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Design of Seismic Retrofit for Oak Street Bridge Bent

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ABSTRACT

In March of 1993, the British Columbia Ministry of Transportation and Highways (The Ministry) contracted Klohn-Crippen (formerly Crippen) to undertake the first phase of detailed seismic retrofit of the Oak Street Bridge. The bridge, originally designed and constructed in the mid 1950's, is a major link connecting Vancouver and Richmond.

The Ministry recently commissioned a program to test model specimens representing a two-column reinforced concrete bent typical of those found on the Oak Street Bridge approach piers. The purpose of the program was to confirm the perceived deficiencies of the existing bents and to test the effectiveness of proposed retrofit measures. Testing of five 0.45 scale model specimens was performed at the University of British Columbia. One was tested as built. Three more were fitted with retrofit measures developed by the Klohn-Crippen/UBC team and designed by engineers at Klohn-Crippen. The last was later retrofit and tested by the UBC team (Anderson et al, 1995).

Analysis methods utilized to determine existing and retrofit capacities included the ASCE/ACI beam shear equation, the modified compression field theory in both two and three dimensions, strut and tie models, the Mander model for sectional analysis, and collapse mechanisms. The adequacy of these methods for retrofit design are discussed in this paper as well as the implications the retrofit schemes had on the seismic upgrade of the Oak Street Bridge.

INTRODUCTION

The prototype for the model specimens was based on pier S28 with pins placed at column inflection points, assumed to be located half way up the column. Two main objectives of each retrofit measure were to enhance the performance of the seriously deficient as-built cap beam and to force flexural hinging into the columns. In keeping with the principles of capacity design, the level of cap beam retrofit increased when the columns were fitted with steel jackets.

All three retrofit measures dramatically improved the shear and moment resistance of the cap beam while enhancing the integrity of the beam/column joint region. Flexural hinge regions were successfully forced out of the cap beam and into the columns. Both analysis and testing indicated that a combination of horizontal and vertical reinforcing was required to adequately increase the shear capacity of the cap beam. Klohn-Crippen's third retrofit scheme (OSB4) was conceived with this knowledge and found to be an ideal solution for the seismic upgrade of many of the concrete bents on the Oak Street Bridge.

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THE AS-BUILT BENT AND ITS DEFICIENCIES

Many of the Oak Street Bridge piers have very light transverse cap beam reinforcing. The volume and arrangement of the stirrup steel is inadequate for shear, for concrete confinement, and for restraint of main bars from buckling in potential hinge zones. Cap beam flexural reinforcing curtails very early with regard to seismic moment demands: negative moment reinforcing reduces to two #11 bars at midspan and positive moment reinforcing reduces to four #11 bars at the column face (with only two feet of embedment into the column). Column bars terminate without hooks at the top of the joints and column ties comprise only three legs of #3 bars at 12 inches on centre in potential top of column hinge zones. Horizontal beam-column joint reinforcing also comprises only three #3 ties at 12 inch centers.

The as-built specimen OSB1, shown in Figure 1, is representative of the bridge piers as they now exist. The model hysteresis plot provided in Figure 1 graphically illustrates the extremely poor lateral load response expected from the existing bridge piers and confirms the severity of the identified deficiencies. Based on test observations and the hysteresis plots obtained, the failure of actual bridge piers during even a moderate earthquake is expected to be sudden, brittle, and potentially catastrophic.

RETROFIT SPECIMENS

Retrofit Specimen One (OSB2)

The design for cap beam strengthening on this specimen incorporated bonded internal post-tension strands inside two parallel longitudinally cored holes (see Figure 2). The post-tensioning was chosen to be internal for two reasons: it was thought more efficient to have the strands bonded and integral with the existing cap beam; and it does not affect the existing appearance of the structure. The strands were placed in the upper region of the kern of the cap beam cross section to optimize the negative moment capacity while ensuring no permanent tension was introduced on the soffit concrete fibres. The coring tolerances on the model specimen were easily achieved.

