Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Behavior and retrofit of bridge outrigger beams

C. R. Thewalt & B. I. Stojadinović
University of California, Berkeley, Calif., USA

ABSTRACT: Many older elevated freeways in California were designed considering lateral seismic loads, but the effects of longitudinal bridge response were in some respects ignored. Consideration of longitudinal response has a significant impact on outrigger beams, which are used when columns cannot be located directly beneath the superstructure. These outrigger beams were designed for shears and flexure resulting from gravity and lateral loads. However, longitudinal bridge excitation introduces biaxial flexure and shear as well as torsion into these members. In addition to problems carrying torsion, the outrigger-column joint connection is often poorly detailed. In this experimental study, three half scale specimens are used to investigate the as-built behavior of the outrigger beam and joint as well as the behavior of two retrofit schemes.

1 INTRODUCTION

Although not a common feature in building structures, outrigger beams and knee joints are fairly common in bridges. In many cases the columns of bridges are offset from the deck, resulting in an outrigger configuration. During seismic excitation, the outrigger must resist flexure from gravity and lateral loads, as well as torsion and flexure due to longitudinal excitation of the bridge. The bent cap details, even in older structures, were capable of some ductility under flexure from lateral loads, but the older torsional details were not ductile. The outrigger-column joints in older bridges typically contain no transverse steel and are unlikely to sustain the complex loads from cyclic bidirectional excitation. In fact, small scale tests (Mazzoni et al 1991) indicate that even the current ACI (1989) provisions may be unconservative for knee joints. For these reasons, an experimental investigation of the complex interaction between the forces on the beam and the joint has been performed.

In this project, the bent cap and joint are investigated under combined loads from gravity, lateral, longitudinal motion of the bridge. These loads induce torsion and biaxial flexure and shear in the cap. A representative outrigger bent was selected from the southern freeway at the Interstate 101/280 interchange in San Francisco. The bent had an outrigger length to depth ratio of two, which was long enough to develop extended torsional cracks while being short enough to be experimentally convenient.

2 EXPERIMENTAL PROGRAM

A series of three half scale models of the selected specimen have been tested. The experiments were performed with the specimen inverted, as shown in Figure 1. In each of the models the amount of transverse steel in the columns was doubled from the actual amount, since the column behavior was not the focus of these tests and a shear failure of the column was undesirable during the retrofit tests. The heavy block that is stressed to the laboratory floor represents the deck. At the top of the specimen, the base of the column, there are two actuators to apply independent longitudinal and lateral displacements. There are two additional actuators which apply vertical loads, that vary during the test to account for the effects of gravity and framing action under lateral displacement.

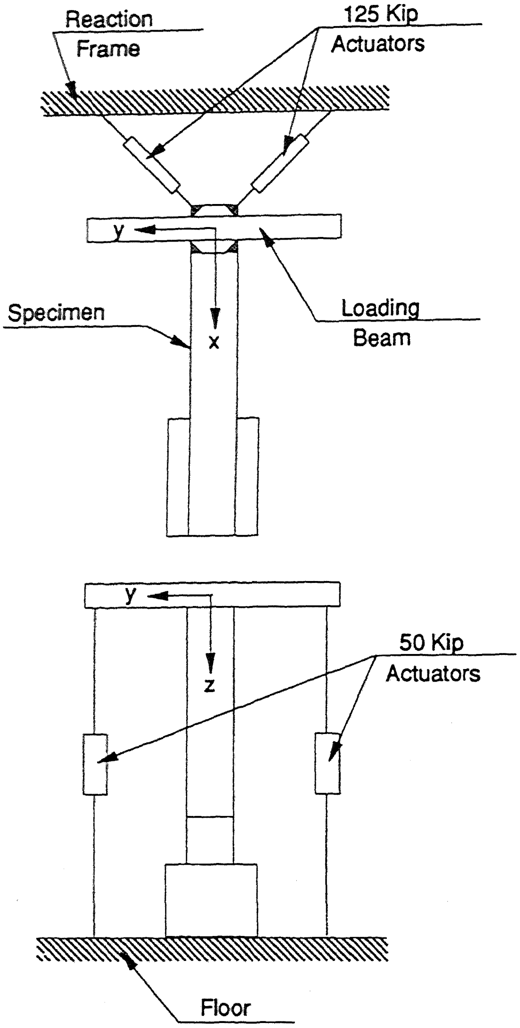

Figure 1: Test Specimen and Loading System

The first specimen represents the as-built condition, typical of 1950's construction in California. The as-built specimen had outrigger stirrups composed of two overlapping U's capped by a tie with two 90 degree hooks. The as-built specimen is shown in the unshaded portion of Figure 2. The cap strength data using ACI (1989) estimates with no interaction is shown in Table 1, and as expected from the open stirrup arrangement the torsional strength after cracking is less than the gross section strength. The column moment capacity was 860 kip-ft in both X and Y directions.

