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Tests of Alternate Seismic Retrofits for Oak Street Bridge

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ABSTRACT

As part of a seismic retrofit program, the British Columbia Ministry of Transportation and Highways sponsored a test series for typical reinforced concrete bents of Vancouver's Oak Street Bridge modelled at 0.45 scale. Five bent models were tested in the UBC Structures Laboratory with a slow cyclic lateral load. One test was on an unretrofitted model, while the other four were of various types of retrofit.

The behaviour of each of the test retrofits has been utilized by the Ministry's consultants, Klohn-Crippen International Ltd., to complete detailed design of the retrofits for the bridge.

INTRODUCTION

The British Columbia Ministry of Transportation and Highways is currently carrying out seismic retrofits of several bridges in the Vancouver region. The 35 year old Oak Street Bridge over the Fraser River, a key link in the local transportation system, has many reinforced concrete bents that are in need of seismic retrofit. Because of the large number of bents, and because other bridges have similar bents, the Ministry funded a program to test five bent models including an as-built model and four models with various retrofit schemes (Anderson et al 1994). The retrofit concepts were a joint collaboration among the authors, with Klohn-Crippen handling detail design and construction of the specimens, and UBC designing and carrying out the test program. The specimens, 45% scale models of the upper half of the two-column bents, were constructed and trucked to the structures laboratory at the University of British Columbia by APS of Langley BC.

The tests on the specimens consisted of applying a constant vertical load to simulate the dead load of superstructure and the appropriate selfweight of the bents, and a slow cyclic lateral load at increasing amplitudes to test the ductility capacity of the bents.

This paper discusses the retrofit schemes and the corresponding hysteretic load deflection behaviour. A companion paper (Jennings et al, 1995) discusses the implications of the tests and application of the results to the retrofit designs for the Oak Street Bridge. An earlier paper (Kennedy et al 1992) discusses the many seismic problems of the bridge.

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BRIDGE DESCRIPTION

Oak Street Bridge was constructed in the mid 1950's, prior to the introduction of any significant seismic design requirements for bridge structures. It carries four lanes of traffic over the North Arm of the Fraser River, and connects Vancouver to Richmond. Because of ship clearance requirements the bridge has long approaches on both the north and south sides. These approach structures for the most part comprise two column bents supporting concrete girders. The bent legs are supported on individual footings which are supported on timber piles. The footings in some cases are sufficiently strong, and in other cases will be retrofitted with overlays and column jackets, to force yielding into the columns. Figure 1 shows a typical four span segment of the approach structure near bent S28 on the south approach.

TEST ARRANGEMENT

The prototype was modelled by specimens representing the portion above the column inflection points (upper half of the height). Height and load restrictions in the UBC Structures Laboratory, and convenience in scaling bar sizes, led to the choice of a linear scale of 0.45. This corresponds to the use of #5 (Imperial) reinforcing bars in the model for the main #11 bars of the prototype.

Figure 2 shows a view of a specimen in the loading frame. The dead load of the deck and the appropriate portion of the selfweight of the bent was provided by five vertical jacks acting through a linkage on the five bearing locations on the bent capbeam. The dead load jacks were maintained under constant pressure control throughout a test. The cyclic lateral load was provided by a jack located at the elevation of the mass centroid of the deck above the cap beam, and transferred to the top of the cap beam by a truss. This arrangement provided the appropriate overturning effect to accompany the lateral load. The truss from the lateral jack to the cap beam permitted a small amount of longitudinal extension of the cap beam, expected as shear cracks and flexural yielding developed. The base of the columns were supported on a rocker arrangement that would allow rotation. A system of Teflon pads and instrumented restrainer bars was installed at the base in an attempt to measure column shears, but this did not appear to give accurate readings.

Displacements were measured at both column joints. Other instrumentation consisted of an external five bar arrangement anchored at one of the joints to measure joint shear deformation, along with an internal system of gauged bars to measure diagonal joint strains. In addition there were of the order of 70 strain gauges, located on the longitudinal reinforcement and on some of the shear stirrups and column ties. The total number of channels of data recorded was about 90, with all data recorded at a scan rate of 0.5 per second, i.e., one scan every 2 seconds.

