Simulated Seismic Load Tests on Prestressed Concrete Piles and Pile-Pile Cap Connections

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The results of tests conducted in New Zealand on prestressed concrete piles and pile-pile cap connections subjected to simulated seismic loading are summarized. The piles were of prestressed pretensioned concrete with a 400 mm (15.7 in.) octagonal cross section and contained spiral reinforcement. The pile caps were of reinforced concrete. The range of pile-pile cap connection details tested included piles with ends embedded in the cap either with or without the ends of the longitudinal steel exposed, and the use of longitudinal steel dowel bars. The test results showed that well detailed prestressed concrete piles and pile-pile cap connections are capable of undergoing large post-elastic deformations without significant loss in strength when subjected to severe seismic loading.

Piles in soft soils supporting building or bridge structures can be subjected to large horizontal displacements when subjected to severe earthquakes. The resulting deformations can result in significant curvatures in some region of the piles. One region of high curvature is at the pile-pile cap interface, due to the end fixity of the pile in the pile cap. A second region of high curvature is within the soil. These critical regions are illustrated in Fig. 1. In addition, for piles embedded in layers of soil, with the soil modulus varying down the depth, the pile curvature may be particularly high at the interface between hard and soft layers of soil.

Severe damage to piles has been observed during some earthquakes. For example, earthquake damage to piles due to large imposed curvatures has been described by Gerwick,1 Sheppard2 and Banerjee et al.3

The regions of the pile where high curvatures occur need to be designed to possess adequate ductility, where ductility may be defined as the ability to undergo large amplitude cyclic deformations in the post-elastic range without significant reduction in strength. Loss of flexural strength of the pile is invariably accompanied by loss of vertical load carrying capacity.

It should be noted that it is difficult to inspect and repair damage which occurs to foundations underground or under water as a result of severe earthquakes. Therefore, it is desirable to design the piles of building and bridge structures so as to remain undamaged during severe earthquakes.

Ideally for such structures the design concept should aim at dissipating seismic energy by ductile yielding at plastic hinge regions in the structure above the founda-
tions or in mechanical energy dissipating devices placed between the foundations and the structure. That is, the foundations should be provided with sufficient strength to ensure as far as possible that they remain in the elastic range while energy dissipation occurs at the chosen yielding locations elsewhere during a severe earthquake.

However, uncertainties do exist with regard to soil-structure interaction and the resulting actual pile behavior during a severe earthquake. It would appear to be essential to detail piles of building and bridge structures so as to be capable of a reasonable degree of ductile behavior. Also, in wharf structures it is commonly desirable to design the piles so as to be capable of dissipating significant energy by ductile plastic hinge behavior.

Previous papers 4,5 have described tests conducted in New Zealand to obtain information on the strength and ductility of prestressed concrete piles subjected to compressive loading and cyclic lateral loading which simulated severe earthquake induced deformations. The piles contained spiral reinforcement.

This paper first presents a summary of the results of an additional six full-scale prestressed concrete piles tested under simulated seismic loading in which the main variables were the quantity and grade of spiral reinforcement.

Then a further stage of the investigation is described in which methods for detailing the connections between prestressed concrete piles and reinforced concrete pile caps are considered. The results of tests conducted on six full-scale connections subjected to simulated seismic loading are outlined. The test variables included different methods for achieving anchorage of the pretensioned pile in the reinforced concrete pile cap, and the effect of having the critical section for pile flexure within the end transfer length of the prestressing strand. Moment-curvature analyses were also conducted for the plastic hinge regions of the piles to check the theoretical approach for assessing the ductility of spirally confined piles.

The results summarized in this paper may be seen reported in more detail elsewhere. 6

**SPIRAL REINFORCEMENT IN PRESTRESSED CONCRETE PILES**

**The Role of Spiral Reinforcement**

Prestressed concrete piles commonly contain longitudinal steel in the form of pretensioned strand in circular array around the cross section. Nonprestressed longitudinal reinforcement may also be present. Transverse spiral reinforcement is placed surrounding the longitudinal steel. The role of the spiral reinforcement is to confine the compressed concrete, to prevent premature buckling of the longitudinal steel, and to prevent shear failure of the member. Spiral reinforcement also improves the anchorage of longitudinal steel. The ends of the spiral reinforcement need to be anchored adequately if the spiral is to function effectively.

Spiral reinforcement acts as confining reinforcement in piles when at high concrete compressive stresses the transverse strains in the concrete become large and cause the
Concrete to bulge out against the spiral reinforcement which then resists the bulging. This confining pressure enhances the strength and ductility of the compressed concrete. Spirals, because of their shape, are in hoop tension and apply a uniform radial confining pressure to the concrete. Small spacing of spirals leads to better confinement of the concrete because the arching of compressed concrete between the spirals penetrates less deeply into the concrete section and hence the concrete is more effectively confined.

Outside the spiral the concrete is not confined and this cover (or shell) concrete commences to spall after the unconfined concrete strength is reached. However, the confined concrete core can continue to carry stress at much higher strains. A well designed pile can continue to carry significant moment and vertical load after the loss of the concrete cover at large deformations in the post-elastic range. This is because the confinement of the concrete core provided by the spiral enhances the strength and ductility of the core concrete which compensates for the loss of the load carrying capacity of the cover concrete which spalls off at large deformations.

The spacing of the spiral reinforcement should also be close enough to prevent premature buckling of longitudinal compressed steel, and the spiral reinforcement should be adequate for shear resistance.

**New Zealand Code Special Requirements for Piles Designed for Seismic Loading**

According to the current New Zealand concrete design code NZS 3101, the potential plastic hinge region of a pile, which is the critical region where strains in the post-elastic range and possible damage would concentrate, is the end region of the pile extending from the underside of the pile cap over a length equal to the longest pile cross-sectional dimension or 450 mm (17.7 in.), whichever is greater. According to NZS 3101, in potential plastic hinge regions when spiral reinforcement is used:

(a) The volumetric ratio of spiral steel \( \rho_s \) required for concrete confinement shall not be less than:

\[
\rho_s = 0.5 \left( \frac{A_g}{A_c} - 1 \right) \frac{f_{cs}}{f_{sh}} \left( 0.5 + 1.25 \frac{P_e}{\phi f_{cs} A_g} \right)
\]

or

\[
\rho_s = 0.12 \frac{f_{cs}}{f_{sh}} \left( 0.5 + 1.25 \frac{P_e}{\phi f_{cs} A_g} \right)
\]

whichever is greater, where:

- \( A_g \) = gross area of pile cross section
- \( A_c \) = area of concrete core of pile cross section measured to outside of spiral
- \( f_{cs} \) = concrete compressive cylinder strength
- \( f_{sh} \) = yield strength of spiral steel
- \( P_e \) = external compressive load on pile
- \( \phi \) = strength reduction factor equals 0.9 but may be taken as 1.0 if a design procedure is used to reduce the likelihood of plastic hinging in the pile

(b) In Eqs. (1) and (2), the external vertical compressive load \( P_e \) should not exceed 0.7 \( f_{sh} A_g \) unless it can be shown that \( P_e \) is less than 0.7 \( P_a \), where \( P_a \) is the confinement load strength of the pile, and \( f_{sh} \) shall not exceed 500 MPa (73 ksi).