Table 1: Outrigger Beam Capacity

Flexural Capacity (joint closing)	490 kip-foot
Flexural Capacity (joint opening)	920 kip-foot
Torsional Capacity (cracking)	290 kip-foot
Torsional Capacity (nominal)	170 kip-foot

The second specimen is a concrete retrofit of the outrigger beam and joint area. The goal in the retrofit design was to enhance ductility without dramatically increasing capacity. The torsional strength was increased to be about double the cracking capacity. The outrigger retrofit consisted of closed stirrups running above the existing top and bottom beam steel and through added side bolsters. Longitudinal steel was added in the bolsters to increase torsional strength, although the torsional capacity was explicitly designed to be below the load that would cause longitudinal hinging in the column. The joint area was confined by horizontal hoops and external vertical ties.

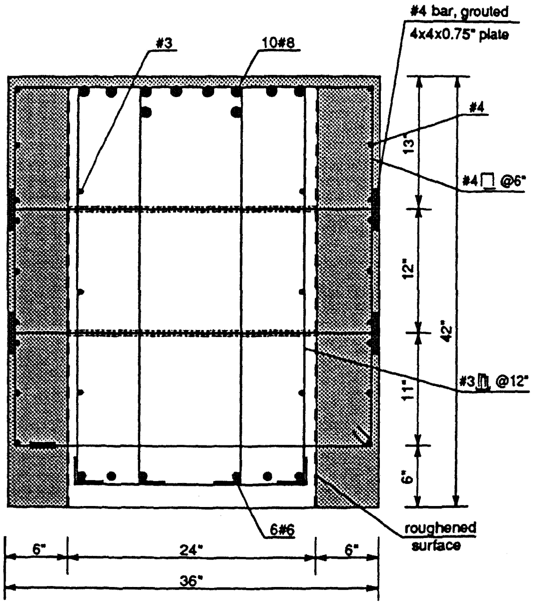

Figure 2: Concrete Retrofit Details

The final retrofit specimen consisted of a half inch thick steel plate jacket around the outrigger and joint, as shown in Figure 3. The plate thickness was selected to be larger than that needed for strength in order to provide adequate confinement between the cross-tie support plates. Additional stiffeners were added to the outside of the plate in the joint area where compression strut reactions were expected. The steel plates were placed around the specimen and welded at the seams, and high strength half inch diameter steel bolts were used as crossties. The bolts were preloaded with a 15 kip axial load as recommended by AISC (1986). After the bolts were loaded, the space between the old concrete surface and the inside of the plate was injected with high strength epoxy.

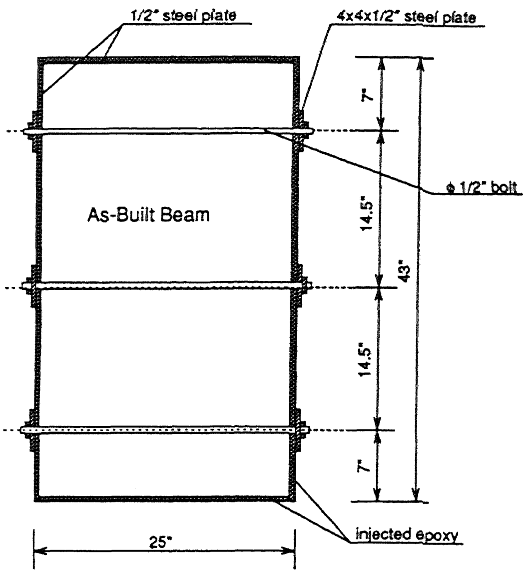

Figure 3: Steel Retrofit Details

Additional specimens are planned to investigate torsion transfer for short outrigger systems, and to test several identical systems under transverse motion only, longitudinal motion only, and combined transverse-longitudinal motion in order to study interaction effects.

The loading history for the specimens is shown in Figure 4. It is a cloverleaf displacement history, with two quadrants applying initial lateral displacements and the other two quadrants applying initial longitudinal displacements. The axial load in the column was forty-five kips plus one half of the applied lateral load. The component proportional to lateral load accounts for the change in axial load due to the lateral response of the frame. The specimen's yield displacement in the longitudinal direction is about 1.5 inches, while in the lateral direction it is 1.0 inch.

