LONGITUDINAL TESTING OF A PRECAST POST-TENSIONED BRIDGE SYSTEM

By Kelly Burnell[1], José I. Restrepo[2], and Sami H. Megally[3]

Abstract

This paper discusses the main findings of a test designed to examine the seismic behavior of a precast post-tensioned segmental bridge with a cast-in-place, hollow, rectangular column. The half-scale specimen modeled a bridge from midspan to midspan and down to midheight of the column. The test was completed in two stages, the first involved a superstructure prestressing design approach to avoid joint openings throughout, and the second of which involved removing some of the tendons to enable opening of the joints in the superstructure and to impose on the joints nearest the column a more severe loading condition. The primary objectives of the test were to investigate the response of the column-superstructure interaction, possible opening of the superstructure joints, plastic hinge formation in the column, and the anticipated system failure mechanism.

Introduction

The benefits of precast segmental bridges over conventional cast-in-place (CIP) construction are well known. Better quality control, limited impact on surrounding areas, and economic advantages make it a popular and versatile bridge building method. However, the spread of this technology into areas of high seismic activity has been hampered due to a lack of experimental research and seismic specific design guidelines. In order to address these deficiencies a large-scale segmental bridge testing program was initiated at the University of California, San Diego with funding from the California Department of Transportation (Caltrans). The two prior phases of the program looked specifically at the performance of segment-to-segment joints under seismic loading

[1] Graduate Student Researcher, Department of Structural Engineering. University of California, San Diego. CA 92093

[2] Associate Professor, Department of Structural Engineering. University of California, San Diego. CA 92093

[3] Structural Engineer, W. Koo & Associates, Inc, Consulting Engineers. Orange, CA 92868

[Megally, et al. 2002]. This paper summarizes the findings of the third phase of the program: a superstructure-pier system test.

The half-scale bridge was composed of a CIP column and a full box girder superstructure span. The column was similar to the columns recently used in the Benicia-Martinez and Carquinez bridges of the Bay Area. While providing similar moment capacities as solid columns of comparable dimension, the greatly reduced volume allows for savings in cost, particularly of the foundation. The superstructure was designed and constructed using the balanced-cantilever method with internal tendons, most of which were bonded. The balanced cantilever method can be used for spans typically up to 500' (152m) and is useful for construction in areas where access from below is limited. Since no joint opening was expected in the first stage of testing, some tendons were left unbonded to facilitate their removal as part of the testing sequence for the second stage.

Reversed cyclic seismic motion was applied to the bridge structure in the longitudinal direction. The test consisted of two stages. The first stage sought to force all damage into the column as is typically done in seismic design. For that purpose the amount of post-tensioning was designed according to the moment capacity of the column so that the superstructure would remain elastic and uncracked under the ending moment demands controlled by the combination of gravity load and column bending moment overstrength in the plastic hinge immediately below the superstructure. In the second stage of the test it was attempted to allow opening of the joints nearest the column. To achieve this goal a quarter of the top flange tendons were removed and additional vertical load was applied to simulate a vertical ground motion equivalent to 1.75g.

The objectives of the test were to investigate the response of the column-superstructure interaction, possible opening of the superstructure joints, plastic hinge formation in the column, and the anticipated system failure mechanism. The separate stages of the testing allowed both a proof of the existing design philosophy of limiting damage to the column as well as an initial look at the bridge's performance when inelastic motion is allowed in the superstructure as well.

Bridge System Test Unit

The prototype structure modeled in the test is a five-span segmental bridge designed according to Caltrans seismic design criteria [Caltrans, 1999] with three interior 100' (30.5m) spans and exterior spans of 75' (22.9m) (Figure 1). The column height is

50’ (15.2m). The bridge was designed to have a segmental superstructure constructed using the balanced cantilever method. The superstructure box girder shape was selected to be similar to the ASBI standard section for short balanced cantilever span lengths [ASBI, 2000]. The test specimen models from mid-height of the column and to the midspan on each side of the column.