(c) Center to center spacing of the spirals along the pile, required to adequately confine the concrete and to prevent premature buckling of the longitudinal steel, shall not exceed the smaller of one-fifth of the pile diameter, or six times the diameter of the longitudinal bar to be restrained, or 200 mm (7.9 in.).

Adjacent to the potential plastic hinge region and for a length equal to three pile diameters or the length of the potential plastic hinge region, whichever is greater, the spiral reinforcement shall be as follows:

(a) Center to center spacing of the spiral reinforcement along the pile shall not exceed the smaller of two-fifths of the pile diameter in the case of a circular cross section, or 12 times the diameter of the longitudinal bar to be restrained, or 400 mm (15.7 in.).

(b) The quantity of transverse reinforcement shall not be less than one-half of that required in the potential plastic hinge region.

Over the remainder of the pile, the center to center spacing of the spiral reinforcement shall not exceed the effective depth \( d \) of the pile unless the soil-structure interaction characteristics of the soil require a smaller spacing.

Spiral reinforcement placed as above may also be assumed to contribute to the shear strength of the member. It should be checked to determine whether it is adequate for the required shear strength and additional spiral reinforcement added if necessary.

Although not specifically required by the code, it could be considered prudent when using Eqs. (1) and (2) to replace \( P_e \) by \( P_a + f_{cs} A_g \), where \( f_{cs} \) is the axial compressive stress in concrete due to prestress alone, since the prestress increases the net compressive stress to be carried by the concrete.

**TESTS ON PRESTRESSED CONCRETE PILES**

Previous papers have described the results of tests conducted in New Zealand on 11 prestressed concrete piles with a 400 mm (15.7 in.) octagonal cross section subjected to simulated seismic loading. An additional six prestressed concrete piles have recently been tested to extend the range of variables investigated.

Fig. 2 shows the loading arrangements used to apply the simulated seismic loading, and the dimensions of the piles, for all 17 piles tested. The piles had a stub at mid-height to simulate a pile cap. During the tests the vertical compressive load was held constant at a predetermined level and to the stub a reversible static horizontal load was applied which was either load or displacement controlled.
The aim of the tests was to determine the available strength and ductility of the piles. Hence, the displacement controlled loading consisted of cycles to displacement ductility factors $\mu = \Delta / \Delta_y$ of increasing magnitude, until failure or until $\mu = \pm 8$ or higher was reached, where $\Delta$ is the maximum horizontal displacement and $\Delta_y$ is the horizontal displacement at yield. The maximum bending moment occurred in the pile unit at the section adjacent to the stub.

The displacement at yield $\Delta_y$ was defined as the displacement measured in the first loading cycle when $0.75$ of the ideal flexural strength of the pile was reached multiplied by $1/0.75 = 1.33$. The ideal flexural strength is the theoretical moment capacity calculated by the conven-

**Table 1. Main properties of the additional pile units.**

<table>
<thead>
<tr>
<th>Unit</th>
<th>$P_e$</th>
<th>$f'_c$</th>
<th>$f_p$</th>
<th>Prestressing tendons</th>
<th>Spiral reinforcement in plastic hinge regions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_e$</td>
<td>$f'_c$</td>
<td>$f_p$</td>
<td>$\rho_1$</td>
<td>$f_y$</td>
</tr>
<tr>
<td>P1</td>
<td>0.3</td>
<td>48.0</td>
<td>8.5</td>
<td>0.0070</td>
<td>1901</td>
</tr>
<tr>
<td>P2</td>
<td>0.3</td>
<td>45.1</td>
<td>8.2</td>
<td>0.0070</td>
<td>1901</td>
</tr>
<tr>
<td>P3</td>
<td>0.3</td>
<td>50.0</td>
<td>8.5</td>
<td>0.0070</td>
<td>1901</td>
</tr>
<tr>
<td>P4</td>
<td>0.3</td>
<td>49.0</td>
<td>8.3</td>
<td>0.0070</td>
<td>1901</td>
</tr>
<tr>
<td>P5</td>
<td>0.3</td>
<td>41.7</td>
<td>8.4</td>
<td>0.0070</td>
<td>1901</td>
</tr>
<tr>
<td>P6</td>
<td>0.5</td>
<td>43.3</td>
<td>8.3</td>
<td>0.0070</td>
<td>1901</td>
</tr>
</tbody>
</table>

Metric (SI) conversion factors: 1 MPa = 145 psi; 1 mm = 0.0394 in.

Notes:
1. Prestressing tendons in each pile: ten 12.5 mm (1/2 in.) diameter seven-wire strands.
2. $\rho_l$ code is calculated using Eq. (1) assuming $\rho = 1$ but with $P_e$ replaced by $P_e + f_y A_e$.
3. The range of $P_e$ actually applied in the tests varied by up to $\pm 12$ percent of the specified axial load at high lateral loads.
tional ACI and New Zealand code approach using the requirements of strain compatibility and equilibrium and assuming a concrete compressive stress block which has a mean stress of \(0.85 f_c'\), an extreme fiber concrete compressive strain of 0.003, the measured stress-strain curves for the steel, and a strength reduction factor \(\phi = 1.0\). This definition for \(\Delta\), has been commonly used in New Zealand. It gives \(\Delta\) as that for the equivalent elasto-plastic system which has the elastic stiffness measured as the secant value at 0.75 of the flexural strength.

Each pile was pretensioned by ten 12.5 mm (\(\frac{1}{2}\) in.) diameter seven-wire strands. The main variables investigated were the vertical compressive load \(P_{el}\), the presence or not of nonprestressed longitudinal reinforcement, the quantity of spiral reinforcement and the grade of spiral reinforcement. The spiral reinforcement was from either Grade 275 (\(f_{yh} = 40\) ksi) steel with a guaranteed yield strength of 485 MPa (\(f_{yu} = 70.3\) ksi), or hard drawn steel with a guaranteed yield strength of 630 MPa (\(f_{yu} = 91.2\) ksi).

It should be noted that the pile steel was continuous through the mid-height stub. That is, methods of connecting piles to pile caps were not investigated in these first pile tests.

**Details and Test Results of the Additional Six Piles**

The main properties of the additional six pile units, Units P1 to P6, are listed in Table 1 and may be seen described in more detail elsewhere. The measured stress-strain relations for the prestressing tendons and the spiral steel for Units P1 to P6 are shown in Figs. 3 to 5. The spiral reinforcement was from either Grade 275 (\(f_{yh} \geq 40\) ksi) steel or hard drawn steel with a fracture strain of at least 37 percent or from hard drawn steel with a fracture strain of about 11 percent. The fracture strains were measured during tension tests as the percentage permanent elongation at fracture over a length of five bar diameters. Hard drawn steel has the apparent advantage that smaller quantities are necessary because of the higher steel strength. Also, it is available in long lengths in coils and therefore the spiral can be formed from one length of steel thus avoiding welding for anchorage within the spiral.

