Ports are a vital link in the transportation of goods worldwide, with more than $1 trillion of goods flowing through U.S. shipping terminals every year. Economies worldwide rely heavily on ports, and it is estimated that $1000 is brought into the local economy from each shipping container. Many ports are located in high seismic regions and are susceptible to strong ground motions, resulting in the potential for devastating physical and economic damage. In addition, most ports are built on poor soils that are susceptible to liquefaction, lateral spreading, and differential settlements, which compound the likelihood of earthquake damage.

Ports damaged by earthquakes require repair. Equally or more important, ports suffer loss of income and reduced activity due to downtime. The loss of income can be long term or permanent as tenants move to other ports while the damaged port is repaired, and the tenants may not return after repairs are completed. This occurred after the 1995 Hanshin earthquake in Japan. Before the earthquake, the Port of Kobe was ranked the 6th busiest port for container shipping in the world. In 1997, shortly after the earthquake, the port had slipped to 17th, and by 2005 it had dropped to 39th.

Because of the short-term and long-term vulnerability, ports must be designed to withstand strong ground motions with minimal damage. Most marginal wharves are partially or fully supported by plumb precast, prestressed concrete piling (Fig. 1). Vertical-pile-supported wharf structures are designed to act as ductile moment frames with plastic...
tent observations can be made.

Figure 2 demonstrates some of these observations through consideration of the lateral resistance-deflection and moment-rotation behavior of three test specimens. Figure 2 shows the behavior of an extended pile connection without axial load as well as the behavior of embedded dowel connections with precast concrete piles without and with axial load. The axial load for the precast concrete pile connection was approximately 10% of the axial load capacity ($0.10 f_c A_g$, where $f_c$ is the specified compressive strength of concrete and $A_g$ is the gross cross-sectional area of the pile) of the prestressed concrete pile, which is within the range of the axial loads expected in practice.

Figure 2 shows the differences in the behavior of the extended-pile connection and the embedded-dowel connection. The extended-pile connection behaves largely as a cast-in-place reinforced concrete connection and limits deterioration and loss of resistance to larger drifts. The precast concrete pile connection without an axial load has more deterioration in resistance and moment capacity but similar maximum resistance to an identical extended pile connection.

Precast concrete piles deteriorate at lower deformations and more severely than cast-in-place concrete column-deck...
connections because the flexural deformations are limited in the piles, which largely act as rigid bodies and rock with connection rotation. The connection must sustain the cyclic deformations. The rocking action causes large edge stresses on the pile and wrenching action of the end of the pile that is embedded in the deck. This leads to early spalling of the pile and deck (Fig. 3). Typically, the cover on the pile is thick (more than 3 in. [76 mm]) to limit corrosion of the prestressing strands, and loss of this cover results in a more significant decrease in the bending resistance than in a typical reinforced concrete column. This loss of concrete cover results in loss of resistance with increasing deformation (Fig. 2). Extended pile connections are used only in special cases where the pile is driven below the bottom of the soffit of the wharf deck. As a result, dowel connections with precast concrete piles are most common and do not meet the performance requirements.

The axial stress demand is important. In a marginal wharf application, the piles support compressive load due to the heavy deck system. The addition of compressive load to the pile significantly changes the connection response (Fig. 2). The pile is prestressed, which results in large compressive stresses. The addition of an axial load of
only 10% of the gross axial capacity, as was used for the specimen that resulted in the measured behavior in Fig. 2, significantly changes the response. The axial load significantly increases the moment capacity and maximum lateral resistance of the connection but also dramatically increases the rate of deterioration in resistance. Part of the lost lateral resistance can be attributed to load-deflection \( P-\Delta \) effects at large deflections (Fig. 2). These \( P-\Delta \) moments reduce the lateral resistance because a portion of the moment capacity is consumed by \( P-\Delta \) effects.

The deterioration noted with the moment-rotation curves is a true measure of damage to the pile and its connection. At large rotations, the compressive force at the edge of the pile, associated with the bending moment and further increased by the axial load, results in large compressive stresses. Hence, with an increase in axial force, spalling occurs at smaller deformations (Fig. 3) and is more severe, resulting in larger deterioration (Fig. 2).

Figure 3 shows damage of the three connections at the same connection rotation level of 0.03 rad. The actual displacement of the pile depends on its effective length. For a short pile at the end of the wharf, the displacement associated with this level of rotation is relatively small because the effective pile length is small. The extended pile connection (Fig. 3) is largely intact with flexural cracking. The precast concrete pile with a dowel connection and no axial load (Fig. 3) is still intact, but the crack pattern suggests that spalling will initiate at slightly larger deformations.