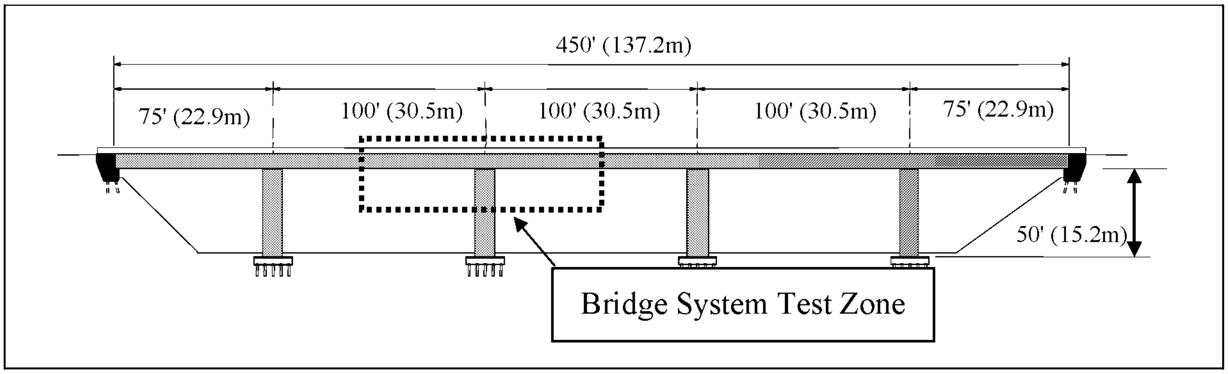

Figure 1: Prototype Bridge Structure

Figure 2 shows general dimensions of the test specimen constructed at the Powell Research Laboratories at the University of California, San Diego. The specimen was built at half-scale of the prototype. Gravity loads were applied to the test structure using a wiffle tree to distribute the loading across the deck. Vertical actuators were used to apply the proper boundary conditions ‘midspan’ moments to the ends of the span. Compliance of the vertical actuators with expected lengthening of the column was assured by monitoring the column growth and by incorporating this parameter in the control feedback loop. Figure 3 shows a photo of the test setup prior to testing.

The column was prismatic and had octagonal boundary elements on each of the corners. The longitudinal reinforcement in the column remained unchanged along the column height; nonetheless, the spacing of the transverse reinforcement and corner spiral spacing in the region away from the plastic hinge was enlarged. Cross section details of the plastic hinge zone of the column are shown in Figure 4. Straight bars were used as shear reinforcement to facilitate construction following the successful performance of similar details in the testing of the San Francisco-Oakland Bay Bridge (Seible, et al, 2004).

The superstructure was made up of ten individual segments of two basic cross sections (see Figure 5). Nearest the column the segments had a thicker bottom flange than the segments further from the column. The tendon layout and stressing for the test specimen was selected to most closely match the prototype tendon profiles at the critical joints nearest to the column. 0.5” (13 mm) diameter strands were used to more accurately match the bonding area of the prototype structure’s tendons. Tables 1 and 2 summarize the mechanical properties of the materials used in the specimen.