The lateral loading program consisted of a series of sequences of cycles of increasing amplitude, where each sequence consisted of three cycles. Several low level sequences were carried out under load control to test the data acquisition system and determine the initial stiffness of the specimen. Yield of the specimen was determined by measuring the deformation at a load equal to 75% of the calculated flexural capacity, and then linearly scaling this to the flexural capacity. This established the yield displacement and provided a basis for calculating displacement ductilities. Testing then continued under displacement control with sequences at ductility levels of 1, 1.5, 2, 3, 4, 6, 9, and 12, or to failure, whichever occurred first.

SPECIMENS

Five identical as-built specimens were constructed. The first, denoted OSB1, was tested in its as-built condition, while the remaining four were later retrofitted with different schemes as discussed below. Specimen dimensions are shown in Fig. 3. Imperial units were used in order to conform with the prototype design. Material strengths were specified to closely match those of the prototype (see Anderson et al 1994). Bar cutoffs were calculated so that the position where the embedment could develop yield in the bar would be the same in the model as in the prototype. This results in a slight difference in the position of the bar cutoffs between the model and prototype. For the retrofits any new mild steel was specified to be Grade 400 and concrete strength was specified as f'_c=6 ksi. Prestressing steel was 7 wire low relaxation grade 1862 strand, or hot rolled stress relieved f_{pu}=160 ksi Dywidag bars.

The first retrofit, denoted OSB2, was a minimal retrofit and consisted of 4-$^1/_2$" and 2-$^5/_8$" diameter prestressing strands placed and grouted in holes cored through the length of the cap beam, producing an average prestress in the cap beam of 412 psi. The prestressing was designed to increase the moment capacity of the cap beam so as to force yielding into the tops of the columns, and to provide an increase in shear capacity. This retrofit caused little change in the appearance of the bent, and required no external formwork. It required highly accurate longitudinal coring of the cap beam.

Specimen OSB3, shown in Fig. 4 involved a heavily reinforced beam placed under the existing cap beam, steel plates anchored into the top of the cap beam, vertical Dywidag bars tying the new beam and top steel to the existing beam and doubling as new shear reinforcement, and finally, circular steel jackets placed around the columns to improve confinement. The new beam and shear reinforcement were designed so that the cap beam would not yield or fail in shear, and yielding would be forced into the top of the columns above the jackets.

Specimen OSB4, Fig. 5, was a combination of some of the features used in OSB2 and OSB3. Prestressing of 342 psi was used in the cap beam to increase the moment capacity and force yielding into the columns, vertical Dywidag bars were added as beam shear reinforcement, and steel jackets were added to the columns.

The OSB5 retrofit used external ungrouted prestressing of 342 psi using Dywidag bars anchored at each end of the cap beam to force hinging into the columns, but for increased shear capacity in both the cap beam and columns, fibreglass wraps as shown in Fig. 6 were used. The principal direction of the wraps is perpendicular to the member, so that the fibreglass improved shear and provided confinement, but did not add significantly to the flexural capacity. The design of the fibreglass wrap was in general conformance with recommendations of the developers of the system (Hexcel-Fyfe). In the cap beam the thickness was calculated to provide a vertical clamping force equal to the capacity of the vertical Dywidag bars in OSB4, which was considerably in excess of that required for shear. This resulted in three wraps on the beam, or a nominal thickness of 0.15 inches. The column wraps were given a nominal thickness of 0.10 inches.

TEST RESULTS

An overall view of the results for the Oak Street Bridge can be made by comparing the hysteretic curves for the five test specimens given in Figs. 7 to 11. The joint displacement in the figures is the average horizontal displacement measured at the two joints, and the base shear is the total base shear or applied lateral load.