The precast concrete pile with a dowel connection and axial load (Fig. 2 and 3) sustains significant spalling. Other specimens exhibited this level of spalling but at significantly higher wharf displacements and connection deformations. Pile and deck spalling have structural implications beyond seismic resistance because they cause loss of prestressing force and expose the reinforcing steel to the harsh marine environment. Hence, repairs are required for these connections even after modest earthquakes. However, the spalling and its correlation to the deterioration in resistance suggest that the performance of the connections may be significantly improved at all performance levels if spalling is delayed.

**Characteristics of a damage-resistant pile-wharf connection**

A research program was conducted to improve the seismic performance of pile-to-wharf connections. The study examined the results of prior research, evaluated recent wharf and pile connection designs, and, in consultation with practicing engineers in this field, developed improved connection designs. Experiments were conducted to investigate the connection designs. Several options were proposed for experimental evaluation:

- **Partial debonding of the dowel bars into the wharf deck and the pile itself.** Debonding reduces the maximum strain demand in the steel. For a given strain, a debonded bar has a greater axial deformation than a bonded bar, a direct result of the strain gradient (uniform versus nonuniform). This delays dowel bar fracture and increases the rotation for a given strain. This technique has been used successfully in precast, prestressed concrete moment frames for buildings. A second feature of debonding the dowel bars is that it alleviates bond stress transfer in that region, which can damage the deck concrete.

- **Addition of a flexible material between the pile and deck to sustain the rotation and reduce damage.** Rocking of the pile under cyclic loading results in edge loading of the pile and contributes to premature concrete damage (Fig. 3). To reduce the damage, the edge loading needs to be distributed over a larger portion of the pile cross section to reduce the compressive stress. Here, a bearing pad strong enough to sustain the compressive stress but flexible enough to sustain the rotation was used.

- **Flexible joint sealant wrap around the embedded perimeter of the pile.** There was discussion with the engineers to modify the 3 to 4 in. (76 to 100 mm) embedment of the pile into the wharf deck by either increasing the embedment depth or eliminating it. This short embedment is approximately equal to the cover depth and results in high shear stress on unreinforced concrete, which in turn results in significant concrete damage. It was postulated that either a greater embedment depth or no embedment at all would reduce the spalling of the deck at larger rotations. However, the practicing engineers deemed neither option acceptable. Their concerns were that eliminating the embedment depth would result in intrusion of salt water into the connection and decrease the shear resistance. As an alternative, a gap was placed between the pile and deck and filled with a flexible material to reduce the shear stress demands. This material seals the joint to prevent intrusion of salt water into the connection.

**Experimental program**

An experimental program was initiated to investigate pile-wharf connections with these damage-mitigating characteristics. Eight full-scale precast concrete pile-wharf deck connections were tested. Each included one or more of these aspects of the connections, as well as the applied axial load and the bearing pad configuration and material. The eight test specimens had the same piles and deck reinforcing and configuration (Fig. 4 and 5). These aspects of the specimen design were determined from an infrastructure review of 14 marginal wharf structures from the Port of Los Angeles, the Port of Oakland, and the Port of Seattle.
Figure 4. Overall specimen dimensions and reinforcing steel. Note: no. 10 = 32M; W11 = 10M; 1 in. = 25.4 mm; 1 ft = 0.305 m.

Figure 5. Deck reinforcement (all specimens). Note: no. 8 = 25M; no. 11 = 36M; 1 in. = 25.4 mm; 1 ft = 0.305 m.
The specimens each had a 24 in. (610 mm) prestressed concrete pile and used an embedded T-headed dowel bar connection. The pile had twenty-two 0.500 in. (13 mm) diameter low-relaxation strands with strengths of 270 ksi (1860 MPa). The strands were pretensioned to 31 kip (140 kN), resulting in a service-level prestress of approximately 1.4 ksi (9.7 MPa) after accounting for relaxation stress losses. The spiral reinforcement was W11 (9.5 mm diameter) smooth wire, and the pitch varied along the length of the pile from 1 in. (25 mm) at the ends to 3 in. (75 mm) in the middle (Fig. 4).