Table 2 shows a comparison of the theoretical flexural strengths of the critical sections of the piles with the experimental maximum moments measured during the tests. The theoretical flexural strengths, \(M_A\) and \(M_e\), were calculated two ways using the requirements of strain compatibility and equilibrium.

\(M_A\) was calculated by the conventional flexural strength theory recommended by the ACI and New Zealand codes. That is, using a concrete compressive stress block which has a mean stress of \(0.85 f_c'\), assuming an extreme fiber concrete compressive strain of 0.003, using the measured stress-strain curves for the tendons, and applying a strength reduction factor \(\phi = 1.0\).

\(M_e\) was calculated using a concrete stress distribution for the compressed region of the core of the pile obtained...
Table 2. Theoretical flexural strengths and experimental maximum moments of piles at face of central stub, kN\cdot m.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Theoretical flexural strength</th>
<th>Experimental maximum moment</th>
<th>$M_{max, exp}$</th>
<th>$M_{max, exp}$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$M_A$</td>
<td>$M_C$</td>
<td>$M_{max, exp}$</td>
<td>$M_A$</td>
</tr>
<tr>
<td>P1</td>
<td>255</td>
<td>271</td>
<td>318</td>
<td>1.25</td>
</tr>
<tr>
<td>P2</td>
<td>241</td>
<td>300</td>
<td>330</td>
<td>1.37</td>
</tr>
<tr>
<td>P3</td>
<td>265</td>
<td>283</td>
<td>337</td>
<td>1.27</td>
</tr>
<tr>
<td>P4</td>
<td>260</td>
<td>336</td>
<td>331</td>
<td>1.27</td>
</tr>
<tr>
<td>P5</td>
<td>224</td>
<td>309</td>
<td>333</td>
<td>1.49</td>
</tr>
<tr>
<td>P6</td>
<td>171</td>
<td>357</td>
<td>400</td>
<td>2.34</td>
</tr>
</tbody>
</table>

Metric (SI) conversion factor: 1 kN\cdot m = 0.737 kip-ft.

from the stress-strain curve for spirally confined concrete proposed by Mander et al.\(^7\) and using the measured stress-strain curves for the tendons. The compressive strain at the extreme fiber of the concrete core $\varepsilon_{cm}$ was taken as the strain at a stress of $0.8 f_{cp}$ on the falling branch of the stress-strain curve, where $f_{cp}$ is the compressive strength of spirally confined concrete. Fig. 6 shows typical stress-strain curves for confined and unconfined concrete proposed by Mander et al.\(^7\) The stress and strains at the peaks of the curves for confined and unconfined concrete are related by:

$$\varepsilon_{cc} = \varepsilon_{cp} \left(1 + R \left(\frac{f_{cp}}{f_{cm}} - 1\right)\right)$$

For the analysis of the six pile units it was assumed that $f_{cp} = f_{cm}$ and $R = 3$. It was found for the range of spiral reinforcement used that $f_{cp}/f_{cm}$ varied between 1.38 to 1.99 and $\varepsilon_{cm}$ varied between 0.0087 to 0.022.

It is apparent from the comparison in Table 2 that the enhancement in the strength and ductility of the compressed concrete due to confinement can result in a considerable increase in the flexural strength above that calculated using the ACI and New Zealand code approach. The more refined analysis including the effect of confinement results in more accurate predictions of the maximum measured (experimental) moments.

Figs. 7 to 10 show the measured lateral load versus lateral displacement hysteresis loops for Units P1, P2, P3 and P5. The gradual reduction in the lateral load capacity of the units with increasing displacements is mainly due to the $P$-$\Delta$ effect of the axial loading. Superimposed on those figures are inclined lines marked A and C which correspond to the theoretical lateral loads when the calculated flexural strengths $M_A$ and $M_C$, respectively, were reached at the critical section of the pile at the face of the stub. The slope of these lines is due to the increasing effect of the $P$-$\Delta$ moment as the lateral deflection is increased.

Figs. 7 and 8 contrast the behavior of Units P1 and P2 which contained 59 and 89 percent, respectively, of the amount of Grade 275 ($f_{pB} \geq 40$ ksi) spiral steel required by Eq. (1) with ($P_r + f_{cp} A_y$) substituted for $P_r$. It is evident that Unit P1 reached displacement ductility factors $\mu$ of at least four while maintaining a flexural strength of greater than 0.8$M_C$. Unit P2, with the larger quantity of spiral steel, reached $\mu$ of at least seven while maintaining a flexural strength greater than 0.8$M_C$. For these two pile units the spiral steel did not fracture during the tests.

Figs. 9 and 10 contrast the behavior of Units P3 and P5 which contained 58 and 122 percent, respectively, of the amount of hard drawn spiral steel required by Eq. (1) with ($P_r + f_{cm} A_y$) substituted for $P_r$. When Unit P3 reached approximately $\mu = 4$, a brittle failure occurred as a result of the fracture of two spiral bars. However, Unit P5, with the greater quantity of spiral steel, reached $\mu > 8$ without a substantial reduction in strength and without fracture of the spiral steel.

**Discussion of the Test Results**

The conclusions reached as a result of the tests on all 17 piles units were:4,5,6

(a) The presence of nonprestressed longitudinal reinforcement in prestressed concrete piles is not essential to the satisfactory ductile performance of the piles. Nonprestressed longitudinal reinforcement need only be present if required for anchorage of the pile end in the pile cap.

(b) Prestressed concrete piles designed using the NZS 3101\(^6\) provisions for spiral confining reinforcement exhibited stable lateral load-displacement hysteresis loops at displacement ductility factors in excess of $\mu = \pm 8$. However, it is recommended that the amount of spiral confining steel be based on an effective axial load, comprising the sum of the internal prestressing force and the maximum externally applied load. That is, in Eqs. (1) and (2), $P_r$ should be replaced by $P_r + f_{cp} A_y$, where $f_{cp}$ is the axial compressive stress in concrete due to prestress alone. This is because the prestress increases the net compressive force to be carried by the concrete.

(c) When the spiral reinforcement was provided by hard drawn steel wire, the displacement ductility factor reached was sometimes limited by the eventual fracture of the confining steel, whereas piles confined by Grade 275 ($f_{pB} \geq 40$ ksi) steel spirals did not suffer confining steel fracture. This was to be expected since the fracture strain of the hard drawn steel was approximately 11 percent whereas the Grade 275 steel was much more ductile.
Fig. 7. Measured lateral load-displacement hysteresis loops for Unit P1.

