13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2705

SEISMIC RETROFIT AND REHABILITATION OF THE KING GEORGE HIGHWAY COLEBROOK ROAD/BC RAIL OVERPASS

Mike R. SCOTT[1] and Jodie C. ATKINS[2]

SUMMARY

The Colebrook Road Overpass on King George Highway was constructed by British Columbia's Ministry of Transportation (MoT) in 1974. As a result of the provincial government's devolution of infrastructure to local governments, the overpass recently became part of the City of Surrey's bridge inventory. The bridge is now 30 years old. With a remaining life expectancy of at least 20 years, and likely much longer, with timely completion of maintenance work. At this time, however, the structure is exhibiting clear signs of accelerated concrete deterioration resulting from elevated chloride ion (salt) concentrations in the deck, deck joints, and concrete components below.

In 1991, this structure was identified by the MoT as being among the highest group of bridges requiring seismic retrofitting.

To determine the appropriate level of seismic retrofitting, a seismic analysis and assessment of the seismic response has been completed for this structure. In order to maintain the capital investment, the City has chosen to proceed with the proposed work plan which encompasses both a rehabilitation and seismic retrofit.

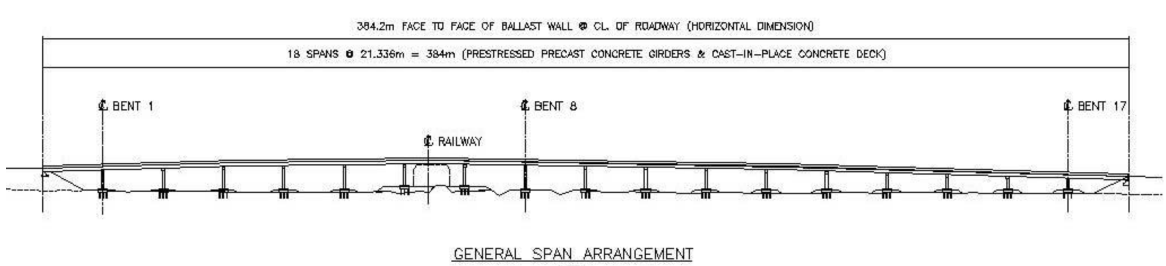

Figure 1. General Span Arrangement

[1]Structural/Bridge Engineer, Associated Engineering, Burnaby, Canada. Email: scottm@ae.ca

[2]Structural/Bridge Engineer, Associated Engineering, Kelowna, Canada. Email atkinsj@ae.ca

INTRODUCTION

The Colebrook Road Overpass is comprised of two, parallel, two-lane structures. The superstructure consists of eighteen (18) precast prestressed concrete girder spans supporting a cast-in-place concrete deck (figure 1 and 4). A total of ten girders support the deck, five carrying the northbound traffic and five carrying the southbound traffic. Each span is approximately 21.3 m long and 21.2m wide with a total structure length of 384 m. The interior substructure system consists of seventeen four-column bents (figure 2). The columns are each supported by a cast-in-place pile cap and three precast concrete 'Herkules' piles. The abutments at each end of the bridge consist of a cast-in-place concrete footing and ballast wall (figure 3 and 5).

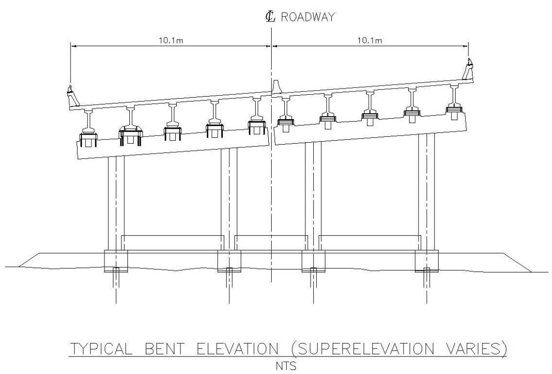

Figure 2. Typical Bent Section

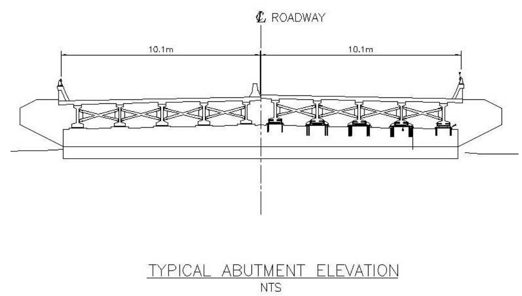

Figure 3. Abutment Elevation

Figure 4. General Arrangement

Figure 5. Typical Bent

This paper provides analysis techniques used for the seismic assessment, identifies seismic vulnerabilities, and presents retrofit strategies. An overview is provided on the combination of seismic and serviceability retrofit items for a cost effective end solution.

SEISMIC ASSESSMENT

The impetus to this project was the clear need for a major serviceability rehabilitation. To determine the seismic vulnerabilities that needed to be addressed in conjunction with the rehabilitation a detailed seismic assessment was first completed. The method used for the seismic assessment of the King George Highway/ Colebrook Road BC Rail Overpass was displacement-based, using plastic collapse mechanism (pushover) analyses described by Priestley et. al. [1]. Independent pushover analyses were performed on three of the seventeen bents. Many of the bents are of similar geometry and selecting three for analysis provided sufficient data for a complete assessment.