The pile concrete compressive strength data up to the 28-day strength was provided, but test cylinders were not available on the day of test. The pile concrete was a 5/8 in. (9.5 mm) maximum aggregate 8000 psi (55 MPa) mixture with a 3 to 9 in. (75 to 230 mm) slump. The piles used for the test were cast from one of three different batches, which had 28-day strengths of 9680 psi (66.7 MPa), 10,650 psi (73.4 MPa), and 8410 psi (58.0 MPa). Due to incomplete documentation, determining which pile came from which batch was not possible. For numerical strength prediction of the test specimens the pile concrete strength was taken to be the average of the three batch strengths, which is equal to 9580 psi (66 MPa).

The piles were embedded 3 in. (75 mm) into a cast-in-place reinforced concrete section representing a segment of the deck. The connection was made using eight ASTM A70613 (60 ksi [410 MPa] minimum yield stress) T-headed no. 10 (32M) dowel bars, which were embedded 20 in. (510 mm) into the deck. The dowels were 6.33 ft (1.93 m) long and were grouted 4.9 ft (1.5 m) into corrugated ducts in the pile using high-strength, nonshrink grout. The grout achieved 5000 psi (35 MPa) and 10,000 psi (70 MPa) compressive strengths in 24 hours and 28 days, respectively. The pile length was 8.6 ft (2.6 m) from the soffit of the deck to the horizontal loading point; this length was chosen to be representative of the short pile length in typical wharf structures based on prior analyses.14

The deck dimensions (Fig. 5) were maximized to ensure simulation of force transfer without interference from the test setup but were constrained to fit into the test rig. Figure 5 shows the deck reinforcement and simulates reinforcement layouts typically used in the prototype marginal wharves. Figure 6 and Table 1 present the specimens, which are identified as specimens 9 through 16 (numbered to be consecutive with the specimens studied by Graff8 and Soderstrom9). The specimens studied the connection modifications, axial load, and bearing pad characteristics. A brief description of each specimen follows:

- Specimen 9 used the embedded dowel bar connection commonly used in current design and serves as the reference specimen.
- The dowel bars in specimen 10 were intentionally debonded into the connection and into the deck, 7.5 in. (190 mm) for each. All other aspects of the connection were the same as for specimen 9.
- The connection of specimen 11 included the debonded region and the addition of a cotton duck bearing pad between the pile and deck interface. Cotton duck pads have high compressive strength and can accommodate large rotations.15 All bearing pads had an octagonal shape with 24 in. (610 mm) outside dimensions to cover the entire head of the pile.
- Specimen 12 was identical to specimen 11 (with a debonded region and a cotton duck pad) with the addition of a soft foam wrap around the perimeter of the length of the pile embedded into the deck.
- Specimen 13 was nominally identical to specimen 12...
The horizontal cyclic load was applied using a 220 kip (977 kN), 20 in. (510 mm) stroke actuator, which was attached to and reacted against the test frame. The actuator was connected to the pile using four 0.87 in. (22 mm) B7 steel rods and two 1 in. (25.4 mm) thick steel plates placed on either side of the pile. A 0.126 in. (3.2 mm) elastomeric pad was placed between the steel plates at the end of the actuator and the face of the pile to evenly distribute the bearing stresses.

The specimens were subjected to a constant vertical load. The cyclic lateral loads were applied under displacement control with increasing amplitude deformation cycles with a test protocol based on the Applied Technology Council’s ATC-24 Guidelines for Testing Steel Components procedure.

Figure 8 shows the external instruments used to monitor the specimen. String pots were used to monitor the horizontal movement at several locations along the pile length (P9-P11). Vertical potentiometers were used to monitor segment rotations, from which average curvatures could be calculated (P14-P21). The bottommost vertical potentiometers monitored connection rotation (P5-P6). Other potentiometers monitored specimen slip, overturning, and movement of the setup. A series of inclinometers was placed on either face of the specimen to monitor rotation; the inclinometers were at the lower end of the pile (Fig. 8). Stringer\textsuperscript{12} provides additional details of the test specimens, setup, and instrumentation.

**Test setup and procedure**

The specimens were constructed to simulate the field orientation (deck cast above the pile) but were tested in an inverted position to fit within the constraints of the test setup and laboratory. A self-equilibrating test rig (Fig. 7) was used. The deck portion of the specimen was posttensioned to a reaction block at the base of the test frame using four 1.25 in. (32 mm), 150 ksi (1030 MPa) high-strength threaded rods posttensioned to 125 kip (556 kN).

The vertical axial load was applied by a 2380 kip (10,600 kN) material testing machine. An assembly was constructed to ensure the axial load imparted minimal horizontal force on the loading head. A recessed, dimpled, and lubricated polytetrafluoroethylene sliding surface with a mirror-finished stainless steel plate mating was set inside a C15 × 15 (C380 × 74) section, which was attached to the loading head. A spherical bearing was placed below the sliding surface to accommodate end rotation of the pile tip.