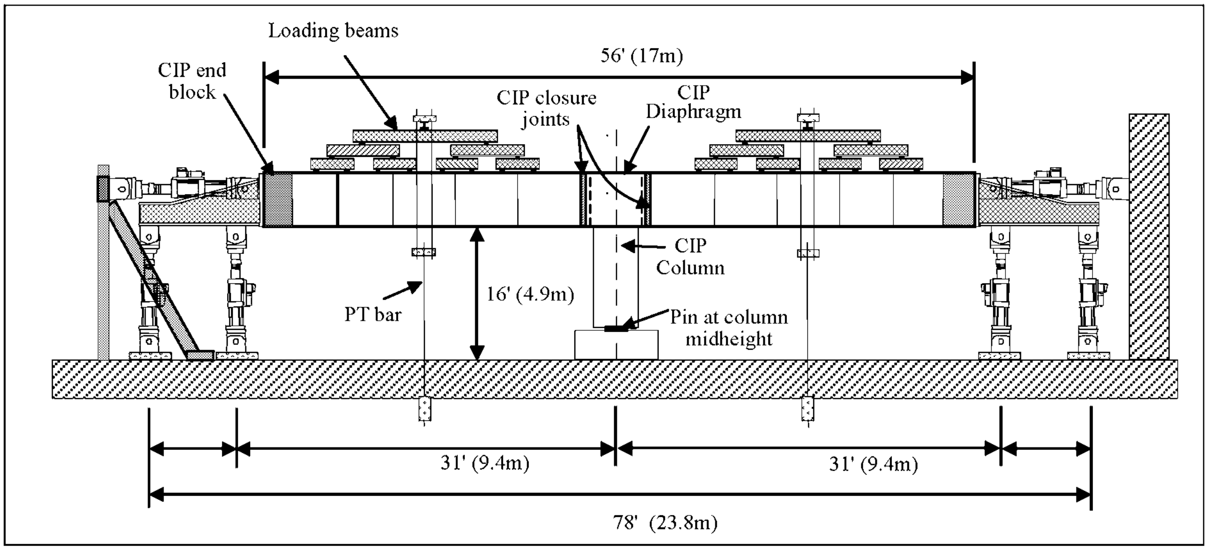

Figure 2: Test Specimen Elevation

Figure 3: Test Specimen Photo

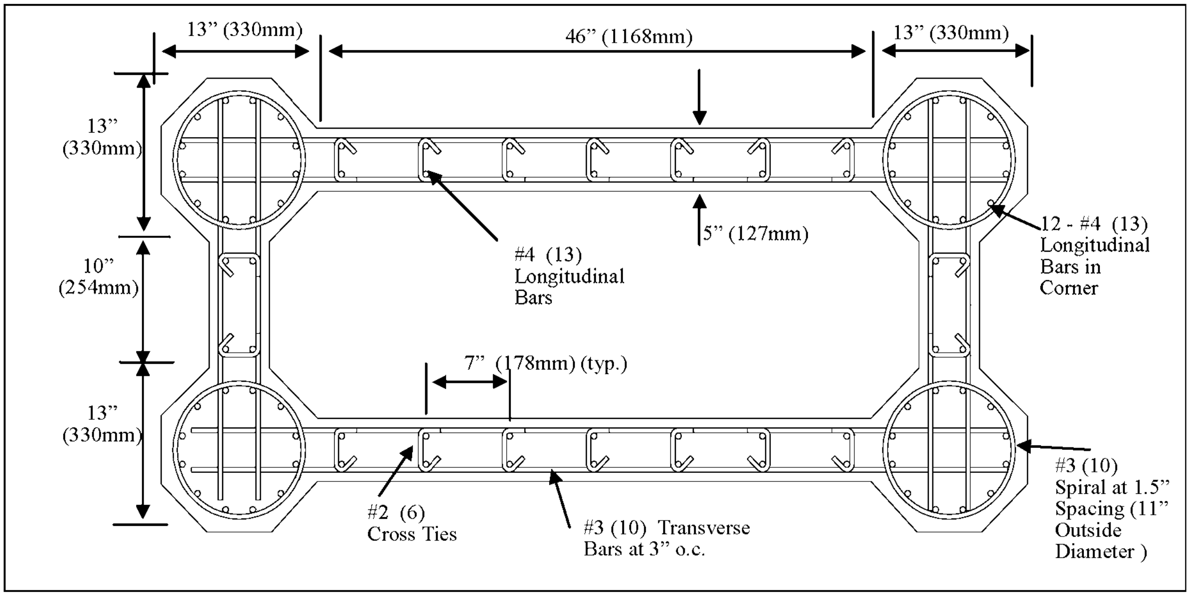

Figure 4: Column Cross-Section Detail

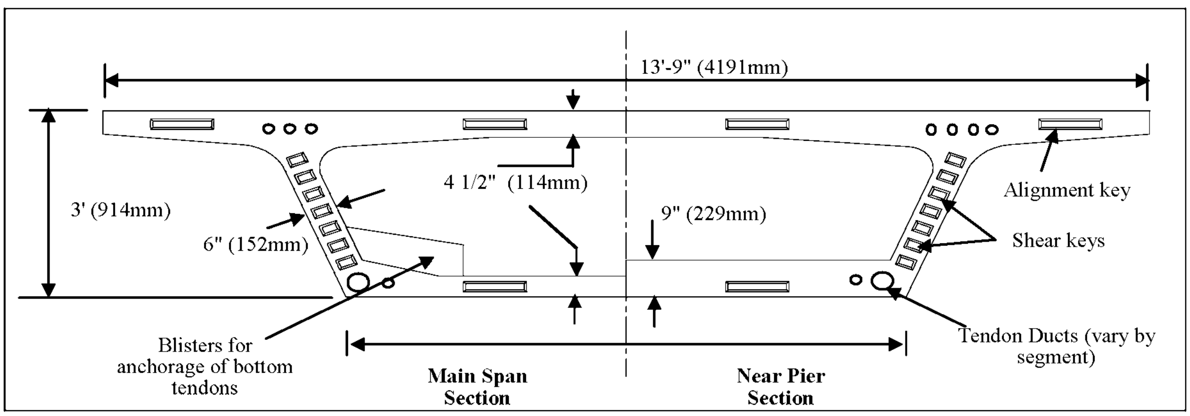

Figure 5: Superstructure Box Cross-Section Detail

Table 1: Steel Material Properties*

	f_y yield strength (ksi (Mpa))	f_{su} ultimate tensile strength (ksi (Mpa))
#3 Column Transverse Bars	66 (455)	102 (703)
#4 Column Longitudinal Bars	65 (448)	105 (724)
#4 Pier Cap Segment Bars	67 (462)	88 (607)