Fig. 8. Measured lateral load-displacement hysteresis loops for Unit P2.

Fig. 9. Measured lateral load-displacement hysteresis loops for Unit P3.

Fig. 10. Measured lateral load-displacement hysteresis loops for Unit P5.

(d) Prestressed concrete piles with between 24 and 50 percent of the NZS 3101 amount of spiral confining steel, calculated with $P_c$ replaced by $P_c + f_{cp} A_g$, failed at maximum displacement ductility factors between 2.5 and 7.0.

(e) Buckling of the tendons occurred at large displacements of the piles in the inelastic range. However, the spiral spacings of 35 to 50 mm (1.4 to 2.0 in.) used in these tests were found to be adequate to prevent premature buckling of tendons. It is considered that the spacing of spirals in potential plastic hinge regions should not exceed four tendon diameters.

(f) It is considered that prestressed concrete piles confined with 50 percent of the NZS 3101 amount of spiral confining steel for an axial load level of $(0.3 f'_c + f_{cp}) A_g$, and with a spiral spacing not greater than the least of four tendon diameters or one-fifth of the pile diameter, will achieve displacement ductility factors of at least $\mu = \pm 4$ during cyclic loading.

(g) The theoretical flexural strengths calculated using the concrete compressive stress block recommended by the ACI$^{10}$ and New Zealand$^{9}$ codes which has a mean stress of $0.85 f'_c$, assuming an extreme fiber concrete compression strain of 0.003, and using the measured material strengths, underestimated the maximum measured moment capacity of the pile units. The ratio of the measured maximum moment to this theoretical moment for the 17 piles tested ranged between 1.12 and 2.39, indicating the considerable effect of the enhancement of concrete compressive strength and ductility due to confinement. A refined flexural strength analysis utilizing a stress-strain curve for confined concrete resulted in much closer predictions of the flexural strength.
TESTS ON CONNECTIONS BETWEEN PRESTRESSED CONCRETE PILES AND REINFORCED CONCRETE PILE CAPS

The previous tests at the University of Canterbury involved prestressed pile units with pretensioned tendons continuous through the central stub (see Fig. 2). Although those tests gave information concerning pile behavior, they gave no information concerning the strength and ductility of connections between precast piles and cast-in-place concrete pile caps as used in practice.

Connection details between precast prestressed concrete piles and reinforced concrete pile caps, to achieve a moment resisting connection, have been recommended by many authorities in the past. For example, the New Zealand Ministry of Works and Development recommend either embedding the end of the pile in the pile cap or breaking back the pile concrete at the end of the pile and embedding the exposed prestressing strands and other reinforcement in the pile cap. Sheppard suggests either breaking back the pile concrete and embedding the exposed prestressing strands in the pile cap or achieving the connection by means of dowel bars grouted into sleeves in the pile and embedded in the pile cap.

The above suggested pile-pile cap connection details were investigated in this project by testing six full-scale connections, Units PC1 to PC6, subjected to simulated seismic loading. For Units PC1 and PC2 the end of the pile was embedded in the pile cap. For Units PC3, PC4 and PC5 the pile steel was exposed and embedded in the pile cap. For Unit PC6 the connection was made by dowel bars. The overall dimensions of the six pile-pile cap units, and the connection details, are shown in Figs. 11 to 15.

Fig. 11. Overall dimensions of the six pile-pile cap units tested.
Type B construction joint for embedded pile surface
(Surface roughness with amplitude of approximately 3mm.)

Fig. 12. Details of Units PC1 and PC2.

UNITS PC 1 & 2
All dimensions are in mm
1mm = 0.0394 in

Fig. 13. Details of Units PC3 and PC4.

UNIT PC 5
All dimensions are in mm
1mm = 0.0394 in

Fig. 14. Details of Unit PC5.
Fig. 15. Details of Unit PC6 and of pile cap reinforcement for Units PC1 to PC6.

Table 3. Main properties of the pile-pile cap units.

<table>
<thead>
<tr>
<th>Unit</th>
<th>Concrete compressive strength at stage of testing, ( f'_c ) (MPa)</th>
<th>Prestress in pile concrete at stage of testing ( f_{cp} ) (MPa)</th>
<th>Longitudinal steel</th>
<th>Spiral reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pile</td>
<td>Cap</td>
<td></td>
<td></td>
</tr>
<tr>
<td>PC1</td>
<td>43.5</td>
<td>33.2</td>
<td>7.3</td>
<td>0.0070</td>
</tr>
<tr>
<td>PC2</td>
<td>43.6</td>
<td>37.1</td>
<td>8.2</td>
<td>0.0070</td>
</tr>
<tr>
<td>PC3</td>
<td>56.0</td>
<td>30.8</td>
<td>7.6</td>
<td>0.0070</td>
</tr>
<tr>
<td>PC4</td>
<td>51.4</td>
<td>33.9</td>
<td>8.3</td>
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<tr>
<td>PC5</td>
<td>53.2</td>
<td>31.6</td>
<td>7.4</td>
<td>0.0070</td>
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<tr>
<td>PC6</td>
<td>50.2</td>
<td>24.9</td>
<td>7.2</td>
<td>0.0070</td>
</tr>
</tbody>
</table>

Notes:
1. Applied external load \( P_e = 0.2 f'_c A_e \), where \( f'_c \) = compressive strength of pile concrete at stage of testing.
2. Pretensioned tendons in each pile: ten 12.5 mm (0.79 in.) diameter seven-wire strands.
3. Spiral reinforcement in potential plastic hinge regions was of 12 mm (0.47 in.) diameter Grade 275 (\( f_y = 40 \) ksi) nonprestressed deformed bars.
4. The epoxy resin used to bond the dowel bars in Unit PC6 had a compressive strength at the time of testing the unit of 109 MPa (15,800 psi) as measured by 25 mm (1.0 in.) diameter by 50 mm (2.0 in.) high cylinders.
Details of the Pile-Pile Cap Units

(a) Pile Details

Table 3 lists the main properties of the piles. The piles were of 400 mm (15.7 in.) octagonal cross section. Each pile was pretensioned by ten 12.5 mm (½ in.) diameter seven-wire strands (see Fig. 11). The prestress in the pile concrete at the stage of testing \( f_{pc} \) is listed in Table 3. The piles of Units PC2 and PC4 contained in addition non-prestressed longitudinal reinforcement consisting of ten 20 mm (0.79 in.) diameter Grade 275 (\( f_y = 40 \) ksi) deformed steel bars.

The transverse reinforcement in the potential plastic hinge regions of the piles was spiral reinforcement of 12 mm (0.47 in.) diameter Grade 275 (\( f_y = 40 \) ksi) plain round steel at 47 mm (1.9 in.) centers. The length of this confined region in each pile, adjacent to the face of the pile cap, was 560 mm (22.0 in.), which was greater than the NZS 3101 code requirement. The \( \rho_r = 0.0293 \) provided in the confined region varied between 0.92 and 1.14 of the required quantity calculated from Eqs. (1) and (2) with \( P_r \) replaced by \( P_r + f_{pc} A_k \), using the actual material strengths and a strength reduction factor \( \phi = 1.0 \).