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**Experimental results**

Assessment of specimen performance was made through consideration of the damage sustained through loading...
Figure 7. Test rig. Note: C15 × 50 = C380 × 74; HSS6 × 6 × 3/8 = 152.4 × 152.4 × 9.5; W14 × 90 = W360 × 134; W24 × 94 = W610 × 140. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN.

Figure 8. Overview of instrumentation layout.
and the measured global response. Figure 9 provides the moment-rotation response for six specimens. (Maximum moment-rotation response is shown because the force-drift response curves include degradation resulting from P-Δ effects. The moment-rotation curves show the actual specimen response.) Figure 9 also shows the analytical response calculated using the finite element software OpenSees, the details of which can be found in Stringer. Figure 10 compares connection performance of specimens 9 and 13.

Specimen 9 was the reference specimen and simulated current practice. The connection reached its peak moment resistance at a connection rotation of 1.4%; deterioration of resistance initiated shortly thereafter (Fig. 9). At the connection rotation of 8%, four dowel bars fractured and the moment resistance dropped to 47% of the maximum. At this deformation, the lateral resistance was essentially zero because of the deterioration and P-Δ moments. Figure 10 shows the observed response at approximately 2.5%, 4%, and 8.4% rotation, which are approximately the damage states corresponding to initial spalling, substantial spalling (includes complete spalling of the cover and damage to the core), and bar fracture.

Specimens 10 to 13 investigated fundamental improvements to the connection. Specimen 10 had debonded dowel bars and exhibited similar moment and shear capacities but less deterioration in resistance compared with speci-
The addition of the full bearing pad reduced the elastic stiffness of the connection by nearly 50% compared with specimens 9 and 10.

Specimen 14 was identical to specimen 13, but it was loaded with a 900 kip (4000 kN) axial load compared with 450 kip (2000 kN) for specimen 13. Specimen 14 exhibited a higher elastic stiffness and peak flexural resistance but had greater and more rapid degradation of resistance (55% loss) compared with specimen 13. This is consistent with observations in prior research (Fig. 2). The hysteresis loops remained full with less pinching than any other specimen tested, and energy dissipation was considerable. However, the P-Δ moments reduced the lateral resistance of the connection, exceeding the connection resistance at approximately 7% rotation. The test was terminated shortly after this deformation to protect the testing equipment because the actuator was supporting rather than loading the specimen. Dowel bar fracture did not occur for this test.

Specimens 15 and 16 used an annular 0.75 in (19 mm) thick cotton duck bearing pad and a 0.5 in (13 mm) thick random oriented fiber bearing pad, respectively, to improve the behavior of specimen 13. The debonded dowel bar steel caused initial pile cover spalling at a slightly lower rotation but delayed both substantial spalling and dowel bar fracture. The hysteretic loops of specimen 10 show significantly less degradation in resistance than specimen 9 between 8% and 9% connection rotation despite experiencing dowel bar fracture.

Figures 9 and 10 and Table 2 show that specimen 13 behaved differently from specimens 9 and 10. Specimen 13 included debonded dowel bars as well; however, a 0.75 in. (19 mm) thick, 24 in. (610 mm) diameter octagonal cotton duck bearing pad and a 0.75 in. (19 mm) thick flexible expansion material wrap around the perimeter of the embedded pile segment greatly reduced the damage at moderate to high drift. The behavior of specimen 13 was better than specimens 9 and 10 (Fig. 10) because it considerably delayed initial and substantial spalling, which occurred at 3.8% and 8.4% rotation, respectively. No dowel bars fractured during this test despite deformation cycles to 8.9% rotation. By 8% rotation the moment resistance had only reduced by 19% of the peak resistance, which was considerably less than either specimen 9 or 10. However, despite these significant improvements in performance, the addition of the full bearing pad reduced the elastic stiffness of the connection by nearly 50% compared with specimens 9 and 10.

Specimen 14 was identical to specimen 13, but it was loaded with a 900 kip (4000 kN) axial load compared with 450 kip (2000 kN) for specimen 13. Specimen 14 exhibited a higher elastic stiffness and peak flexural resistance but had greater and more rapid degradation of resistance (55% loss) compared with specimen 13. This is consistent with observations in prior research (Fig. 2). The hysteresis loops remained full with less pinching than any other specimen tested, and energy dissipation was considerable. However, the P-Δ moments reduced the lateral resistance of the connection, exceeding the connection resistance at approximately 7% rotation. The test was terminated shortly after this deformation to protect the testing equipment because the actuator was supporting rather than loading the specimen. Dowel bar fracture did not occur for this test.