Retrofit Specimen Two (OSB3)

The retrofit of specimen OSB3, shown in Figure 3, combined a new reinforced concrete underlay beam clamped to the existing cap beam with vertical post-tension rods inside of cored holes. The width of the new beam was chosen (based on advice from the contractor) to match the existing column width, which is 6 inches wider than the cap beam on the prototype, so as to provide easier access for the concrete pour. The vertical post-tension bars were necessary for shear but also provided a clamping force which improved the bond conditions for the top layer of existing reinforcing. A steel plate was installed on top of the existing beam to increase the amount of negative moment reinforcement. Steel column jackets were provided on this specimen to prevent column shear failure.

Retrofit Specimen Three (OSB4)

During construction of retrofit specimens OSB2 and OSB3, information was gained about problems to expect during construction. Also during this time it was discovered that the coring tolerances desired for the Oak Street Bridge piers, using the OSB2 configuration, may be difficult to achieve. Much of the experience gained here helped determine the requirements of the third retrofit scheme OSB4, shown in Figure 4.

The OSB4 retrofit specimen incorporated longitudinal post-tensioning through a single cored hole in the cap beam, two rows of vertical cap beam post-tensioning rods, and a round steel column jacket. A single horizontal tendon was chosen because larger coring tolerances could be accommodated and one core hole is more economical than two. The amount of longitudinal post-tensioning required to force plastic hinging into the columns was not sufficient alone to prevent beam shear failure, therefore vertical post-tensioned bars were necessary. Note also that transverse clamping plates were placed between the vertical P/T anchors and the top of cap beam in order to improve bond conditions for the existing negative moment cap beam reinforcement.

ANALYSIS AND DESIGN TOOLS

One of the most important aspects of the test was capbeam shear performance, although the reinforcing details contributing to both shear and flexural capacity were inadequate. The following tools were utilized to predict the performance of the as-built and the three retrofit model specimens.

ASCE/ACI Beam Shear Equation

The ASCE/ACI Committee 426 relationship for v_c , equation (1), was adopted for the more traditional determination of the capbeam concrete shear strength. The shear capacity of the stirrup steel was determined assuming 45 degree truss action.

$$V_c = \phi\left(0.85 + 120\rho_w\right)\sqrt{f_c'}\, b_w d \;\le\; 2.4\phi\sqrt{f_c'}\, b_w d \qquad (1)$$

Modified Compression Field Theory

The Modified Compression Field Theory (MCFT) is recognized by many as a more sophisticated method for assessing the capacities of concrete elements. Using sectional analysis considering equilibrium, compatibility and stress/strain relationships with an allowance for concrete tensile strength, the MCFT combines axial load, moment and shear to determine the response of the section. When designing transverse reinforcement, the shear capacity is broken down into concrete and steel components. The concrete component was found to be slightly higher with this theory than with the ASCE/ACI beam shear equation.

A three dimensional program utilizing the modified compression field theory, SHELL 474 (Adebar et al, 1994), was also used as a background check for the cap beam capacity of all of the specimens tested. In general, this program gave slightly more conservative capacities (by 5 to 10 percent) than the MCFT used in two dimensions. Given that these retrofit designs were to be tested, the cap beam capacities were permitted to fail the SHELL 474 analyses by a 5 to 10 percent margin.

Strut and Tie Models

With the inherent discontinuity of the transverse mild reinforcement in the second retrofit specimen (the underlay scheme) the effectiveness of both the traditional approach and the MCFT were thought to be inconclusive, although they were still used for comparison. A strut and tie model (effectively a truss) incorporating a series of concrete compression struts in combination with transverse and longitudinal tension ties was believed to provide a more accurate representation of the force distribution through the cap beam.

Collapse Mechanisms

The typical collapse mechanism for a concrete bridge bent incorporates top and bottom column hinging. It is considered prudent practice to keep hinge zones out of the cap beam because of its critical role in transferring girder bearing loads to the columns. One of the criteria imposed during the design stage was that the retrofit cap beam would be strong enough to force top-of-column hinging. Mechanism analyses for the model bents were run using "DRAIN-2DX" (Kanaan et al, 1995). Nominal material properties were used for calculation of the column moment/axial interaction curves and appropriate overstrength factors were then applied to the resulting cap beam demands. Only the top half of the concrete bent was modelled, therefore the mechanism for the model had only two top-of-column hinges. The section used to represent the hinge zone in the model was taken at the top of the hinge and overstrengths of 1.2 and 1.35 were used for non-jacketed and jacketed columns respectively.

