A fully reversed cyclic test was conducted at 1/4 scale on a typical hollow rectangular reinforced concrete pier designed to support the skyway structures of the new East Bay Spans of the San Francisco Oakland Bay Bridge. The test unit was loaded quasi-statically in double bending to simulate the lateral force-deflection response of a generalized prototype pier under seismic forces in the longitudinal direction of the bridge. The test unit had a cross section scaled to 1/4 of pier E15E with a clear height scaled to 1/4 of pier E9E. It was intended to model the general flexural response of the East Bay Bridge piers in double bending, and provide a benchmark with which to evaluate the accuracy of predictions made with the three-dimensional, non-linear finite element model developed by ANATECH Corp.

These predictions are compared with the test results and the sufficiency of the predictions' accuracy is evaluated. The test is described in detail. Results from the test are also reported separately from the finite element model predictions. Existing design equations are evaluated for their efficiency and their ability to reflect the real phenomena observed in the test unit behavior.
Structural Testing of the San Francisco-Oakland Bay Bridge
East Span Skyway Structures: Longitudinal Pier Test

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INTRODUCTION
The structural components of the new San Francisco-Oakland Bay Bridge East Bay Spans that are expected to deform inelastically during a major earthquake event are currently being tested in the Charles Lee Powell Structural Laboratories at the University of California, San Diego. A series of quasi-static, fully reversed cyclic structural tests was designed to demonstrate the ability of structural components to withstand inelastic deformations in excess of those expected from a 1500 year return period earthquake on either the San Andreas or the Hayward Faults. The first test in this series modeled the response of a reinforced concrete Skyway Pier under seismic loads in the bridge longitudinal direction [Hines ‘00]. This paper introduces the test setup for this first test and discusses some of the test results that are relevant to the design of a typical skyway pier, pictured schematically in Figure 1(a).

Figure 1. (a) East Bay Skyway pier detail: pre-cast box girder, hollow rectangular pier with confined boundary elements, battered cast in steel shell piles. (b) Hybrid prototype pier elevation, section and assumed moment distribution.
TEST UNIT DESIGN
The test unit was designed to form plastic hinges under cyclic loading in the longitudinal direction of the bridge. Since a major objective of the test was to evaluate the accuracy of the predictive analytical model and not necessarily to evaluate the behavior of a specific skyway pier, it was acceptable to create a hybrid prototype pier that could be tested at a reasonable scale and was guaranteed to form plastic hinges as opposed to failing in shear. Combining the 5.5 m section depth of Pier E15E with the 28.1 m clear height of Pier E9E, a ¼ scale test unit was created that fit within laboratory space and equipment constraints. This hybrid prototype pier, pictured in Figure 1 (b), had an aspect ratio of \( \frac{M}{VD} = 2.55 \) in double bending. The ¼ scale test unit cross section and an isometric view of the test setup are pictured in Figures 2(a) and 2(b).

![Diagram](image)

Figure 2. (a) Test Unit cross section geometry and reinforcement. (b) Test Setup, isometric view.

The test unit was loaded quasi-statically according to a standard, incrementally increasing, fully-reversed cyclic loading pattern, with constant axial load. The lateral load was applied through a load frame by two 2000 kN (450 kip) MTS actuators positioned at column midheight in order to deform the test unit in double bending. In order to ensure zero rotation of the load stub, each vertical actuator supplied only 1780 kN (400 kips) axial load, leaving a 220 kN (50 kip) reserve in each actuator to help stabilize the load stub. The moment capacity of this reserve was roughly 10% of the expected column ultimate moment. Additional axial load was provided by two 890 kN (200 kip) hollow core jacks, and the combined 750 kN (170 kip) weight of the load stub and load frame. The total axial load of 6090 kN (1370 kips) corresponded to an axial load ratio of \( \frac{P}{f'c'A_g} = 0.10 \) in the prototype structure, assuming \( f'c = 35 \text{ MPa} \) (5 ksi) in the test unit and \( f'c = 30 \text{ MPa} \) (4.35 ksi) in the prototype.