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constructability and to increase the stiffness of the connection. These two tests evaluated different pad thickness and material because randomly oriented fiber pad is a recycled material and is less expensive than cotton duck. Comparing both tests with other specimens evaluates the effectiveness of the annular bearing pad concept. Both specimens experienced higher elastic and axial stiffnesses than specimen 13 and a slightly higher peak moment resistance. Spalling of the connection initiated at 5.5% rotation, a delay of 4.1% rotation, and 1.7% rotation from specimens 9 and 13, respectively.

Although the connection performance was dramatically improved prior to initial spalling, specimens 15 and 16 showed loss of resistance after spalling. Nevertheless, the total deterioration was significantly less than that of specimens 9, 10, and 14.

Significant spalling occurred suddenly for specimens 15 and 16 rather than progressively, as noted for the other specimens. The deterioration in resistance at the ultimate deformation was less for specimens 15 and 16 than for the other specimens. Comparison of these two specimens indicates that there was not a dramatic difference in the cotton duck pad and randomly oriented fiber pad performance. Both exhibited a similar delay in the initiation of dowel bar fracture.

The maximum moment resistance of all specimens was essentially identical for specimens with the bearing pads (specimens 13, 14, 15, and 16); these specimens had flexural resistance values that were approximately 85% to 96% of the original specimen. Table 1 shows that the yield stress of the dowel bars for specimens 9 and 10 is 11% higher than for specimens 13 through 16, and therefore this difference is not as significant as might be originally postulated. The difference is alleviated when considering the ratio of the peak-to-nominal moment strengths, as indicated in column 5 of Table 2. Whereas specimens 9 and 10 achieved approximately 1.08$M_n$ (where $M_n$ is the nominal moment strength), the remainder of the specimens achieved 0.98$M_n$ on average.

There are distinct differences in performance for specimens with no bearing pad over the head of the pile (specimens 9, 10), specimens with a full cotton duck pad over the head of the pile (specimens 13, 14), and specimens with an annular cotton duck pad or randomly oriented fiber pad over the head of the pile (specimens 15, 16). Figure 11 shows the visible connection damage for specimens 9, 13, and 15 at a rotation of 0.05 rad. Specimen 9 had substantial spalling to the deck and the pile exposing the spiral reinforcing of the pile with some spalling beginning to penetrate the core of the pile. Specimen 13 experienced only moderate spalling of the cover, with no exposure of any reinforcing steel. In addition, the deck concrete was completely intact. Finally, specimen 15 had only pile cracking with no spalling to either the pile or the deck concrete. The advantages of the bearing pad connections, and in particular the annular pad configuration, is clear compared with other connections.

Specimens 9, 10, 13, 14, 15, and 16 were evaluated in greater detail, and Fig. 9 shows their moment-rotation hysteretic response. The total connection moment includes the $P$-$\Delta$ effects caused by the axial loading on the pile. Therefore any observed strength loss in these plots can only be attributed to damage to the pile and connection. Specimens 11 and 12 were omitted from this detailed evaluation because they were intermediary tests used to develop the damage-resistant connection details. Table 2 details the performance of each specimen, including peak shear and moment resistances, the normalized moment capacity (which compares the measured and predicted flexural strengths using the day of test material properties), the

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<th>Table 2. Performance characteristics of six key specimens</th>
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Note: $M_n$ = nominal moment strength; $M_{peak}$ = peak moment; $M_{peak,9}$ = peak moment achieved by specimen 9. 1 ft = 0.305 m; 1 kip = 4.448 kN.
An appropriate approximation in the design is to assume a constant depth of 0.25D (where D is pile diameter) from the compression face of the pile. The design thickness of the bearing pad is then selected using the design rotation and the maximum tolerable compressive strain in the bearing pad (Fig. 12). While testing of annular bearing pad connections is limited, the design recommendations for the bearing pad thickness use other bearing pad test results. Based on geometry (Fig. 12), pad thickness \( t_{pad} \) can then be calculated using Eq. (1).

\[
t_{pad} = 0.25D \frac{\tan \theta_{spall}}{\varepsilon_{p,CR}}
\]

where

\( \theta_{spall} = \) pile end rotation at pile spalling

\( \varepsilon_{p,CR} = \) critical strain limit for the bearing pad that results in spalling of the pile

Test results indicate \( \varepsilon_{p,CR} \) should be limited to 0.4 for a cotton duck pad and 0.6 for a randomly oriented fiber pad. Equation (1) was used to develop Fig. 13, which gives the thickness of the bearing pad as a function of the pile diameter. The controlling pile end rotation was taken as 0.05 rad because experiments show that at this rotation significant loss of resistance is prevented and significant repair is not required.