The center to center spacing of the spirals was 3.8 strand diameters and 2.4 longitudinal reinforcing bar diameters. The spiral steel within the piles was anchored by welded laps. The spiral reinforcement provided for confinement was in excess of that required by NZS 3101 to resist the shear forces resulting from the overstrength in flexure.

(b) Pile Cap Details

The reinforcing details of the six pile cap units are shown in Figs. 12, 13, 14 and 15. The reinforcement placed in the six pile caps is shown in detail in Fig. 15.

The pile cap reinforcement was nonprestressed and was designed to withstand the moments transferred to the cap by a possible overstrength moment from the pile of 1.5 times the nominal (ideal) flexural strength of the pile calculated using the NZS 3101 and ACI 318 code approach, using a concrete compressive stress block with a mean stress of 0.85 \( f_{ck} \), assuming an extreme fiber concrete compressive strain of 0.003 and using the measured stress-strain characteristics of the steel. This overstrength factor for a pile containing the code specified quantity of confining steel and \((P_r + f_{pc} A_k) f_{ck} A_k = 0.36\) was based on overstrength factors for reinforced concrete columns found previously.

The main longitudinal reinforcement in the pile cap consisted of four 24 or 28 mm (0.94 or 1.10 in.) diameter Grade 275 (\( f_y = 40 \) ksi) steel deformed bars placed in both the top and bottom of the pile cap. The ends of this longitudinal reinforcement were welded to threaded rods for connection with the loading frame (see Fig. 16). In addition, two 12 mm (0.47 in.) diameter deformed bars were provided at each side of the pile cap as longitudinal face steel. No longitudinal pile cap reinforcement passed through the embedded end of the pile. The four-legged stirrups of 12 mm (0.47 in.) diameter Grade 275 (\( f_y = 40 \)

![Fig. 16. Loading arrangements for the pile-pile cap units.](image-url)
ks)', plain steel bars, placed at 300 mm (11.8 in.) spacing, were designed to carry one-half of the externally applied axial load on the pile.

(c) Connection Details

For Units PC1 and PC2, shown in Fig. 12, the end of the pile was embedded over an 800 mm (31.5 in.) length of the cast-in-place pile cap. The surfaces of the ends of the piles to be embedded were roughened by a pneumatic hammer before the pile caps were cast. This surface roughness had an amplitude of approximately 3 mm (0.12 in.), which was in accordance with a Type B construction joint of NZS 3109.14 A light spiral, from 10 mm (0.39 in.) diameter Grade 275 ($f_y \geq 40$ ksi) steel plain bar, was placed around the pile in the pile cap.

It should be noted that the spiral within the pile continued to the end of the pile. The 800 mm (31.5 in.) embedment length of pile gave an anchorage length, measured from the pile cap face, of 800/12.5 = 64 strand diameters for the prestressing strands for both Units PC1 and PC2 and 800/20 = 40 bar diameters for the longitudinal non-prestressed steel in Unit PC2. This was an easy connection to construct since no breaking back of the pile concrete was necessary.

For Units PC3 and PC4, shown in Fig. 13, the pile steel was exposed over the end 600 mm (23.6 in.) and the end 650 mm (25.6 in.) of pile was embedded in the pile cap. In practice the ends of the piles would be jack hammered after driving to break back the concrete to expose the steel. However, in these laboratory specimens jack hammering was mainly avoided by blocking out the ends of the piles when they were precast to leave almost the required length of end steel exposed. Nevertheless, some jack hammering of the pile end was necessary after precasting to achieve the required length of pile in the laboratory. The projecting prestressing strands in both units, and the non-prestressed longitudinal steel in Unit PC4, were left straight and were surrounded by the same spiral steel as in the potential plastic hinge region of the pile. The 650 mm (25.6 in.) embedment length gave an anchorage length, measured from the pile cap face, of 52 strand diameters and 33 nonprestressed bar diameters.

For Unit PC5, shown in Fig. 14, the pile steel was exposed over the end 850 mm (33.5 in.) and the end 900 mm (35.4 in.) of pile was embedded in the pile cap. In addition, each exposed tendon had an "olive" shaped anchorage with its center at 600 mm (23.6 in.) from the face of the pile cap. The "olive" anchorage was formed in each seven-wire tendon by unraveling the tendon back to the anchorage position, placing a 12.7 mm (1/2 in.) diameter steel nut over the central wire of the tendon, and twisting back the other six wires of the tendon to take up their original strand form. The length of strand beyond the "olive" anchorage was sufficient to prevent the tendon from unraveling when twisted back. The other details of Unit PC5 were as for Unit PC3.

For Unit PC6, shown in Fig. 15, the connection was made by dowel bars. Four 20 mm (0.79 in.) diameter Grade 275 ($f_y \geq 40$ ksi) deformed steel bars were inserted into 40 mm (1.6 in.) diameter holes drilled 530 mm (20.9 in.) deep into the end of the pile. Epoxy resin, consisting of two parts resin and one part hardener, was used to bond the bars to the pile concrete. The ends of the dowel bars in the pile cap were anchored by 90 degree standard hooks and were surrounded by a spiral with the same bar diameter, but twice the spacing, as the spiral used in the end of the pile. The anchorage lengths of the dowel bars in the pile and in the pile cap satisfied the requirements of NZS 3101.9

(d) Construction Details and Material Properties

The piles were cast and cured in the stressing bed of a precast concrete factory and transported to the University of Canterbury where the pile caps were cast in place. The pile caps were cast with the pile axis in the horizontal plane.

Table 3 shows the prestress in the concrete of the piles achieved at the time of testing the units, taking into account the losses in prestress occurring up to that time. Demec (demountable mechanical) strain gauges were used to measure the concrete strains on the surface of the piles directly after the transfer of prestress and up to the time of testing. This enabled an estimate to be made of the losses in prestress due to concrete shrinkage and creep.

The compressive concrete cylinder strengths of the pile and pile cap of each unit, at the stage of testing the units, are listed in Table 3. At the stage of testing the units, the concrete in the piles was 187 to 238 days old and the concrete in the pile caps was 75 to 135 days old. All concrete was provided by a ready mix concrete firm. The compressive strength of the epoxy resin used to bond the dowel bars in Unit PC6 is shown in the notes of Table 3.

The measured stress-strain curves for the non-prestressed longitudinal and spiral steel, and for the prestressing tendons, are shown in Figs. 3 and 4 and the steel strengths are listed in Table 3.

Electrical resistance strain gauges [5 mm (0.20 in.) Showa foil gauges] were placed on the spiral reinforcement in selected positions in the pile and pile cap, and also on some longitudinal non-prestressed steel in the piles. Linear potentiometers were used to measure the displacements of the units during testing and the curvatures in the critical regions of the piles (see Fig. 16).