The initial plan was to use a rectangular cloverleaf representing equal ductility loops, but preliminary analysis showed that the actual ratio of longitudinal to lateral displacements was sensitive to bridge length and abutment conditions, and the longitudinal response could be larger or smaller than the lateral. Since there was no rational reason to impose displacements based on the yield displacements, it was decided to use square loops. The test sequence consists of two complete loops at each displacement level, with a displacement sequence of 0.25 inch, 0.5 inch, 1.0 inch, 2.0 inch, etc., increasing to failure.

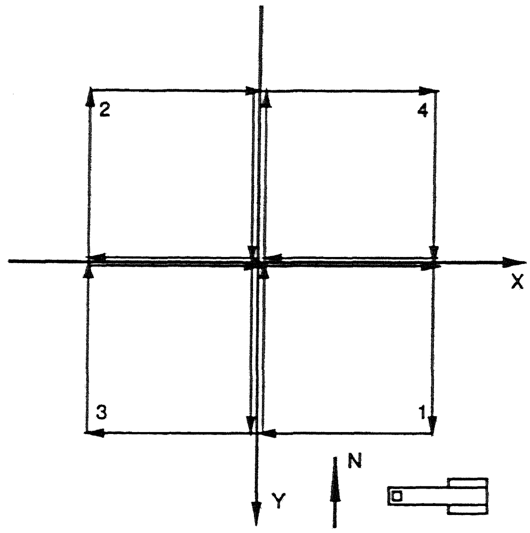

Figure 4: Applied Displacement Pattern

The instrumentation on the specimens includes strain gauges at a variety of locations, as well as potentiometers distributed over the length of the beam to measure the curvature and torsional rotation distribution. The joint was instrumented on two faces to measure axial and shearing deformations.

3 RESULTS

3.1 As-Built Specimen

During the test, response remained fairly linear up to the 1 inch loading level, but significant torsional cracks began to occur at this level. Interestingly, an X pattern of torsional cracks never developed, since the shears produced from gravity and torsion tended to reinforce each other on one side of the beam and cancel on the other. The effect produced diagonal cracks from the interior of the joint across the beam. This was consistent with observations after the Loma Prieta earthquake.

At increasing displacements, the torsional cracks continued to open on each cycle, but failure occurred within the joint during a joint closing motion. The column bars at the outer face of the joint eventually peeled away from the joint, and the concrete in front of the hooked beam bars at the outer corner of the joint crushed. These effects limited the force that could be developed in the outer bars of the column and beam.

The response during closing action consisted of increasing moment and fairly constant stiffness up until the second half of the first two inch loop. After this point the joint suffered severe damage as described above. It can be seen from the hysteresis loops that the final closing moment was only one sixth of the maximum achieved. The opening moments were more stable, increasing up through the four inch cycle, although there was marked stiffness degradation in the final cycle.

The torsional capacity increased up through the four inch cycle, but the stiffness decreased with each cycle after one inch was reached. The premature joint failure during closing limited the forces that could be transferred from the column through the joint, so extended torsional testing was impossible. Since one of the primary interests of the as-built test was to investigate the torsional behavior of the beam, the joint was repaired so that the test could continue to beam failure. The repair consisted of the removal of the joint concrete and the addition of vertical and horizontal steel in the joint, with additional crossties across the joint to prevent the lateral spreading that was observed during the test. This repair was more conservative than either current ACI (1989) provisions or the ACI-ASCE Committee 352 (1985) provisions, in that both horizontal and vertical shear steel was added in the joint. The goal was to prevent further joint damage and examine the outrigger behavior.

The flexural behavior of the as-built and repaired specimen are shown in Figure 5. It is clear that the joint repair greatly enhanced the flexural capacity of the system. The torsional response of the system is shown in Figure 6. The large diagonal crack in the beam was not repaired when the joint was repaired, so the torsional capacity did not increase. The diagonal crack continued to open on each cycle and ultimately when the gap was wide enough to lose aggregate interlock the stirrups broke in rapid succession and the specimen broke in two.