The Mander model (which predicts the compressive strength of confined concrete) was used to determine moment-curvature plots for the top-of-column section. The ductility capacity of a hinge in this region was then estimated assuming concrete spalling as the ultimate limit state. The column hinge zone concrete shear capacity was taken as $3.5\sqrt{f_c'}$ for rotational ductilities less than 2 with a degraded concrete shear capacity of $1.2\sqrt{f_c'}$ for rotational ductilities in excess of 4.

DISCUSSION OF RESULTS AND EFFECTIVENESS OF TOOLS

The mechanism analysis utilizing capacity design principles was performed on all specimens and found to be a reliable tool for assessing cap beam demands. After observing the test results it was determined that a common 1.2 overstrength could be used for both jacketed and non-jacketed columns provided the section where the model hinge would form was taken to be the centre of the actual hinge. Note that this overstrength value is based on measured reinforcement yield strengths rather than 5th percentile yield values normally used in design.

OSB1

The test model failed at a beam shear demand of 58 kips following the formation of a flexural shear crack adjacent to the haunch where the stirrup steel drops off dramatically (see Figure 1). The failure mode was brittle with no ductility capacity.

The traditional theory (ASCE/ACI equation) found the as-built capbeam shear capacity to range from 43 kips near midspan to 75 kips adjacent to the haunch. The capacity of the section passing through the inner endpoint of the flexural shear crack (approximately a distance "d" from the haunch) was determined to be 58 kips. Although this approach incorrectly predicted a shear failure closer to midspan at a demand of only 43 kips, it erred on the conservative side.

The MCFT predicted the potential for yielding of the top reinforcement (depending on the angle of the compression strut used) to occur over a distance extending from the haunch to midspan at a beam shear demand of approximately 50 kips. It was further suspected that bar pullout would govern over yielding, as the top bars were prematurely cut off and therefore inadequately anchored. This debonding of the negative moment reinforcement was observed during the testing and consequently helped facilitate large flexural shear cracks spanning from this debonded plane to flexural cracks in the beam soffit. The failure in the performance of the top steel indicates that the early curtailment of the negative moment reinforcement was as serious a deficiency as the lack of transverse reinforcement. The MCFT ultimately predicted the cap beam to fail at approximately 55 kips shear demand adjacent to the haunch.

OSB2

A significant beam shear crack developed in the same critical region of the cap beam that produced failure in OSB1, however the post-tensioning was sufficient to enable the beam to carry the shear demands while forcing plastic hinging into the columns. Shear failure eventually occurred in the tension column hinge zone at a global ductility of about six (see hysteresis loops in Figure 2), three times that expected during the design seismic event. The beam shear crack was an indication that a more ideal retrofit would require vertical reinforcing as well. Note that the column shear failure was predicted to occur in the tension column hinge zone at a rotational ductility in excess of four.

The cap beam column joint region was not expected to fail during the OSB2 test because of the compressive benefit of the internal post-tensioning. Although there was cracking within the joint at moderate ductility levels, the crack widths were not of a serious nature.

OSB3

Testing found the retrofit capbeam of OSB3 to be very robust with little cracking observed throughout the duration of the test. This was likely due to the fact that the observed column hinge overstrength of 1.15 was approximately 15 percent lower than the 1.35 used for retrofit design. The lower portion of the beam-column joint experienced much of the observed damage with concrete spalling leading to main column reinforcement buckling at a ductility of 9. At an ultimate global displacement ductility of 12, significant degradation of lateral load (see hysteresis loops in Figure 3) occurred during cycling as column bars fractured.

The beam-column joint region of specimen OSB3 was expected to be adequate due to its increased dimensions. Calculation of joint shear capacity, based on limiting the principal concrete tensile stress, supported this assumption. During testing, however, the joint suffered large diagonal cracks, likely caused by strain migration into the joint and debonding of joint reinforcement.