TEST PREDICTIONS
The force-deflection behavior of the test unit was predicted based both on a three-dimensional, non-linear finite element model developed in cooperation with Anatech Corp. and on a moment-
curvature analysis with an assumed plastic hinge length. Figure 3(a) shows the hysteretic finite element prediction along with the simpler moment-curvature prediction. Both models predicted a similar force-deflection envelope, with failure resulting from fracture of the longitudinal bars at around $\mu_A = 8$. Figure 3(b) shows the moment-curvature prediction envelope in detail, with predicted displacement ductility levels as well as performance levels based on specified strain limit states [T.Y. Lin Intl. '99]. In the figure, “FEE” stands for Functional Evaluation Earthquake, “SEE” stands for Safety Evaluation Earthquake, and “Design” refers to the maximum strain levels observed from non-linear time-history analyses of the bridge. Figure 3(b) shows the predicted failure level exceeding $\mu_A = 10$, whereas Figure 3(a) shows failure predicted closer to $\mu_A = 8$. This discrepancy resulted from the fact that the theoretical displacement at $\mu_A = 1$ underestimated the experimental value by 24%, most likely owing to the cover concrete being weaker and the column therefore more flexible than assumed. The displacement ductility levels in Figure 3(b) are based on the theoretical value of $\mu_A = 1$, whereas the levels in Figure 3(a) correspond to the experimental value of $\mu_A = 1$, so that the full hysteretic prediction could be compared to the test results.

![Figure 3](image)

**Figure 3.** (a) Fully-reversed, cyclic test prediction based on a three-dimensional, non-linear finite element analysis. (b) Force-deflection envelope and specified performance levels based on a moment-curvature analysis and assumed plastic hinge length.

**TEST RESULTS**

The test unit failed in flexure by buckling and fracture of the longitudinal reinforcing bars after reaching 2.04 times the maximum design displacement level corresponding to $\epsilon_c = 0.01$ in Figure 3(b). This level of ductile deformation capacity suggested that the test unit design was sufficiently conservative with respect to both confinement and shear capacity. Hence, it can be inferred that the skyway piers themselves are also designed conservatively.

Figure 4 shows that the deformations in the push direction exceeded those in the pull direction. This imbalance resulted from the fact that after $\mu_A = 3$, the main displacement transducer, responsible for giving feedback to the actuators, began to slip. Undetected until after the test, the
transducer subsequently developed a ratcheting behavior, where it allowed the test unit to deform further than desired in the push direction without fully retracting in the pull direction, resulting in the asymmetric loading history visible in Figure 4. The real displacement of the test unit was calculated from the actuator displacement and the load frame stiffness which calibrated experimentally at displacement levels lower than $\mu_\Delta = 3$. The real displacement ductility levels corresponding to first cycle peak displacement levels are listed in Table 1.

![Drift-Displacement Graph](image)

Figure 4. Test hysteretic behavior with the force-deflection prediction based on a moment-curvature analysis of the column section.

<table>
<thead>
<tr>
<th>Displacement [mm]</th>
<th>Load [kN]</th>
<th>Nominal $\mu_\Delta$</th>
<th>Real $\mu_\Delta$</th>
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<td>-8</td>
<td>-6.49</td>
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</table>
Test Observations
Test observations are described with reference to Figure 5(a). During both the push and pull excursions of the first cycle at $\mu_A = 1$, new diagonal flexure-shear cracks connected existing flexural cracks in the tension boundary element, and some new vertical cracks formed between the boundary elements. On the pull excursion, vertical cracks were found on the bottom corner of the tension boundary element, showing evidence of incipient spalling during the previous push excursion. This incipient spalling, shown in Figure 5(b) was not noticed during the push excursion, because incipient spalling was expected closer to $\mu_A = 3$ and was therefore not the focus of observation. By the third cycle at $\mu_A = 1$, incipient spalling had occurred on the extreme compression fibers of every boundary element.

Figure 5. (a) Column cross section with orientation and crack marking notation. (b) Incipient spalling on the first excursion to $\mu_A = 1$. (c) Spalling at $\mu_A = 2$.

During the first cycle at $\mu_A = 2$, spalling of the cover concrete had clearly spread from 152 mm (6 in.) to 406 mm (16 in.) above the footing on all boundary elements, as shown in Figure 5(c). By the third cycle at $\mu_A = 2$, enough concrete had spalled off to expose some cover concrete reinforcing bars (see Figure 5(a)).

During the first cycle at $\mu_A = 6$. Flexure-shear cracks penetrated through the entire middle region of the column, beginning to slope at the tension end of the middle region and reaching the compression end with a maximum slope of 35° from the vertical. By the third cycle, longitudinal bars in the boundary element core were observed to begin to buckle over two spiral spacings and concrete had spalled well into the middle region.