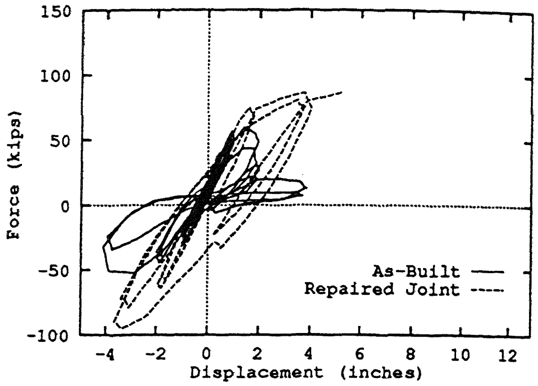

Figure 5: Transverse Response at Top of Column

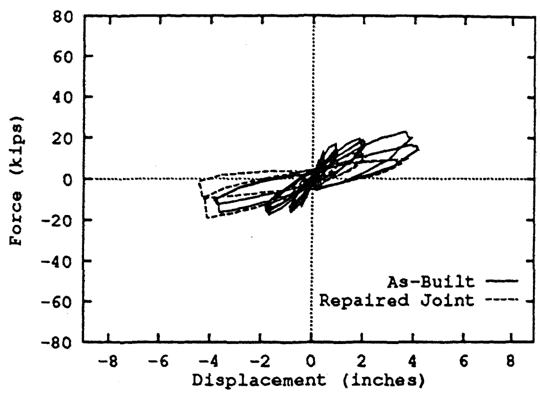

Figure 6: Longitudinal Response at Top of Column

3.2 Concrete Retrofit

The specimen with the concrete retrofit performed well throughout the testing sequence. In fact it never failed, and the test was terminated because the hydraulic actuators had reached their stroke limit.

The flexural capacity was stable through the 8 inch cycles, as shown in Figure 7. The maximum joint closing displacement of around 12 inches represents a ten percent drift. The joint retrofit also performed well, with only minor cracks forming in the joint region throughout the test.

The torsional response, shown in Figure 8, was not as stable and decreased with increasing displacement as the outrigger became more cracked. Unlike the as-built specimen, which concentrated damage along a single inclined crack, the cracks in the concrete retrofit specimen were well distributed throughout the cap beam. The side bolsters began to shift relative to the as-built beam during the four inch loops and by the eight inch loops significant relative displacements were present and the bolsters had separated from the beam at the bottom of the specimen. The behavior would have been enhanced if the new closed stirrups were positioned below the bottom steel but the location shown in Figure 2 was mandated by the research sponsor.

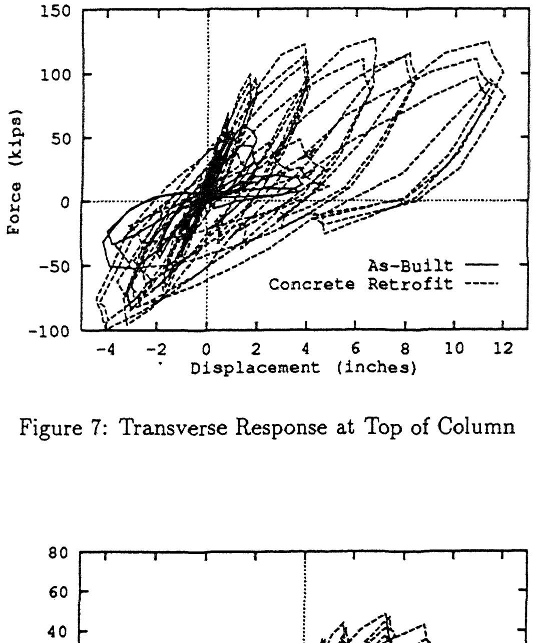

Figure 7: Transverse Response at Top of Column

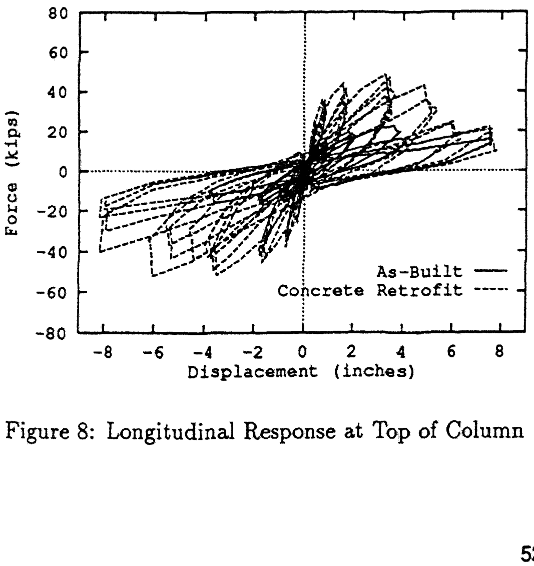

Figure 8: Longitudinal Response at Top of Column

3.3 Steel Retrofit

The steel specimen behavior was quite different than the concrete, in that the relative strengths of the system components were now drastically different. The addition of the half inch thick plate made the entire outrigger extremely strong and rigid relative to the column. Before the test there was some concern that it would be hard to estimate the extent of damage within the concrete under the plates, but the column essentially acted as a fuse, limiting the amount of force that was transmitted into the system.