*** average of 3 bar tests**

Table 2: Concrete Material Properties*

	Compressive Strength (ksi (Mpa))			
	7-Day	14-Day	28-Day	Test Day
Column Top	7.3 (50)	-	9.3 64)	11.6 (80)
Pier Cap Segment	7.7 (53)	9.0 (62)	10.0 (69)	11.6 (80)
Pier Gap	7.8 (54)	9.2 (63)	10.3 (71)	12.5 (86)
1st pair of Segments	-	-	-	9.1 (63) (avg)

***average of 3 cylinder tests**

The superstructure was instrumented with both concrete surface gauges and linear displacement transducers crossing all the joints. The critical joints nearest the column had instrumentation both on top and bottom and on the west, center, and eastern side of the flanges. Vertical displacements and rotations were also monitored along the superstructure.

The column had over one hundred and fifty strain gages to monitor the strain development on the longitudinal, spiral, and transverse bars. The gauges were concentrated within the plastic hinge zone and spaced more sparsely in the bottom of the column. Linear displacement devices were placed on two corners to obtain measurements converted to curvature information. Linear potentiometers were also used to obtain shear deformations in one web of the column.

A unidirectional loading protocol in the longitudinal direction of the bridge was used. Two cycles at increasing ductility levels were used in order to allow for further testing of the system without considerable strength deterioration in the column. At the commencement of testing stage 2 a cycle was repeated at system displacement ductility 4. The loading protocol for stages 1 and 2 of the test are shown in Figure 6. During stage 2 of the test additional vertical load was applied to the superstructure equivalent to a vertical acceleration of 1.75g.

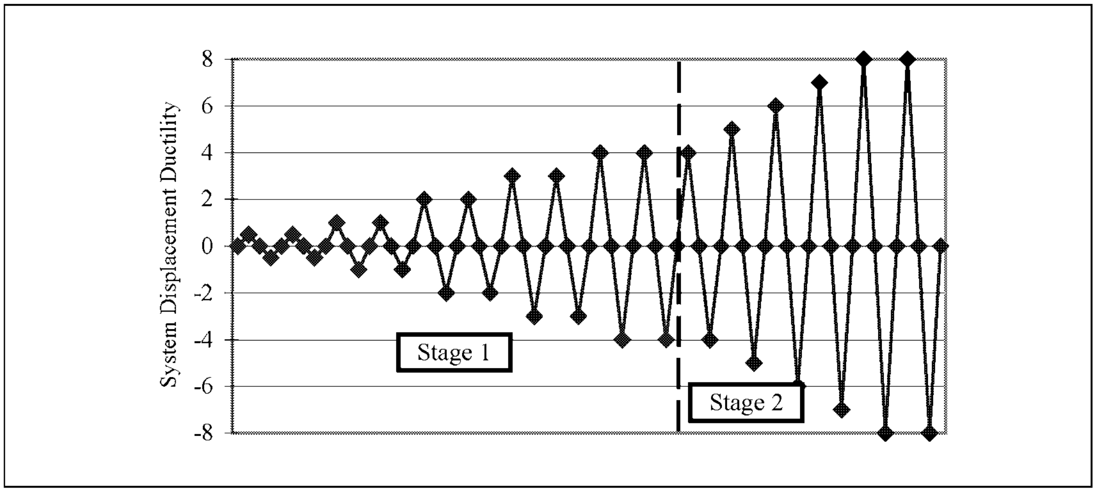

Figure 6: Loading Protocol

Analytical Considerations and Test Predictions

In order to predict the system's motion and design the test set-up accordingly, a preliminary analysis of the column displacement characteristics was performed. A moment curvature analysis was done using ANDRIANNA [ANATECH, 1999], and the results were translated to a force-displacement prediction for the column (see Figure 7). The plastic hinge length (L_p) of the column was predicted by Equation 1 according to work done on similarly shaped columns [Dazio, et al, 2002].

$$L_p = 0.08L + 0.3L_w + 9d_b \qquad \text{(Eq.1)}$$

Where L is the column shear span, L_w is the column width in the direction of loading and d_b is the longitudinal bar diameter.

The loading of the system was done in displacement control based on the system displacement ductility (see Figure 6). Ductility levels of the system were derived from the theoretical yield displacement (Δ_y) of the system calculated according to Equation 2 [Priestley et al, 1996].

$$\Delta_y = \Phi_y * L^2/3 \qquad \text{(Eq. 2)}$$

Where Φ_y is the reference yield curvature.

