural responses for the PPC Model-1 with a headed embedment model confined with spiral reinforcement.](image)

**Deformation and cracking history**

All the pile models failed due to the formation of a plastic hinge in the pile rather than in the pile cap. The pile length (L) for all models was specified as 3700 mm, measured from the interface of the pile cap, and the maximum lateral deflection at the tip of the pile was $\Delta$. The maximum drift ratio of the PPC Model-1 was defined as on the ratio of the maximum lateral deflection and pile length. When all models had long piles, therefore, the connection moment capacity was dominated by the flexural bending effect. The $P-\Delta$ moment induced by axial load, however, was excluded from the connection moment capacity. As a result, cracks open and close, leading to a greater rate of system stiffness degradation than that observed under monotonic loading. Load reversal requires a more sophisticated constitutive modeling and this issue was investigated to better represent pile-to-pile cap connections. The maximum stresses predicted by the finite element model reflected the observed crack propagation at the pile-to-pile cap interface. When constant axial loads were applied and the explicit step size of the monotonic lateral loading was utilized, the pile-to-pile cap connection developed diagonal cracks at the base of the pile cap close to the supports and hair cracks at the pile interface during 0.5% of drift. In addition, the applied constant axial loads primarily affected the diagonal cracks occurring in the surrounding supports. Figure 7 shows sequential crack deterioration propagates along the pile and inside the connection, showing the crack strain values. It is apparent that a potential plastic hinge could develop at about 1.0-2.0 B, equivalent to 900 mm measured from the interface between pile head and pile cap, where B is pile width or diameter. Diagonal and vertical cracks in the connection were noticed starting at 1.0% drift, and overall cracks lay on the pile along 1700 mm measured from the pile cap. The crack damage particularly propagated along the 1700 mm of tensile and compressive fibers of the pile.

![Figure 6. A comparison of load-displacement responses with varying pile embedment lengths for the PPC Model-1.](image)

Furthermore, an interfering action resulted when a moment was applied at the interface of the pile and the pile cap, thereby increasing the contact pressure at the junction of the pile face and the pile cap. It is obvious that when concrete systems are subjected to cyclic loading the entire system undergoes tension-compression reversals. As a result, the pile head then experienced a plastic hinge when concrete and reinforcing steel surrounding the interface between pile head and pile cap was totally damaged and the inside of the connection was already cracked. It has become apparent that under the increasing lateral cyclic loadings the unconfined and confined concrete at the interface between pile head and pile cap have
already been damaged. The result shows the predicted stress-strain relationship of concrete at the interface zone, where concrete in this zone has experienced compression and tension during the lateral cyclic loading applied. It was shown by the nonlinear finite element analysis that the confined concrete has reached the maximum compressive stress and compressive strain with magnitudes of 42.0 MPa and 0.00382, respectively. The tensile stress and tensile strain were detected at 5.50 MPa and 0.0032, respectively. At this stage of loading and when the stresses of the tendon were checked at the interface zone, the prestressing steels have already yielded.

![3D FEM discretization](image1)

![Crack propagation](image2)

**Figure 7.** FEM discretization of PPC and severity crack propagation at the ultimate loads.

**Reduced concrete strength on pile cap.**

It has been observed that when this type of connection model was subjected to increasing lateral loads, the caps failed through concrete cracking accompanied by large strains of the prestressing steel in the potential plastic hinge zone. Under the condition of maximum lateral load, the unconfined concrete of the pile in the compressive fibers spalled out and the confined concrete has crushed, producing yielding in the prestressing steels surrounding that area. The pile caps, as well as the pile ends, were generally undamaged after achieving the ultimate loads that were either induced by monotonic or cyclic loading. When the concrete strength of pile cap for the PPC Model-1 was reduced to 40% as applied to the PPC Model-2, strength and displacement ductility of the connections reduced. Keeping the headed-pile embedment length (1.0 pile width) constant for two models, the displacement ductility and moment capacity is not significantly reduced since the reinforcement detail in the pile cap remains the same as the PPC Model-1. The longer embedment length and higher concrete strength of the pile cap yields higher strength and ductility for the connection. For this reason, when the embedment length and concrete strength are increased by 50% and 67% respectively, the maximum lateral displacement is then increased by approximately 50%.

**CONCLUDING REMARKS**

The finite advance element analyses of nonlinear pile-to-pile cap connections under monotonic and cyclic loads incorporating complex constitutive material models and parametric study of pile embedment lengths demonstrate good analytical results and reliable seismic performance compared to experimental results.

The nonlinear inelastic pushover and cyclic reversed analyses accurately predict the pile damage showing the crack behavior at the pile head and pile-to-pile cap joint for both PPC models. Given this crack damage history at the final stage of loading, it can be concluded that the effect of shear forces was not significant. The results show that the use of headed embedment is opposed to plain embedment provides effective confinement of the joint region, allowing for less congestion with comparable levels of performance.

Based on the analysis, it is of interest that the headed embedment model gives better energy dissipation in the connection and this model is more convenient in practice, particularly when utilized for a pile foundation-supported long bridge. It is recommended that using a headed-pile embedment length of 1.5 times pile width or pile diameter produces a strong connection and reduces stress concentration and crack damage at the interface and inner pile-to-pile cap connections.

**REFERENCES**

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