As the test progressed the base of the column near the outrigger sustained increasing damage. Concrete began to spall at the two inch cycle, and by the four inch cycle significant column bar buckling was already evident near the column base. The column hinged under both transverse and longitudinal loads.

The transfer of torsion from the jacketed beam to the support block worked well, but flexural problems were imminent in the beam's weak direction at the point of attachment to the support block. Had the column been stronger in the longitudinal direction there would have been extensive bar yielding at the outrigger/block interface. Thus, enhancing the column capacity or confinement in the hinge region would not have resulted in a better system.

As can be seen in Figure 9, the flexural response of the system was already decreasing by the second six inch cycle. The torsional response shown in Figure 10 appears to dissipate more energy than the loops for the concrete retrofit. This is because the dissipation in the steel specimen is emanates from the column hinging flexurally. The test was terminated after the six inch cycles because of extensive damage at the base of the column.

4 CONCLUSIONS

The as-built test clearly demonstrated that the outrigger system was deficient in two key areas. First, the essentially unreinforced joint was incapable of developing the strengths of the connected column and beam, especially under cyclic loads. The second problem was that with a repaired joint the outrigger becomes the weak link susceptible to brittle shear failure. The nature of the joint failure, including crushing in front of the lower beam steel hooks and peeling off of the straight column bars influenced

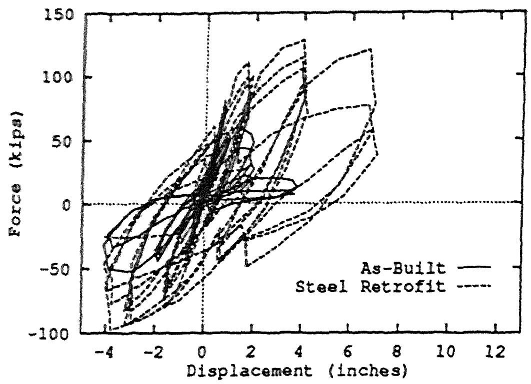

Figure 9: Transverse Response at Top of Column

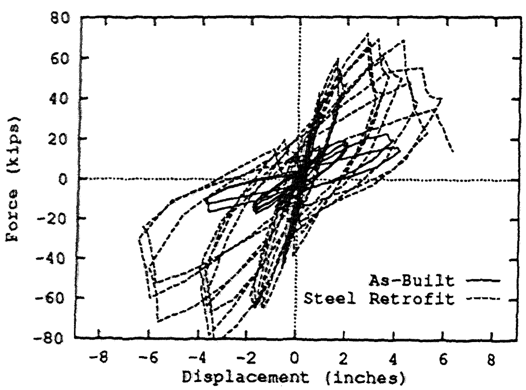

Figure 10: Longitudinal Response at Top of Column

the joint retrofit design.

The concrete retrofit was well behaved, allowing a ten percent lateral drift in the joint closing direction with no noticeable loss in flexural capacity. The torsional retrofit in the outrigger worked well. Its design level of approximately twice the cracking torsional capacity was also well chosen in that it limited the maximum longitudinal load that could be taken by the specimen. The longitudinal load must be controlled to avoid flexural failure in the weak direction of the outrigger beam at the point of attachment to the deck. The joint retrofit, consisting primarily of steel and concrete external to the original joint, performed well, transferring the full loads needed to fail the cap and column.

The steel retrofit certainly limited damage to the cap beam and joint area, but the drift capacity of the system was dramatically reduced because all damage was now forced to the base of the column, for both lateral and longitudinal loads. Simply detailing the plastic hinge region in the column to retain capacity through a larger rotation would have forced the failure to the cap/deck interface through flexure in the weak beam direction, since there was already evidence of bar yielding at this location under the reduced load levels.

The development of a retrofit strategy to achieve desired displacement levels while maintaining loads requires careful consideration of the complete system. It was found that simply strengthening the weak link was not the best thing to do, one must not overload the next weakest link in an undesirable fashion.

5 ACKNOWLEDGEMENTS

The authors would like to thank the Karl Kardel Company of Oakland California who graciously contributed their time and equipment to inject the epoxy into the steel specimen.

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