Testing of the Pile-Pile Cap Units

Fig. 16 shows the loading arrangements for the pile-pile cap units. The units were tested upside down. A DARTEC testing machine applied an axial load to the pile of $0.2 f'_c A_p$. This load was held constant throughout the test and simulated the vertical load from the structure above the pile. Cyclic bending moment and shear force, simulating the effects of seismic loading, were applied to each unit by a steel cantilever arm which was bolted to the ends of the longitudinal reinforcement of the pile cap at one end of the pile cap.

This enabled a horizontal load to be applied cyclically by an MTS actuator acting between the top end of the pile and the top end of the steel cantilever. Fig. 17 shows the deformations of the pile-pile cap units during testing. A
pattern of statically applied displacement cycles was used to simulate lateral seismic loading.

The displacement ductility factor $\mu$ was defined as the maximum displacement reached in the loading run $\Delta$ divided by the displacement at yield $\Delta_y$, where $\Delta_y$ was defined as the displacement measured in the first loading cycle when 0.75 of the ideal flexural strength of the pile was reached multiplied by 1.33, as for the pile tests. The first loading cycle was followed by displacement controlled loading consisting of cycles to displacement ductility factors $\mu$ of increasing magnitudes up to at least $\mu = \pm 8$.

**Observed Behavior of Pile-Pile Cap Units**

Figs. 18 to 23 show the measured lateral load $H$ versus lateral displacement $\Delta$ hysteresis loops for the six pile-pile cap units. The gradual reduction in the lateral load capacity of the units with increasing displacements is mainly due to the $P-\Delta$ effect of the axial loading rather than to degradation of the flexural strength of the units. Photographs illustrating damage of the units observed at large displacements are also shown in the figures. The damage in the units concentrated in the critical regions of the piles and not in the pile caps.

Units PC1 to PC5 exhibited stable load-displacement hysteresis loops up to displacement ductility factors of at least approximately $\mu = \pm 8$. Hence, any loss of anchorage of the prestressing strands of the piles which may have occurred in the pile caps did not seriously influence the behavior of the units during cyclic lateral loading. As indicated previously, the embedment length of the strands at the ends of the piles in the pile cap was much less than the approximately 140 strand diameters required for full development recommended by NZS 3101 and ACI 318. It is evident that the spiral reinforcement around the strands in the anchorage region assisted to maintain anchorage of the strands.

The measured lateral load-displacement hysteresis loops of Units PC1 and PC3 show less energy dissipation than the loops of the companion Units PC2 and PC4, respectively. This was because Units PC2 and PC4 con-
tained nonprestressed longitudinal reinforcement in the piles in addition to prestressing tendons. The “olive” anchorages for the tendons of Unit PC5 resulted in higher flexural strengths being reached by that unit than for the otherwise equivalent Unit PC3, due to better anchorage of the tendons in Unit PC5. Unit PC6, with dowel bars forming the connection, was more flexible than the other units.

The post-elastic deformations of Unit PC6 concentrated in a wide crack that formed at the pile-pile cap interface and not in the pile, and hence the plastic hinging did not spread along the pile as in the other units. In all six units only minor cracking was observed in the pile cap during testing. Some bulging of the pile cap concrete around the pile end was also observed in Unit PC6.

The electrical resistance strain gauges on the spiral reinforcement of all units except Unit PC6 showed significant tension yielding of the spirals in the extreme concrete compression fibers of the plastic hinge region of the piles. Some yielding of spirals was also recorded in those piles within the pile cap. For Unit PC6 the spirals in the extreme compression fibers barely reached yield, indicating again that yielding in that unit did not spread along the pile but concentrated at the end of the pile. The strain gauges on the spirals at the sides of the piles, measuring mainly spiral strains due to shear, indicated only limited yielding due to shear in all units.

Spalling of the cover concrete of the piles was observed to commence at a displacement ductility factor of approximately $\mu = 2$.

**Methods of Theoretical Flexural Strength Analysis**

The strength of the pile-pile cap units as designed was governed by the flexural strength of the piles. The theoretical flexural strengths of the piles, at the critical section at the face of the pile cap, were calculated by four alternative theoretical methods. Each method used the requirements of strain compatibility and equilibrium of internal forces, and assumed that plane sections remain plane after bending. Further details of the four theoretical methods are outlined below. In all methods, the strength reduction factor $\phi$ was assumed to be 1.0.

**Method A** — The theoretical flexural strength was calculated assuming a rectangular concrete compressive stress block with a mean stress of $0.85\ f'_c$ and an extreme fiber concrete compressive strain of 0.003 as recommended by NZS 3101 and ACI 318, using the measured stress-strain curves of the prestressing tendons and the
non prestressed longitudinal deformed bar reinforcement, and assuming full development of the longitudinal steel. Note that the prestressing tendons in the pile cap of all units, except Units PC1 and PC2, were not prestressed and hence were treated as nonprestressed steel in the calculations.

Method B — The theoretical flexural strength was calculated as for Method A, except that the effect of the possible lack of full development of the tendons at the critical section was taken into account. According to ACI 318 and NZS 3106,\(^9\) for strand the transmission length is approximately 50 \(d_\text{s}\), and the development length is approximately 140 \(d_\text{s}\), where \(d_\text{s}\) is the strand diameter.

For Units PC1 and PC2 the length of strand in the pile cap was 800 mm (31.5 in.) which was 64 \(d_\text{s}\). For these two units it was assumed that within the transmission length the stress in the tendon increased from zero at the end of the pile to \(f_{p,\text{m}}\), due to prestress at 50 \(d_\text{s}\), from the pile end. Beyond that point the achievable stress in the tendon increased linearly from \(f_{p,\text{m}}\) to \(f_{p,\text{c}}\) over the next 90 \(d_\text{s}\). On this basis, the achievable stress in the tendons at the critical section was 1168 and 1278 MPa (169 and 185 ksi) for Units PC1 and PC2, respectively. It should be noted that \(f_{p,\text{c}} = 1901\) MPa (276 ksi).

For Units PC3 and PC4 the tendons were not prestressed in the pile cap. The 650 mm (25.6 in.) (52 \(d_\text{s}\)) length of strand in the pile cap was assumed to be capable of developing a tendon stress at the critical section of the pile at the face of the pile cap of (52 \(d_\text{s}/140 d_\text{s}\)) \(f_{p,\text{m}} = 706\) MPa (102 ksi).