The strut and tie model appeared to give conservative capacities for retrofit specimen OSB3 because the cap beam did not sustain significant damage from the test This was presumably the result of the overestimated column hinge zone overstrength factor used in design.

OSB4

OSB4 test results, shown in Figure 4, were similar to those of OSB3 (underlay scheme). While joint performance was improved in the region above the fillet, the ultimate performance of the region immediately above the steel jacket was virtually identical in the two tests. The gravity loads were reliably resisted through the extreme limits of the test, such that even at ductility demands of six times the design values, collapse would not be expected.

INFLUENCE ON RETROFIT DESIGN

Observation of the hysteresis loops accompanying the figures confirms that all of the retrofit specimens performed well with ductility capacities well in excess of that expected during the design seismic event. Retrofit specimen OSB4 was the favoured scheme and will be used to retrofit many of the approach piers on the Oak Street Bridge.

Retrofit scheme OSB4 has two important advantages over OSB2: the addition of stirrups via vertical post-tensioning produced a more robust cap beam retrofit than achievable with longitudinal post-tensioning alone, and longitudinal coring tolerances are less critical for the single hole of OSB4 rather than the two holes of OSB2, mainly because of the differences in anchorage details. The main advantages of the OSB4 scheme over OSB3 are that it is more economical with improved constructability.

The traditional approach to calculating cap beam shear strength proved useful for assessing the as-built cap beam and functioned well as a guide to the retrofit cap beam capacities. However the MCFT was thought to be more reliable for estimating the capacities of specimens OSB2 and OSB4 and will be the primary method used to design the retrofits for the approach pier cap beams.

CONCLUSIONS

The seriousness of the cap beam reinforcing deficiencies identified during the seismic assessment phase of the Oak Street Bridge were confirmed and their effect on the global seismic response of the piers was demonstrated. Inherent ductility reserves in the as-built column details proved to be reliable, and the cost benefits of omitting column jackets on a substantial number of pier bents can therefore be achieved without loss of confidence in the seismic performance of the bridge. Effective and reliable retrofit solutions were devised and proved, giving a high degree of confidence in the expected performance of the upcoming retrofits on the Oak Street Bridge.

Significant net cost savings will be achieved from the proof-test project. It is reasonable to assume that there will be future cost benefits and improved confidence in other major and minor bridge retrofit projects throughout British Columbia based on knowledge gained during the proof-test project.

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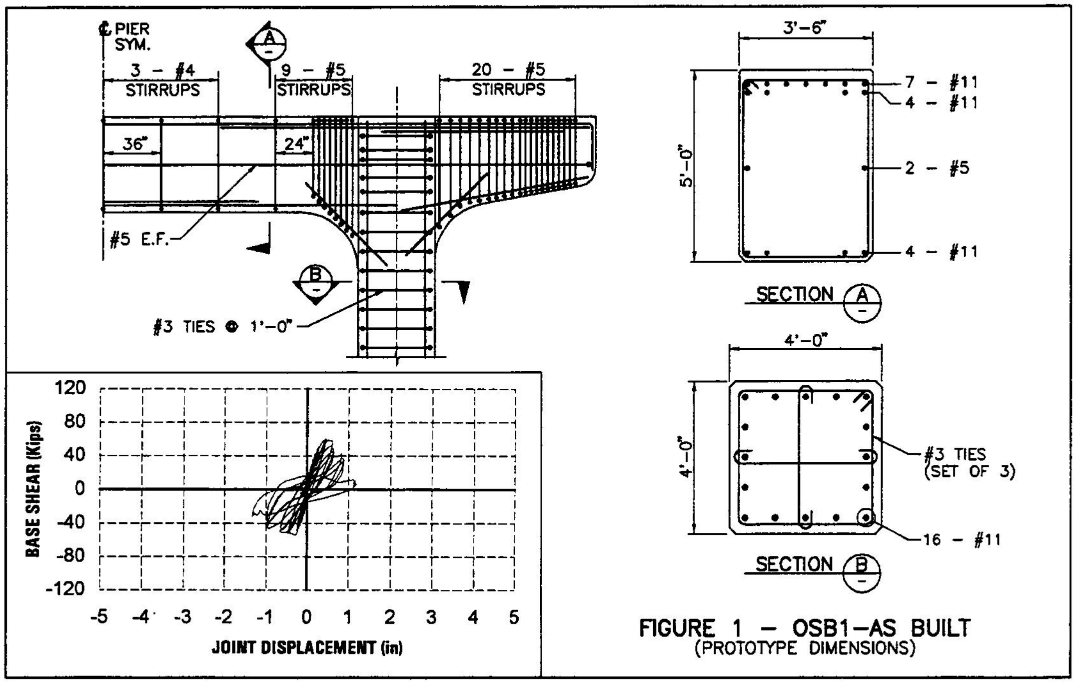