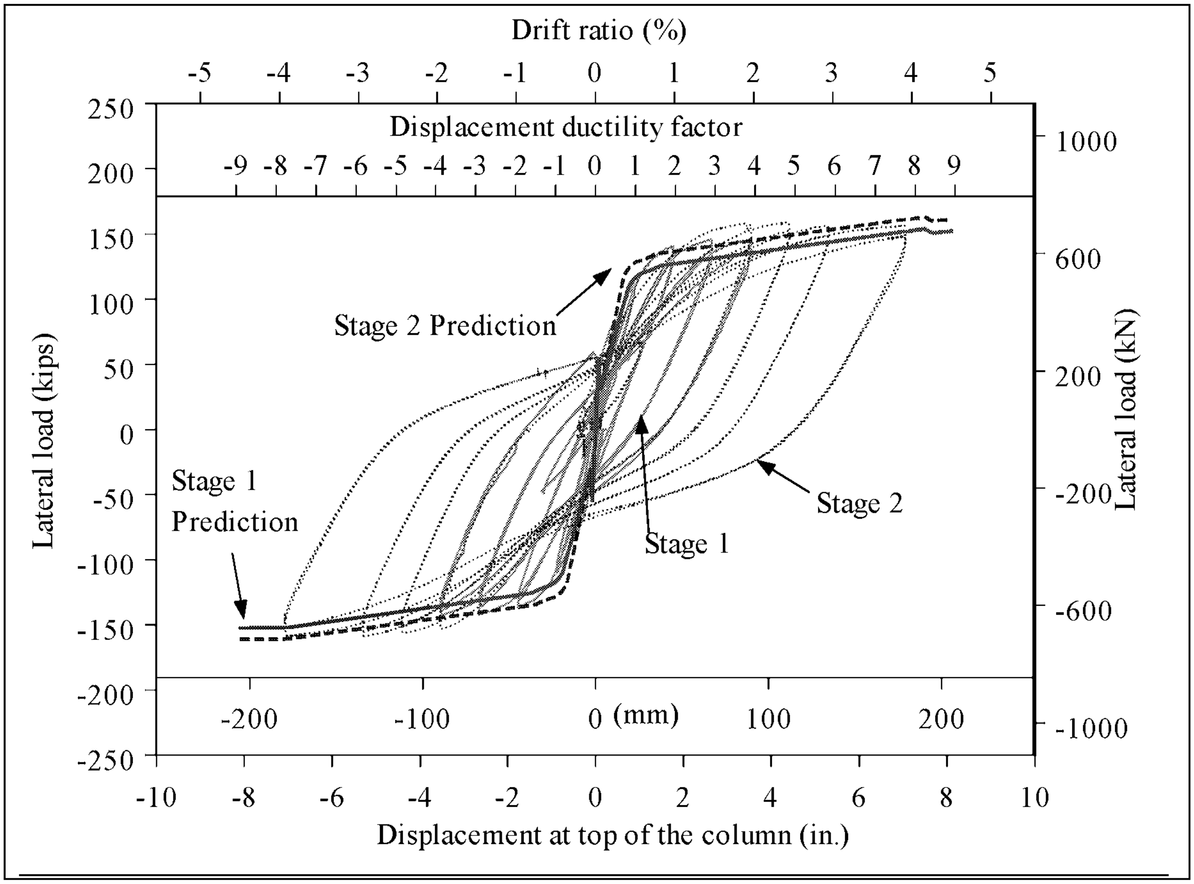

Figure 7: Force Displacement Diagram for Stages 1 and 2 with Monotonic Prediction

The conditions under which the joints open were an important consideration of this testing program. Stage 1 of the test program attempted to not allow any opening of the joints. The summary of the predicted joint opening forces during stage 1 is shown in Figure 8. The moment required to cause decompression as well as tensile cracking (assumed to be 6√f'c in ksi (0.5√f'c in Mpa)) of the concrete is compared to the moments the bridge structure was expected to undergo due to the dead and seismic loads applied during stage 1. It can be seen that the expected applied moment was below the decompression moment at all joints. The second stage of the test both lowered the expected opening moments by removal of tendons, and increased the applied forces due to additional vertical loads. The summary of superstructure moments for phase 2 in Figure 9 shows that opening of the joints could be expected in the top of the joints nearest the column. A summary of the number of 0.5" (13mm) superstructure tendons crossing critical joints nearest the pier segment as well as their eccentricities is given in Table 3.