When the column had reached the peak at $\mu_A = 8$, incipient spalling was apparent on the wall connecting the compression boundary elements. There was, however, no sign of significant spalling or buckling of the wall longitudinal bars.
Flexural and Shear Displacements

Figure 6 shows the total column displacement broken down into its flexural and shear components, calculated from measured curvature instrumentation and panel deformation readings. The difference between the magnitudes of the flexural and shear displacements shown further demonstrates the tendency of the column to have behaved primarily in flexure. The maximum shear displacement at $\mu_A = 6$ reached close to 20% of the total column deformation. This shear displacement was recorded primarily in the plastic hinge regions of the column over the first 1524 mm (60") above the base and below the load stub. As mentioned previously, vertical cracks were observed to have formed between the boundary elements in these regions. Vertical slippage along such cracks may have in some part contributed to the measured shear displacement. The maximum contribution of shear displacement from other parts of the column totaled only 8% of the total shear displacement.

Equivalent Plastic Hinge Length

The Seismic Design Criteria [S.Y. Lin. Intl. '99] for the new East Bay Bridge specify strain limit states known to be conservative with regard to the flexural deformation of the skyway piers. The plastic hinge length specified by the same criteria according to ATC-32 [ATC '96] was not, however, calibrated based on columns with similar geometry and reinforcement. There has been much discussion among all parties involved in the Skyway design as to whether the equation

$$L_p = 0.08L + 9d_{bl}$$

(1)

where $L$ is the column shear span and $d_{bl}$ is the longitudinal bar diameter, is suitable for the Skyway Piers. Previous tests conducted on structural walls with highly-confined boundary elements [Hines et al. '99] demonstrated that the experimental plastic hinge lengths of such walls exceeded the theoretical values calculated using Equation (1). This increase in the experimental
plastic hinge length for deeper members is the result of a tension shift effect in the plastic hinge zone whose magnitude varies according to the member depth.

The experimental equivalent plastic hinge length for a typical skyway pier in the longitudinal direction was calculated based on the method proposed by Priestley [Priestley et al. '96] from experimentally derived flexural displacements and curvatures and an assumed length of the plastic region, \( L_{pr} \). Priestley assumed \( L_{pr} = 0.3L \), where \( L \) is the column shear span. For this test, \( L_{pr} = 0.325L \) was determined experimentally to be by averaging the y-intercepts in the push and pull directions of the linear curvature distributions shown in Figure 7(a). These linear distributions were calculated from regression analyses of the first four experimental curvatures above the footing at \( \mu = 6 \). Figure 7(b) shows the experimental values for \( L_p \) over the range of displacement ductilities 3-6.

![Curvature profiles](image)

Figure 7. (a) Curvature profiles at the base of the column. (b) Experimental equivalent plastic hinge length during the final stages of testing.

The consistency of these values over several cycles at high levels of displacement ductility, supports the addition of a tension shift component to Equation (1), such that

\[
L_p = 0.08L + \alpha D + 9d_{bl}
\]  

(2)

where \( D \) is the total section depth and \( \alpha = 0.1 \) is defined according to the Longitudinal Pier Test results.
CONCLUSIONS
The ¼ scale model of a typical skyway pier for the new East Bay Spans of the San Francisco-Oakland Bay Bridge exceeded the maximum design level deformation demand in the bridge longitudinal direction by more than a factor of 2. The test unit began to fail in flexure by buckling and fracture of the longitudinal reinforcing steel at the column base at a displacement ductility in excess of $\mu_A = 9$. This failure is considered conservative due to the nature of the loading history, which deliberately cycled the test unit at high levels of displacement a greater number of times that would be expected in an earthquake. Spalling initiated in the extreme cover concrete as early as $\mu_A = 1$, and the elastic stiffness of the test unit proved to be lower than the predicted stiffness. It is proposed to solve this problem of premature spalling by providing a gap between the cover concrete and the pile cap or bent cap, similar to the measures taken to protect flared columns [Sanchez ‘97]. Shear deformations were observed to account for roughly 20% of the total column deformation. The experimental equivalent plastic hinge length exceeded the value specified by the bridge Seismic Design Criteria [T.Y. Lin Int'l.] by 29%, probably due to tension shift effects proportional to the depth of the wall.

REFERENCES

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