FIGURE 1 – OSB1–AS BUILT
(PROTOTYPE DIMENSIONS)

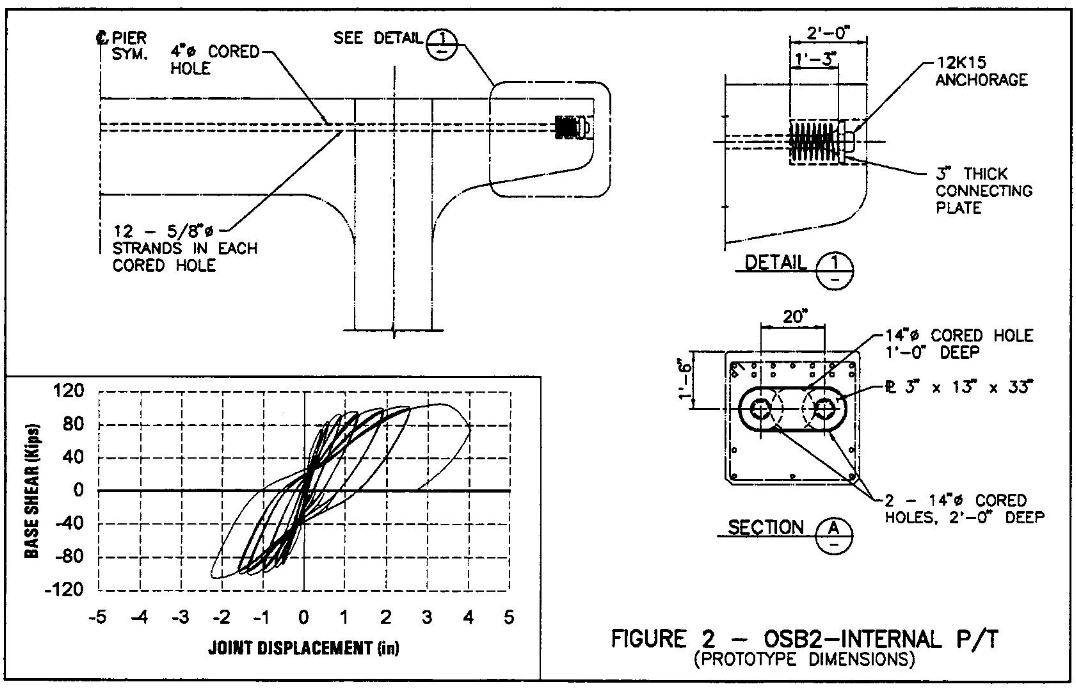

FIGURE 2 – OSB2–INTERNAL P/T
(PROTOTYPE DIMENSIONS)