The as-built specimen showed very poor, brittle, behaviour, as evidenced by the hysteretic curves shown in Fig. 7. During the test, a large diagonal crack, consistent with negative moment, formed near each end of the cap beam. The crack increased in width with each cycle of load until ultimately the concrete compressive zone suddenly failed. The top of the crack coincided with the cut-off of several of the top bars. The wide cracks were due to bond failure, as evidenced by horizontal splitting at the level of the top reinforcement. Neither the joint region nor the columns showed any serious cracking. The theoretical lateral load capacity based on flexural member strengths was never achieved because of the force limiting cap beam shear failure.

The results for the prestressed retrofit OSB2 showed a marked improvement in both strength and ductility when compared to the as-built specimen, as evidenced by Fig. 8. In OSB2, diagonal cracks formed in the beam at nearly the same location as in OSB1, but they did not open as wide, even at the larger load levels, and the beam did not fail in shear. Flexural shear cracks formed in the columns in the region of the flair, gradually getting wider and longer as the ductility level increased. Up until the last load cycle, when a sudden column shear failure occurred, there was very little strength degradation, with the three hysteresis curves at each sequence nearly falling on top of each other.

The hysteretic response of specimen OSB3 is shown in Fig. 9. The specimen eventually failed in flexure in the column immediately above the steel jackets. At high ductility levels the concrete cover above the steel jackets spalled, allowing the column longitudinal bars to buckle and eventually fracture. The force displacement curves show very good performance to a ductility level of 9 with the strength continuing to increase. The addition of the underbeam and vertical reinforcement to the cap beam made the beam sufficiently strong in both flexure and shear to force flexural hinging into the columns with little damage to the beams. The cap beam developed negative moment and shear cracks near the joints at early stages of loading, but they did not progress or grow as the displacements increased, indicating that the retrofit scheme adequately protected the cap beam from damage. At a ductility level of 6, the column and the joint region had many cracks but there was no significant spalling of the cover concrete. Spalling became extensive at ductility level 12. The vertical column bars along the exterior face of the column all buckled, and some fractured as the number of cycles increased at this level.

The OSB4 test, with vertically and horizontally prestressed cap beam and column jackets, behaved similar to the OSB3 specimen. The specimen eventually failed in flexure in the column region above the column jackets by spalling of the cover concrete and buckling of the vertical column reinforcement. The hysteretic force displacement curves are shown in Fig. 10, and show very good performance to a ductility level of 9. At low ductility levels flexure and shear cracks developed in the cap beam, but these did not widen or grow as the test continued and the beam retrofit was successful in forcing the damage into column flexure. At ductility 4 there is considerable cracking in the column but little cracking in the joint. Ductility 6 showed wider cracks in the column and the beginning of diagonal cracks across the joint. At a ductility level of 9 some spalling occurred near the column corner bars, with the result that the corner bars buckled. At ductility level 12 level there is much more spalling of the cover concrete, more bars buckled, and some bars fractured.

The final test, OSB5, was the specimen with externally prestressed cap beam and fibreglass wrap on beam and column. The performance of this retrofit, as shown in Fig. 11, was very good up to a displacement ductility level of 9 where the test had to be terminated because of limitations in the displacement capacity of the loading system. This specimen was more flexible than the two previous specimens which had full length steel column jackets, thus the yield displacement was larger, and so even though the maximum displacement was greater than in the previous tests the ductility level was not as high. This specimen showed less cracking than each of the other retrofits. As loading progressed more cracks developed in the joint area but not in the short length of the beam that was visible. At peak deflection all the cracks were relatively small, including those in the joint region. A large crack did develop in the flared portion of the column just above the top of the column wrap. The size of the crack did not seem to affect closure on load reversal or give rise to spalling. During the sequence to ductility 9 a small amount of spalling took place at one corner on the interior face of the column just above the fibreglass wrap, allowing the corner column bar to buckle.

DISCUSSION

The performance of the as-built specimen, OSB1, clearly shows the need for seismic retrofitting of these types of bents. The failure of the test specimen was brittle and sudden, characteristics that are undesirable for seismic response. In addition, the strength was far less than adequate by today's standards. Notwithstanding the loss of lateral strength and potential loss of lateral stability, the vertical load carrying capacity of the bent would be questionable if it had been subjected to additional load cycles.