For Unit PC5 the tendons were also not prestressed in the pile cap. The 900 mm (35.4 in.) (72 \(d_\text{s}\)) length of strand in the pile cap was assumed to be capable of developing a tendon stress at the critical section of 978 MPa (142 ksi) which neglected the additional anchorage of the "olives." The nonprestressed deformed bar reinforcement in Units PC2, PC3 and PC6 had sufficient embedment length in the pile cap for full development according to NZS 3101.\(^9\)

Method C — The theoretical flexural strength was calculated as for Method A, except that the concrete compressive stress distribution of the concrete core of the pile was obtained using the stress-strain curve for spirally confined concrete proposed by Mander et al.\(^7\) The concrete compressive strain at the extreme fiber of the core \(\varepsilon_{c,\text{m}}\), was assumed to be that at 0.8 \(f_{c,\text{r}}\) on the falling branch of the stress-strain curve, where \(f_{c,\text{r}}\) is the compressive strength of spirally confined concrete. \(R = 3\) in Eq. (3) and \(f_{c,\text{r}} = f_{c,\text{r}}\).
were assumed. For the range of spiral reinforcement in the six units it was found that $f_{c} / f'_{c}$ varied between 1.49 to 1.62 and $\epsilon_{c}$ varied between 0.010 to 0.014.

Method D — The theoretical flexural strength was calculated as for Method A, except that the possible lack of full development of the tendons at the critical section was taken into account as for Method B and the stress-strain curve for spirally confined concrete was used as for Method C.

Comparison of the Theoretical Flexural Strengths of the Units

Table 4 shows the theoretical flexural strengths of the piles at the section at the face of the pile cap calculated by Methods A, B, C and D. It is apparent that the differences between the theoretical flexural strengths calculated by Methods A and B are very small. This is because of the relatively small extreme fiber concrete compressive strain of 0.003 and the deep neutral axis depth which resulted in relatively low tensile strains in the longitudinal steel. Hence, very few tendons had insufficient anchorage to develop the stress required of the strain distribution.

By contrast, there is a large difference between the theoretical flexural strengths calculated by Methods C and D. This is because of the relatively large extreme fiber concrete compressive strain which resulted in large tensile strains near the extreme tension fiber. Hence, many tendons had insufficient anchorage to develop the stress required of the concrete strain distribution. That is, the achievable tensile stress in those tendons was less than the stress required by the concrete strain distribution. Note that the effect of the loss of the cover concrete at large extreme fiber concrete strains was taken into account in Methods C and D. Cover concrete was assumed to have spalled at strains greater than 0.004.

It is apparent that the differences between the theoretical flexural strengths given by Methods A and D for each pile are not large in the case of Units PC1 to PC5. This is because the increase in flexural strength due to concrete confinement at high curvatures in Method D was reduced by the inability of some tendons to develop very high tensile stresses, as a result of inadequate anchorage. To take advantage of the beneficial effect of concrete confinement would require longer strand anchorage lengths than the 52 $d$, or 64 $d$, provided in these units.

For Unit PC6 the flexural strengths calculated using Methods C and D were less than those calculated using Methods A and B, because of the more significant effect of the loss of cover concrete for that unit.

Figs. 18 to 23 show plotted the theoretical lateral load strengths of the units calculated using the flexural strengths given by Methods A, B, C and D. The slope of the theoretical lines is due to the $P-\Delta$ effect which results in the lateral load strength reducing with increasing lateral displacement. The difference between the strengths given by Methods A and B are so small that those strengths are plotted as a single line in the figures for some of the units.

Moment-Curvature Analyses of Piles of Pile-Pile Cap Units

Theoretical moment-curvature relations for the critical section of the pile at the face of the pile cap were computed for each prestressed concrete pile of the pile-pile cap units. The analyses used the requirements of strain com-
patibility and equilibrium of internal forces and assumed that plane sections remain plane after bending. The stress-strain curve for spirally confined concrete proposed by Mander et al. [with $R = 3$ in Eq. (3) and $f'_{tu} = f'_{c}$], and the measured stress-strain curves for the tendons and non-prestressed longitudinal reinforcement, were used.

Two theoretical moment-curvature relations were calculated for each pile. The first relationship was obtained assuming full development of the longitudinal steel. The second relationship was obtained taking into account the possible lack of full development of the tendons, if the development length provided in the unit was inadequate. Full details of the assumptions and the computer programs are given elsewhere.5,15

Figs. 24 to 29 show the two theoretical monotonic moment-curvature relationships computed for the critical section of the pile at the face of the pile cap of Units PC1 to PC6. It can be seen that for Units PC1 to PC5, as expected, the theoretical moment-curvature relationships calculated assuming full development of the strand lie above the theoretical relationships calculated taking into account the possible lack of full development. For Unit PC6 (Fig. 29) the opposite occurs because in the calculation the effect of the presence of the tendons was omitted entirely because of the short length of tendons beyond the critical section.

**Comparison of the Theoretical and Experimental Flexural Strengths**

Table 4 shows the experimental maximum moments for the piles, measured at the face of the pile cap, compared with the theoretical flexural strengths.

The comparison is also shown in terms of lateral loads in Figs. 18 to 23. It is apparent that for Units PC1 to PC5 the measured peak lateral loads in most cycles were well above the theoretical strengths given by Methods A, B and D. The theoretical strengths given by Method C, which assumes full development of the longitudinal steel, and takes into account confinement of the concrete, overestimates the measured strengths of all of the piles, except that of Unit PC6 and some loading cycles for Unit PC5. It can be observed that the flexural strength of all units was given conservatively by Method A, because apparently in these tests the loss of flexural strength due to possible inadequate anchorage of the tendons was compensated by the increase in strength due to confinement of the concrete.

The experimental points measured at the peaks of the loading cycles during the tests on each unit are also plotted in the moment-curvature diagrams shown in Figs. 24 to 29. The experimental curvatures plotted are the average curvatures measured over the 150 mm (5.9 in.) gauge length of pile adjacent to the face of the pile cap (see Fig. 16). For Units PC1 to PC4 (Figs. 24 to 27) most of the experimental points lie between the two theoretical curves, indicating that the development length assumptions of NZS 3101 and ACI 318 are for strand, when spirally confined as in these units, are conservative. The spiral steel around the strand in the pile end in the pile cap was evi-
conveniently beneficial in assisting to maintain anchorage of the strand.

For Unit PC5 (Fig. 28) many experimental points lie above the theoretical moment-curvature curve obtained assuming full development of the strand, which would have been due to the improved anchorage of the strand achieved by the "olive" type detail (see Fig. 14).

For Unit PC6 (Fig. 29) most of the experimental points lie above the moment-curvature curves given by both theoretical methods. It should be noted that the experimental curvatures plotted in Figs. 24 to 29 were measured by potentiometers bearing against the surface of the pile cap (see Fig. 16) and hence include the deformations due to any slip of longitudinal steel in the pile cap as well as due to pile bending. This slip was particularly noticeable for Unit PC6.