### Connection steel

The connection between the pile and the deck is achieved with T-headed dowel bars grouted into ducts within the core of the pile. The dowel bars should be placed within the core of the pile with the largest moment arm to make the connection as efficient as possible. With precast, prestressed concrete piles, the connection invariably has smaller moment resistance than the pile because of the elastic rotational stiffness, the connection rotations at initial spalling, substantial spalling (exposure of reinforcing steel), dowel bar fracture, and the deterioration in moment resistance at a connection rotation of 0.08 rad.

### Design procedure

The test results show that debonded dowel bars, an annular cotton duck pad or randomly oriented fiber pad over the head of the pile, and a flexible expansion material wrap around the embedded perimeter of the pile resulted in significant improvements in the seismic performance. A design procedure for this improved connection was developed. The design procedure focuses on the specific seismic detailing of the improved connection and does not address the analysis of the wharf deck, the pile body, or other general matters. The wharf layout, the size, the detailing, and the spacing of the piles must be designed to meet their respective performance objectives by usual methods.

The bearing pad connections have only been tested under a limited range of axial loads and geometric configurations (for example, pile diameter, pile length, and bearing pad thickness). Care should be taken in extrapolating these design recommendations outside the conditions of the experimental testing.

### Bearing pad thickness

An annular cotton duck pad or randomly oriented fiber bearing pad placed over the head of the pile significantly improved the seismic performance over standard embedded dowel bar connections. The bearing pad should cover the head of the pile, with the circular hole having a diameter equal to the diameter of the outside perimeter of the dowel bar group (Fig. 6).

Although the depth of the compression block (or the location of the rotation point) varies depending on the end rotation of the pile, there is minimal fluctuation in the depth of the compression block at end rotations greater than 3%. An appropriate approximation in the design is to assume a constant depth of 0.25D (where D is pile diameter) from the compression face of the pile. The design thickness of the bearing pad is then selected using the design rotation and the maximum tolerable compressive strain in the bearing pad (Fig. 12). While testing of annular bearing pad connections is limited, the design recommendations for the bearing pad thickness use other bearing pad test results. Based on geometry (Fig. 12), pad thickness \( t_{pad} \) can then be calculated using Eq. (1).

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\( \varepsilon_{s,\text{lim}} = 0.08 \), which corresponds to a steel strain limit for ASTM A706 steel bars subjected to multiple cyclic inelastic deformation cycles.\(^{19}\)

\( X_c = \) distance from the rotation point to the extreme tension bar

### Interface shear resistance

The interface between the pile and the deck must be able to transfer the inertial shear forces generated in the deck to the pile. Because the pile and the deck concrete were cast at different times, it is appropriate to use shear friction principles to determine the shear strength of the interface.

The total shear resistance \( V_u \) therefore is taken as the sum of the shear resistances due to the bearing pad shear friction \( V_{bp} \) and concrete core shear friction \( V_{sf} \) (Eq. [3]).

\[ V_u = V_{bp} + V_{sf} \quad (3) \]

Finally, the factored shear resistance \( \phi V_u \) (where \( \phi \) is the strength reduction factor) must be greater than the connection shear demand \( V_u \) (Eq. [4]).

\[ V_u \leq \phi V_u \quad (4) \]

The component of the shear resistance from the concrete...
The shear resistance of the core concrete as calculated by Eq. (5) cannot exceed the lesser of the limits in Eq. (6) and Eq. (7).

\[ V_{sf,\max} \leq K_1 f'_c A_{core} \]  

where

\( f'_c \) = compressive strength of the concrete, taken as the lower strength between the pile and the deck concrete

\( A_{core} \) = area of the concrete core in compression under the applied axial load and flexural moment

\( K_1 \) = fraction of concrete strength available to resist interface shear

The values of \( K_1 \) and \( K_2 \) are dependent on the concrete surfaces in contact where \( K_1 \) equals 0.25 and \( K_2 \) equals 1.5 ksi (10 MPa) for concrete cast against hardened intentionally roughened concrete with a surface amplitude greater than 0.25 in. (6 mm), and \( K_1 \) equals 0.2 and \( K_2 \) equals 0.8 ksi (5.5 MPa) for concrete cast against hardened but not intentionally roughened concrete.