Analytical methods currently available are not very accurate in predicting the shear capacity of concrete members such as the cap beam, when the amount of shear reinforcement is as low as in this case. The OSB1 test helped to verify the rather crude predictions. It also very dramatically demonstrated the poor failure behaviour likely to be found in the prototype bridge. The failure modes that were observed included the early and large shear cracks which developed in the expected locations, i.e. on a diagonal near each end of the cap beam, where shear is high and shear reinforcement inadequate. In addition the top reinforcing steel extending from the columns along the top towards the central region, demonstrated bond failure. These bars were detailed with numerous cutoffs, and the lack of significant confining steel from shear stirrups made it impossible for the top steel to develop its yield capacity.

In contrast to the "as-built" OSB1 specimen, all the retrofitted specimens, with the possible exception of OSB2, behaved very well. OSB2 developed a satisfactory lateral load capacity and a ductility about 4 or displacement about 2 inches, and the load was stable through several cycles at each load level. However, the columns suffered a sudden shear failure in the last load cycle at a ductility of about 6 (3" displacement), and the large shear crack in the beam proved worrisome.

The OSB3 test included a column retrofit and an underbeam post-tensioned to the cap beam. With both the cap beam and the columns now improved, it is not surprising that the failure occurred in the joint region. Maximum load and displacement were highly satisfactory. More significantly, cracks were small and widely distributed. OSB3, however, suffers from two real problems; its appearance is less desirable due to the heavy underbeam and the post-tensioned rods extending below the soffit, and the likelihood of higher cost due to the extensive formwork needed for the underbeam.

Many of the advantages of OSB3 are characteristic of the OSB4 specimen. Longitudinal and vertical prestressing of the cap beam in OSB4 provided vertical shear steel and horizontal compressive stress for shear strength, and enhanced the bond of the horizontal steel for cap beam flexure.

One of the more interesting results was the performance of OSB5, which was post-tensioned and then wrapped with fiberglass cloth bonded with epoxy. The specimen performed well up to the displacement limits of the test system. Cracking was limited by the confining action of the fiberglass. The replacement of the column steel jackets of OSB3 and OSB4 by fiberglass on OSB5 permitted more curvature in the columns, contributing to the greater displacements. Since the yield displacement was also increased, the ductility was less than for OSB3 and OSB4. The confinement offered by the fiberglass contributed to maintaining the bond on the longitudinal bars and limiting crack widths.

CONCLUSIONS

The poor behaviour of the original bent, and the various responses of the test bents, served to provide a rational basis for design of economical and practical retrofits for the actual bridge. While analytical methods can provide a starting point in this type of design, they are particularly crude in the case of the older, lightly reinforced structures that have a variety of deficiencies. The test program has provided the designers with a sound basis for proceeding with the retrofit.

ACKNOWLEDGMENTS

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REFERENCES

Jennings, D., Kennedy, D., Anderson, D.L., and Sexsmith, R.G., 1995, "Design of Seismic Retrofit for Oak Street Bridge Bent", Proceedings, Seventh Canadian Conference on Earthquake Engineering, Montreal.

Hexcel-Fyfe Co. - TYFO S Fibrwrap System. Information available from Hexcel-Fyfe Co., 1341 Ocean Ave., Del Mar, Calif. 92014.

Kennedy, D., Turkington, D, and Wilson, J., 1992, "Design for Earthquake Retrofit and Widening of the Vancouver Oak Street Bridge, Annual Conference, Canadian Society for Civil Engineering, Quebec.

Anderson, D.L, Sexsmith, R.G., Seethaler, M., Kennedy, D, and Jennings, D., 1994, "Oak Street Bridge Two Column Bent Tests", University of British Columbia Dept of Civil Engineering Earthquake Engineering research Facility Technical Report 94-01.