Table 3: Tendons Crossing Innermost Joints

	Number of Tendons in Stage 1	Number of Tendons in Stage 2	Eccentricity of tendons in segments nearest column
Top Tendons	40	30	11.4” (290mm)
Bottom Tendons	18	18	17.7” (450mm)

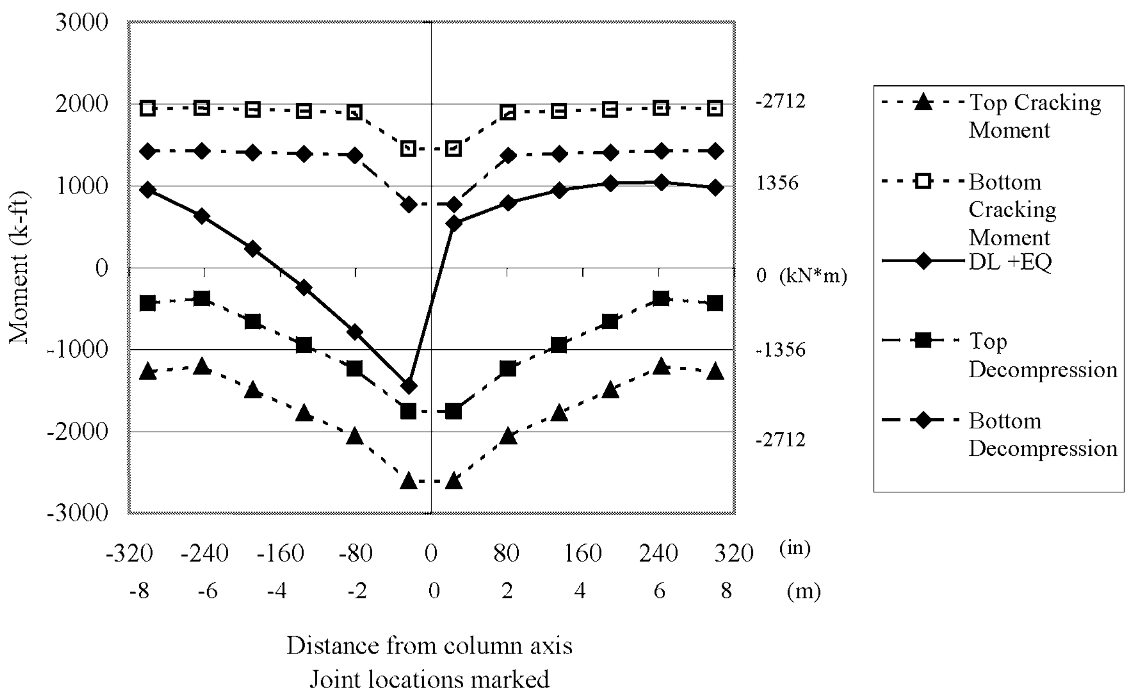

Figure 8: Summary of Superstructure Moments for Testing Stage I

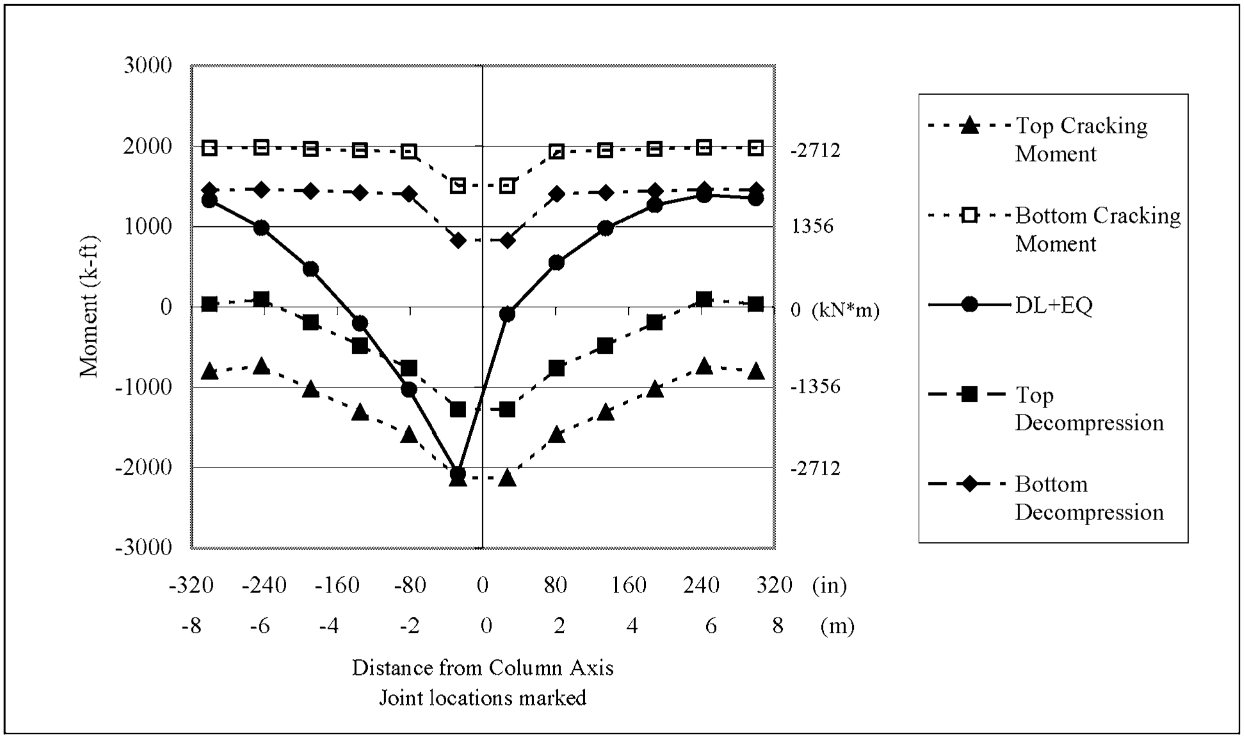

Figure 9: Summary of Superstructure Moments for Testing Stage 2

Test Observations

Figure 7 shows the force displacement diagram in the longitudinal direction of the system test. The graph shows the lateral force versus the drift ratio, displacement ductilities, and displacement. The overall performance is typical for a well-confined, reinforced concrete column showing high-energy dissipation capacity, high displacement ductility, and stable hysteretic response. At each ductility level the second cycle showed a similar response to the first. During the second phase of testing the moment capacity of the column increased due to the higher axial load applied to the column. The monotonic prediction for the stage 1 axial load is also shown on the graph. The prediction underestimated the lateral force on the column by about 7%, likely due to variations in material properties and construction irregularities. A plastic hinge developed at the top of the column as expected, and the capacity was maintained up a system displacement ductility of 8.

The curvature profiles normalized to the Column Depth (L_W) obtained from the systems test for all ductility levels are illustrated in Figure 10. The data shows a comparable curvature profile to a previous test of this type of column [Burnell, et al, 2003] as well tests of a similar column to be used in the San Francisco-Oakland Bay Bridge [Dazio, et al, 2002].