The shear resistance component due to friction between the bearing pad and the concrete of the pile and the deck is calculated by Eq. (8).
\[ V_{bp} = \mu_{bp} C_c \]  

(8)

where

\[ \mu_{bp} = \text{coefficient of friction between the bearing pad and concrete, conservatively taken as 0.2 according to Lehman et al.}^{15} \]

\[ C_c = \text{net compression force on the bearing pad material due to the applied axial load and flexural moment} \]

**Flexible joint sealant wrap thickness**

The flexible joint sealant wrapped around the perimeter of the embedded pile section seals the joint interface and permits large connection rotations without spalling of the pile and deck or the resulting deterioration in resistance. The wrap must be thick enough to prevent the side of the pile from coming in contact with the deck soffit cover concrete so that at the maximum expected rotation of the pile end, the edge of the pile should avoid contact with the deck cover concrete (Fig. 12).

The design equation is generated from the geometric relationship in Eq. (9), and strain in the flexible joint sealant is limited to 0.5.

\[ t_{wrap} = 2(t_{pad} + e) \tan \theta_{fail} \]  

(9)

where

\[ t_{wrap} = \text{thickness of flexible perimeter wrap} \]

\[ e = \text{embedment depth of pile into deck soffit} \]

The flexible expansion material is recommended, and it has a secant stiffness from 10 psi (70 kPa) to 25 psi (170 kPa) at 25% compressive strain.

**Conclusion**

This research evaluated embedded dowel, moment resisting prestressed concrete pile-to-wharf connections that are commonly used in seismic design of marginal wharves and developed an improved connection that significantly delays the damage and deterioration of moment resistance of the pile, wharf deck, and connection until much larger seismic deformations.

When combined with prior research, this study shows that current connections sustain significant damage and deterioration of resistance starting at relatively small seismic deformations:

- The connections have early onset of pile and deck spalling at relatively small inelastic deformations. Significant spalling to the deck soffit cover concrete and severe spalling into the core of the pile occur with increasing deformation. This spalling causes dramatic loss in resistance even at moderate deformation. The spalling damage and resulting deterioration of resistance that results from this damage increases dramatically with increasing compressive axial load.
  - The spalling damage requires significant repair even after moderate seismic deformation.
  - Extended pile connections that are used when the pile is driven below the bottom of the wharf deck exhibit different behavior, which is closer to that of cast-in-place reinforced concrete moment connections.

As a consequence, research was completed to develop an improved connection for the prestressed concrete piles, and the following conclusions were made:

- The addition of a bearing pad over the head of the pile reduces the initial elastic stiffness of the connection compared with current connection designs. It significantly delays (by approximately 0.04 rad connection rotation compared with standard connections) pile and connection damage, yet achieves the same maximum resistance and inelastic deformation capacity as the current connection. Post-peak strength degradation is reduced.
  - The addition of a flexible expansion material joint sealant wrapped around the perimeter of the embedded pile segment effectively eliminates spalling of the wharf deck.
  - Connections employing a full cotton duck bearing pad over the head of the pile displaced in near rigid body rotation with on average 90% of the pile tip displacement resulting from end rotation of the pile at the interface. They also experienced axial deformations due to the low compressive stiffness of the pad material.
  - Annular bearing pad connections also experienced mostly rigid body rotation, but to a slightly lesser degree than the full pad connections, with approximately 80% of the measured displacement resulting from end rotation of the pile. These specimens have a higher elastic lateral stiffness than full bearing pad connections, and they maintained similar axial stiffness noted with current connection design due to the concrete plug in the center of the annular bearing pad.
  - The normalized moment resistance of the connections subjected to the same axial load was similar and symmetric.
  - Higher axial load resulted in larger moment resistance, greater deterioration of resistance, and greater loss of
effective resistance due to $P$-$\Delta$ effects.

- Spalling of the pile cover concrete coincides with bearing pad strains of 0.4 and 0.6 for cotton duck and randomly oriented fiber materials, respectively.

An improved moment resisting connection was proposed, and significant improvements in performance were achieved:

- The connection employs a cotton duck or randomly oriented fiber bearing pad over the head of the pile. The pad is an octagonal-shaped annular ring, which covers all of the pile concrete outside the perimeter of the dowel bars. The annular ring permits easier placement of the grouted dowel bars, and it improves the stiffness and deformation of the connection. The improved connection also includes a flexible expansion material wrap around the perimeter of the embedded section of the pile, and the dowel bars are deliberately debonded over a 15 in. (380 mm) length centered at the pile head.