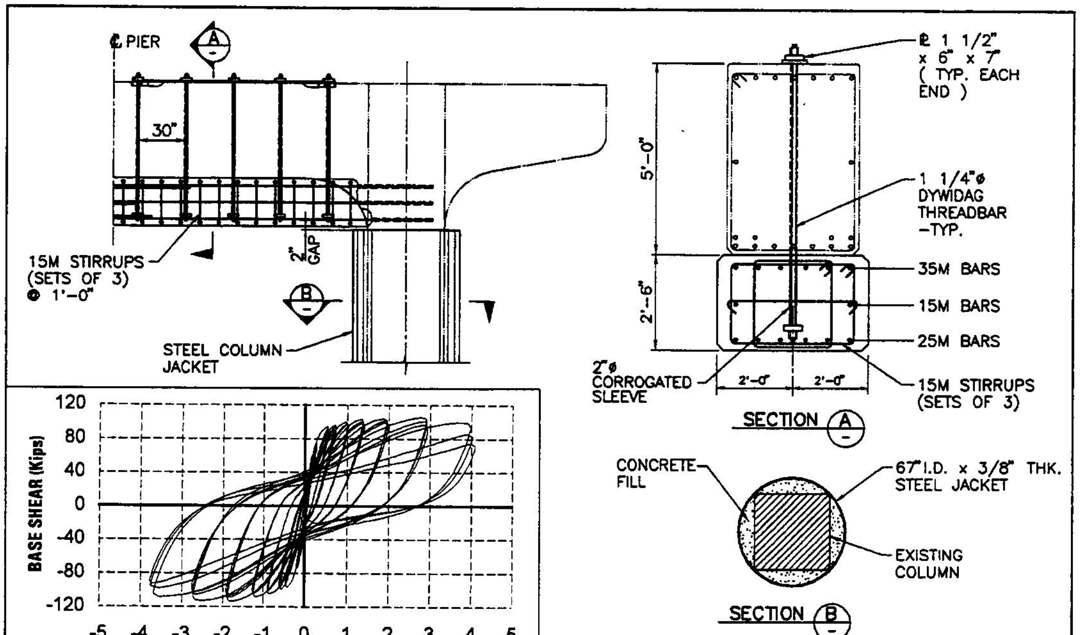

FIGURE 3 – OSB3–UNDERLAY BEAM
(PROTOTYPE DIMENSIONS)

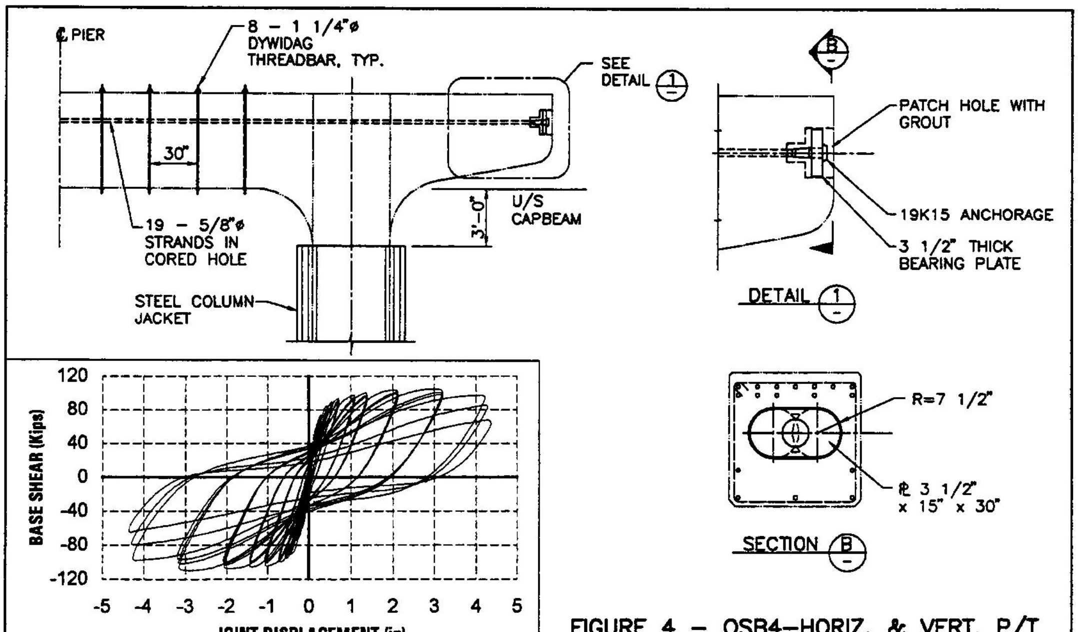

FIGURE 4 – OSB4–HORIZ. & VERT. P/T
(PROTOTYPE DIMENSIONS)