It is apparent from Figs. 18 to 29 and Table 4 that the proposed theoretical Method D, for calculating the theoretical flexural strength of a prestressed concrete pile at the face of the pile cap, is a reasonably conservative approach. For example, the values of $M_{\text{max,exp}}/M_D$ listed in Table 4 for Units PC1 to PC6 are 1.24, 1.15, 1.27, 1.23, 1.29 and 1.17, respectively. Method D uses the stress-strain curve for spirally confined concrete and includes the effect of possible inadequate anchorage of the tendons. The conservatism of Method D is partly due to the conservative recommendations of NZS 3101 and ACI 318 for the development length of strand when placed within spirally confined concrete.

Also, the values of flexural strength listed in Table 4 calculated by Method D assume that the flexural strength is reached when the concrete compressive strain in the extreme fiber of the confined core reaches the value corresponding to $0.8 f_{cc}'$ on the falling branch of the stress-strain curve of the concrete, where $f_{cc}'$ is the compressive strength of confined concrete. The theoretical monotonic moment-curvature relationships plotted in Figs. 24 to 29, which assume possible inadequate anchorage of the strand at the critical section of the piles, gave peak moments which for Units PC1 to PC5 were on average 4 percent higher than the values for Method D listed in Table 4. Hence, the simplifying assumption made to obtain the concrete compression strain at the extreme fiber of the confined core at maximum moment for Method D is adequate.

Figs. 24 to 29 also indicate that the theoretical monotonic moment-curvature relations are in reasonable agreement with the experimental points obtained from the cyclic load tests for these units. In general, the monotonic moment-curvature theory gave conservative estimates for the available ultimate curvature of the piles. The available ultimate curvatures of the piles of Units PC3, PC5 and PC6 measured experimentally exceeded that calculated theoretically (see Figs. 26, 28 and 29).

**CONCLUSIONS**

1. Pretensioned prestressed concrete piles, with 400 mm (15.7 in.) octagonal cross sections designed using the
NZS 3101\(^{9}\) seismic design provisions for spiral reinforcement, when tested displayed ductile behavior. Very stable lateral load-lateral displacement hysteresis loops were exhibited when the piles were subjected to axial load and to imposed cyclic lateral displacements well into the postelastic range. The presence of nonprestressed longitudinal reinforcement in prestressed concrete piles is not essential to the satisfactory ductile performance of the pile but does permit a greater dissipation of seismic energy by the pile. The spiral spacing was 3.8 strand diameters and this close spacing effectively prevented premature buckling of the 12.5 mm (1/2 in.) diameter seven-wire strands in the piles.

2. Several different details for achieving a moment resisting connection between the end of a precast prestressed concrete pile with a 400 mm (15.7 in.) octagonal cross section and a cast-in-place reinforced concrete pile cap were tested. All connection details tested were found to permit the flexural strength of the pile to be reached and permitted satisfactorily ductile behavior of the connection. Only minor cracking was observed in the pile cap. Conclusions concerning the connection details and their performances are:

(a) The preferred type of pile-pile cap connection is considered to be when the end of the precast concrete pile is roughened and is embedded in the cast-in-place concrete pile cap. A surface roughness of 3 mm (0.12 in.) amplitude was used in the tests. This type of connection was the easiest to construct and resulted in the least damage to the pile cap during the tests. The second most preferred connection type is when the concrete at the end of the pile is broken back and the exposed tendons are left straight and are embedded within the cast-in-place concrete pile cap. A third connection type, consisting of dowel bars bonded with epoxy resin in holes drilled in the pile end and anchored in the pile cap, are not so preferred because it was observed during the tests that the damage during inelastic cyclic loading concentrated in a wide crack at the end of the pile.

(b) Spiral steel, similar to that in the potential plastic hinge region, should be provided within the region of the pile that is embedded in the pile cap, especially in the broken back pile head type. The major role of this spiral steel is to improve the bond of the strands and also to assist the transfer of the lateral forces to the surrounding concrete within the pile cap.

(c) A satisfactory comparison was obtained between the experimentally measured peak moments and corresponding curvatures and the theoretical monotonic moment-curvature relationships of the piles. The theoretical moment-curvature relationships were calculated assuming a stress-strain model for spirally confined concrete, using the actual stress-strain relations for the tendons and nonprestressed longitudinal steel, and accounting for possible inadequate anchorage of the tendons based on the development lengths for strand recommended by codes. This procedure results in a suitable theoretical method for determining the flexural strength of prestressed concrete piles at the critical section at the face of the pile cap.

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APPENDIX — NOTATION

\[\begin{align*}
A_c &= \text{area of concrete core measured to outside of spiral} \\
A_g &= \text{gross area of concrete section} \\
A_{pt} &= \text{total area of prestressing strand} \\
A_{st} &= \text{total area of nonprestressed longitudinal reinforcement} \\
d_p &= \text{diameter of prestressing tendon or reinforcing bar} \\
f'_c &= \text{compressive cylinder strength of concrete} \\
f_{cc}' &= \text{compressive strength of spirally confined concrete} \\
f_{cp} &= \text{axial compressive stress in concrete due to prestress alone} \\
f_{pe} &= \text{stress in tendon due to prestress alone} \\
f_{pu} &= \text{ultimate strength of prestressing steel} \\
f_{su} &= \text{ultimate strength of nonprestressed steel} \\
f_y &= \text{yield strength of nonprestressed longitudinal reinforcing steel} \\
f_{yh} &= \text{yield strength of spiral reinforcing steel} \\
H &= \text{horizontal load} \\
M_{A}, M_{B}, M_{C}, M_D &= \text{theoretical flexural strength of pile calculated by Methods A, B, C and D, respectively} \\
M_{max} &= \text{maximum moment capacity of pile} \\
P_e &= \text{axial compression load} \\
P_o &= \text{concentric load strength of member} \\
s &= \text{spiral pitch (center to center spacing)} \\
\Delta &= \text{horizontal displacement} \\
\Delta_y &= \text{horizontal displacement at yield} \\
\varepsilon_{cc} &= \text{concrete compressive strain when strength of confined concrete } f_{cc}' \text{ is reached} \\
\varepsilon_{co} &= \text{concrete compressive strain when strength of unconfined concrete } f'_c \text{ is reached} \\
\varepsilon_{cm} &= \text{concrete compressive strain at extreme compression fiber of core} \\
\varepsilon_{fr} &= \text{fracture strain of nonprestressed steel} \\
\varepsilon_{pu} &= \text{ultimate tensile strain of prestressing steel} \\
\mu &= \text{displacement ductility factor } = \frac{\Delta}{\Delta_y} \\
\rho_s &= \text{ratio of volume of spiral reinforcement to volume of concrete core} \\
\rho_{st} &= \frac{A_{st}}{A_g} \\
\rho_t &= \frac{A_{pt}}{A_g} \\
\rho_s \text{ code} &= \text{code required } \rho_s \text{ calculated using Eqs. (1) and (2)} \\
\rho_s \text{ provided} &= \rho_s \text{ used in the pile units} \\
\phi &= \text{strength reduction factor}
\end{align*}\]