- These changes result in significant delays and reductions in spalling of the pile and wharf deck, identical maximum moment resistance of the connection, large reductions in the deterioration of resistance of the connection, large reductions in repairs required after seismic deformation with no repair required for connection rotations in the order of 0.05 rad, and slightly larger total rotational capacity than the current connection design.

- A design procedure for the improved connection was developed and presented.

Several areas warrant further study:

- The annular bearing pad connections were only tested under one axial load (10% gross axial capacity of the pile); thus it would be prudent to examine the behavior of the connection under higher (20% axial capacity) and lower (5% axial capacity) axial loads.

- Full bearing pad connections should be tested using different pad thicknesses and materials.

- For both annular and full bearing pad connections shorter pile lengths should be tested to investigate the behavior of the connections under increased shear demands.

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**References**


Notation

\( A_{\text{core}} \) = area of the concrete core in compression under the applied axial load and flexural moment

\( A_g \) = gross cross-sectional area of the pile

\( A_{sf} \) = area of reinforcing steel crossing perpendicular to shear plane

\( C_c \) = net compression force on the bearing pad material due to the applied axial load and flexural moment

\( D \) = pile diameter

\( e \) = embedment depth of pile into deck soffit

\( f_c \) = specified compressive strength of concrete

\( f_y \) = yield strength of reinforcing steel resisting shear sliding

\( K_1 \) = fraction of concrete strength available to resist interface shear

\( K_2 \) = limiting interface shear resistance

\( l_{db} \) = design debonded length

\( M_n \) = nominal moment strength

\( M_{\text{peak}} \) = peak moment

\( M_{\text{peak},9} \) = peak moment achieved by specimen 9

\( P \) = axial load

\( t_{\text{pad}} \) = thickness of annular bearing pad

\( t_{\text{wrap}} \) = thickness of flexible perimeter wrap

\( V_{lp} \) = shear resistance due to bearing pad shear friction

\( V_t \) = total shear resistance

\( V_{sf} \) = shear resistance of connection through friction

\( V_{\text{sf, max}} \) = maximum shear resistance due to shear friction

\( V_u \) = shear demand

\( X_c \) = distance from rotation point to extreme tension bar

\( \Delta \) = deflection
\( \varepsilon_{\text{p,CR}} \) = critical strain limit for bearing pad that results in spalling of pile

\( \varepsilon_s,\text{lim} \) = steel strain limit of 0.08, which corresponds to a steel strain limit for ASTM A706 steel bars subjected to multiple cyclic inelastic deformation cycles

\( \varepsilon_s \) = average strain in debonded steel bar

\( \theta \) = rotation

\( \theta_{\text{fail}} \) = limiting connection rotation at connection failure as determined by dowel bar fracture, per performance objectives (taken as 0.085 rad)

\( \theta_{\text{spall}} \) = pile end rotation a pile spalling

\( \mu \) = coefficient of friction

\( \mu_{\text{bp}} \) = coefficient of friction due to bearing pad, conservatively taken as 0.2

\( \phi \) = strength reduction factor
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Abstract

Pile-supported marginal wharves are critical components of the nation’s infrastructure. Pile-wharf connections have sustained damage in earthquakes. Current connection design uses headed dowel bars anchored in grouted ducts in vertical piles and cast integrally with the deck. Typically, the pile is embedded 3 in. (76 mm) into the deck. Although prior research found these dowel connections to have adequate resistance, they sustained damage and strength deterioration in both pile and deck even at moderate deformations. The necessary postearthquake repair is difficult and economically disruptive. Research was conducted to develop a new pile-wharf connection less vulnerable to seismic damage. Several structural concepts were evaluated, including intentionally debonding the dowel bars, employing a bearing pad between the head of the pile and the deck, and adding a flexible joint around the embedded portion of the pile. A prototype connection was developed and studied experimentally to investigate the effect of axial load, bearing-pad material, and bearing-pad configuration. The proposed connection reduces damage and strength deterioration relative to current connections, delaying pile and deck spalling well beyond the expected seismic deformation demand level. The experimental observations and measures were used to develop performance-based design expressions and a design procedure for this new connection.

Keywords

Connection, deck, pile, seismic, wharf.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute’s peer-review process.

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