FIGURES

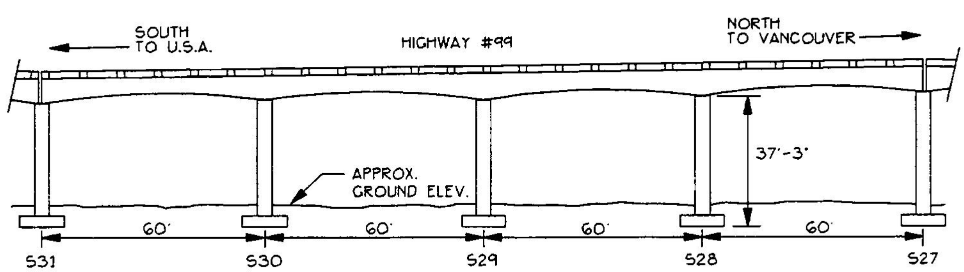

Figure 1. Typical Four Span Segment of Oak Street Bridge.

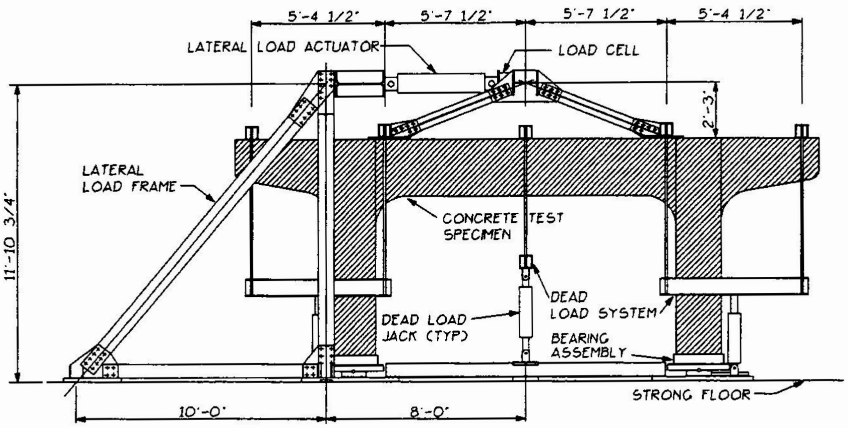

Figure 2. Typical Test Arrangement.

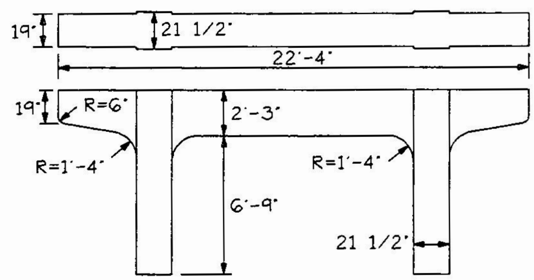

Figure 3. Specimen OSB1.

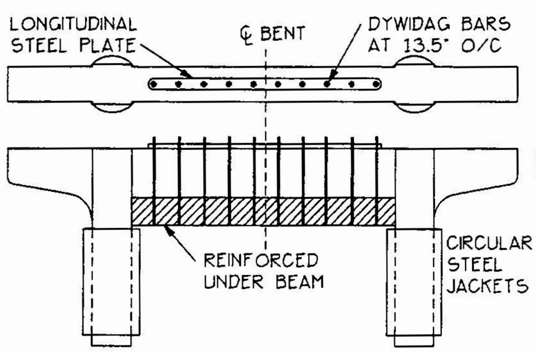

Figure 4. Specimen OSB3.

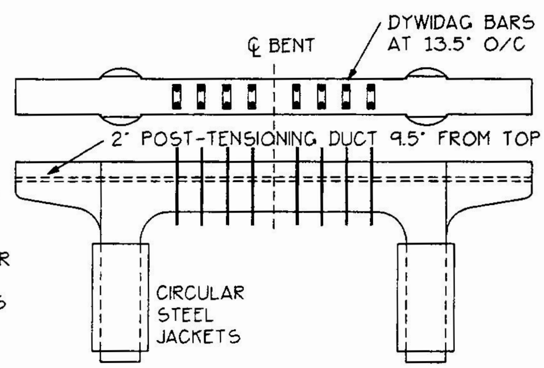

Figure 5. Specimen OSB4.