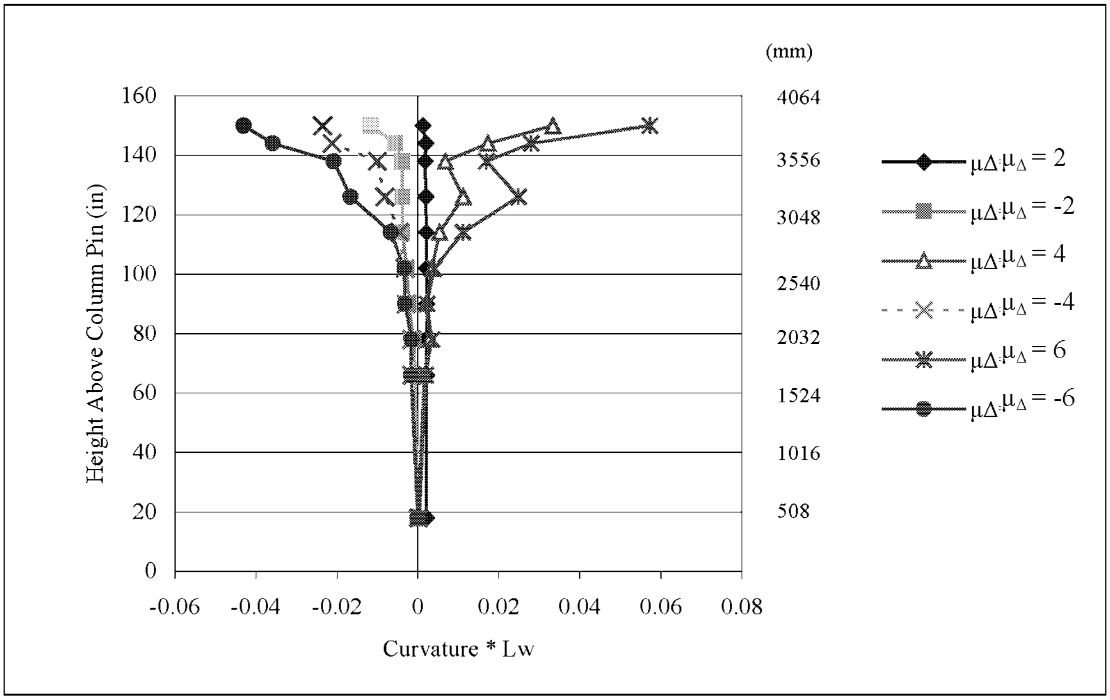

Figure 10: Systems Test Column Curvature Profiles

During stage 1 of the test there was no opening of the superstructure joints. In stage 2 of the test when μ_Δ=4 was reached, a very clear crack openings formed exactly at the closure joints connecting the first pair of segments to the pier segment. The crack width at μ_Δ=4 was approximately 0.5mm and it closed upon unloading. The plastic hinging of the column limited the amount of seismic moment transferred to the joints and limited the degree of joint opening. At μ_Δ=5 the crack width was approximately 0.7 mm and increased very little as additional ductility levels were reached, because the demand was limited by the moment capacity in the plastic hinge at the top of the column. The crack at the joint location closed after the seismic load was removed at each ductility level.

As the displacement increased and the plastic hinge began to develop, a flexural-shear crack pattern appeared throughout the column with the largest cracks measuring 0.4mm at μ_Δ=2 and increasing to approximately 2.5mm at μ_Δ=4. Up to the end of stage 1

(μ_Δ=4) there was little evidence of spalling in the corner elements. This varies from earlier tests of similar columns which began spalling at μ_Δ=3 [Burnell, et al, 2003] likely due to the extreme biaxial loading used in those tests and higher strength concrete in this test. Heavy spalling in the corner elements did occur at μ_Δ=5 on both sides of the column and strain penetration cracks into the pier segment formed. By the final ductility level (μ_Δ=8) the corners had spalled off revealing the spiral and longitudinal reinforcement of the column. The column was not brought to failure in order to maintain as much integrity in the superstructure as possible for future tests.

Conclusions

This test examined the seismic behavior of a precast post-tensioned segmental bridge with a hollow CIP column and internal bonded tendons, under longitudinal seismic motion. In the initial stage of the test, the specimen performed as planned, damage was limited to the column and expected levels of cracking and energy dissipation by the column occurred showing that precast segmental bridges can be designed according to the traditional seismic principle of limiting damage to the column. The column performed very well at high ductilities, but due to the uni-directional loading damage was less severe than previous bi-directional loading tests. Under a more severe loading case and a lower level of prestressing in the superstructure in the second stage of the test, some cracking occurred in the superstructure joints, but most damage was again limited to the column. The hollow column with confined corner elements sustained high ductility levels without losing strength.

Further analytical research will examine how much joint opening is to be expected when a design is used which does not limit all damage to the column, thus allowing a savings in the amount of prestressing required in the superstructure.

Acknowledgments

Funding for this test program was provided by Caltrans under research contract no. 59A0337. The precast segments were constructed by Kie-con Precasters. Tensioning of all strands was completed with the assistance of Dwydag Systems International and the epoxy used between joints was donated by Sika Corporation. Additional construction,

erection, and testing were performed by the capable staff at the Powell Structural Systems laboratories at the University of California, San Diego.

References

ANATECH Corporation, *ANDRIANNA-User's Guide*, Version 1.0, 1999

ASBI. *Recommended Practice for Design and Construction of Segmental Concrete Bridges.* April 2000.

Burnell, K., Megally, S., and Restrepo, J. I., *Cast-In-Place Hollow Rectangular Column Test, Preliminary Results Summary.* Submitted to Caltrans. May 7th, 2003.

Caltrans. *Seismic Design Criteria.* Version 1.1, July 1999.

Dazio, A., Hines, E. M. and Seible, F., *Structural Testing of the San Francisco-Oakland Bay Bridge East Spans Skyway and Main Span Structures, Task B: Skyway Diagonal Pier Test* SSRP-2002/01, University of California at San Diego, CA, Oct. 2002.

Megally, S.H., Garg, M., Seible, F., and Dowell, R.K., *Seismic Performance of Precast Segmental Bridge Superstructures*, Research Report SSRP-2001/24, University of California at San Diego, CA, May 2002.

Priestley, M.J.N., Seible, F., and Calvi, G.M., *Seismic Design and Retrofit of Bridges.* Wiley, New York, 1996.

Seible, F., Dazio, A., and Restrepo, J. I., *Proof Testing in Support of the New San Francisco-Oakland Bay Bridge,* Earthquake Engineering and Structural Dynamics, *in print.* 2004