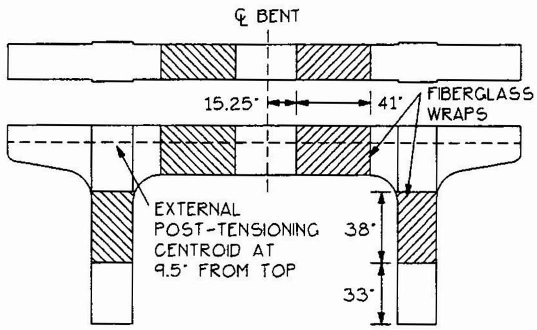

Figure 6. Specimen OSB5.

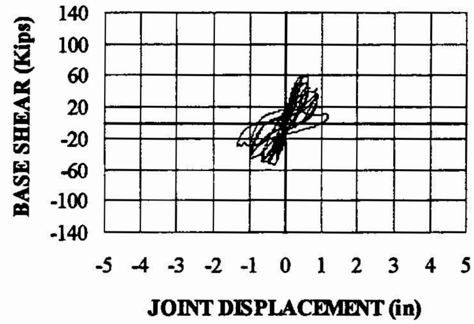

Figure 7. Hysteresis for OSB1.

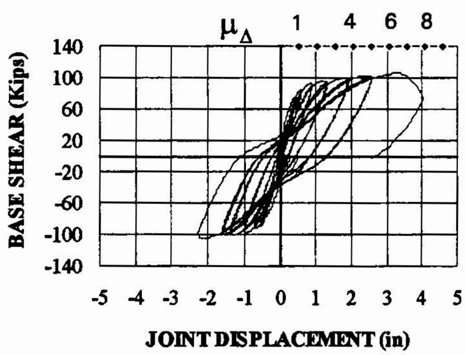

Figure 8. Hysteresis for OSB2.

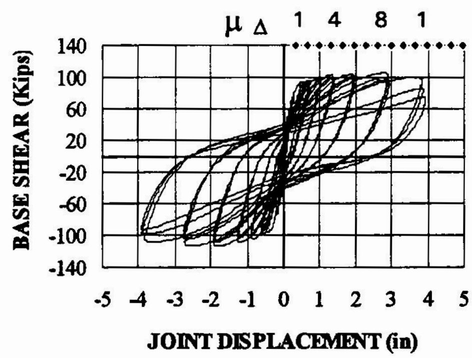

Figure 9. Hysteresis for OSB3.

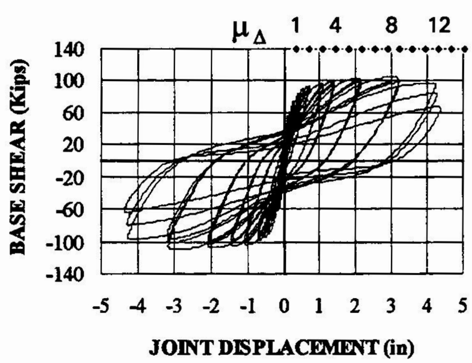

Figure 10. Hysteresis for OSB4.

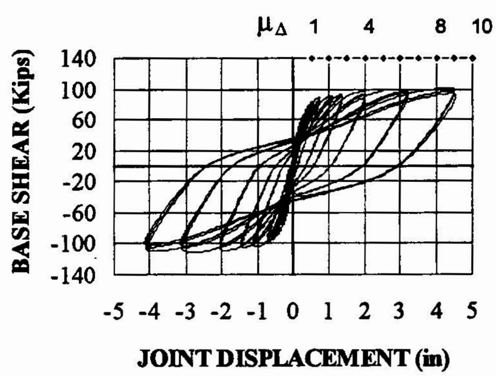

Figure 11. Hysteresis for OSB5.