Push Over Analysis - Deformation Capacity

Static frame models were created using the structural analysis program, SAP 2000. The two-dimensional models are incrementally displaced analytically, tracking the formation of plastic hinges, shear degradation, joint degradation, and plastic rotation. Plastic hinges form within the bent as the incremental displacement was increased. The bent becomes more flexible, which decreases the slope on the pushover curve. The slope continues to decrease until a plastic mechanism forms. Once a mechanism has formed the bent is pushed further until the ultimate plastic rotational capacity of the governing plastic hinge is reached. The lateral displacement at this point would be the deformation capacity if shear, joint, or other undesirable failure modes are not predicted. Another key result is that upper limits to the forces that can be developed within each pier, and therefore, that can be applied in turn to components such as bearings and footings, are obtained for individual pier assessment and subsequently for retrofit component design.

The serviceability and collapse limit states are related to inelastic rotations of plastic hinges, onset of member or joint shear failure, or other degradation mechanisms. Essentially, the pushover analyses identify the displacements that will cause each of the failure mechanisms that eventually lead to collapse. Seismic deformation demands are then compared to the deformation capacities as described above.

Global Spectral Analysis - Deformation Demand

In addition to the two-dimensional models of the individual bents, a global three-dimensional model of the entire structure was created to determine deformation demand. Within SAP2000 the design earthquake is applied to the model to determine the displacement demands that the structure will experience during the seismic event. The design earthquake's spectral curve was computed using importance factors, soil conditions and foundation factors provided in CAN/CSA S6-00 [2]. Using the spectral curve a multi-modal elastic analysis was performed. Cracked section properties, foundation and bearing flexibilities were included. Comparing the displacement demands from the global model to the local displacement capacities determined using the pushover analyses and assessment procedures by Priestley, et.al. [1] provides an indication of the behaviour the bents are likely to experience during an earthquake.

At present the northbound and southbound superstructures are independent of each other. However, as part of the rehabilitation and seismic retrofit work we envisage installing a concrete overlay over both structures, which would tie each span and the northbound and southbound structures together. This work, with an increase of 100 mm in deck thickness, has been incorporated in the global model.

Section Properties

Gross section properties are used for the superstructure members. The stiffness of the capbeams, columns, tie beams, and pile caps are based on cracked section properties, 0.45 I_{gross} for these bents [after Priestley, 1].

Expansion Joints

A check of seismic seat length requirements at piers and abutments was completed prior to development of the global model. This check deemed the seat length insufficient for seismic demands. Based on anticipated deck joint rehabilitation schemes, it was decided to model the structure as if concrete link slabs would be installed, making the deck continuous over most internal supports. With this feature modelled, the seismic assessment results could be used to develop retrofit details without re-analysing the bridge. The global model was therefore created without expansion joints. Intermediate thermal movements will be accommodated by a combination of column flexibility and bearing articulation. At abutments the thermal expansion and contraction capability will be provided by strip seals designed to accommodate the increased thermal movements, including secondary transverse movements induced by the north-end plan curvature of the bridge.

Abutment Stiffness

The abutments at each end of the overpass consist of a cast-in-place footing and ballast wall. The substructure is bedded on approximately 7.6 m of approach fill. The transverse spring stiffness of the abutment is modelled by calculating the stiffness of a trapezoidal section of approach fill based on the width of the footing; as described by Wilson et. al. [2]. The longitudinal abutment spring stiffness is modelled based on the ballast wall surface area and approach fill stiffness under earthquake loads provided by Maroney et.al. [3].

In developing the model of both the longitudinal and transverse abutment stiffness the footing and ballast wall are assumed to provide adequate friction to prevent sliding relative to the surrounding fill.

Soil/Spring Stiffness

Each of the seventeen bents is supported by a series of Herkules Type 800 precast concrete piles. As-built pile logs indicate that the piles were driven to dense material which varies in depth along the length of the bridge. The piles are modelled as frame elements and supported laterally by a series of soil springs that increase in stiffness with depth.

Care had to be taken when Modelling foundation flexibility. When plastic hinging occurs in the piers, the local deformations can not be obtained directly from linear analysis. The plastic hinges increase flexibility of the pier and lower displacements translated to the piles. To estimate this, the bent lateral displacement (at the bearing elevation) is taken from the push over curve when the plastic mechanism forms. Then, the lateral pile cap displacement is calculated using relative displacements when the mechanism forms, this is assumed as the upper displacement limit of the pile cap. The actual lateral displacement demand on the bent at bearing elevation is then calculated subtracting the upper displacement limit of the pile cap from the total linear displacement demand at bearing elevation. Alternatively, foundation flexibility can be incorporated the push-over models.

Demands on additional bridge elements were assessed using a capacity design approach, in which the demands on the footings, piles, anchorages, shear keys and diaphragms were extracted from the pushover analyses.

EXISTING STRUCTURAL CONDITION

A recent detailed deck corrosion survey by Associated Engineering (2002) showed that approximately 10% of the deck was delaminated. The delaminations varied from 0% - 30% of any given span, with the majority being in the area of the deck joints. Analysis of the concrete core samples removed from the deck during that survey indicated that the chloride ion concentration at the reinforcing level was above the corrosion initiation threshold. Similar results were found in the pier caps tested (figure 6). The bridge has deck joints (compression seals) over every support, and the continued leakage of these joints, despite past efforts to repair them, is the main reason for the relatively advanced deterioration now seen.

SEISMIC AND SERVICEABILITY RETROFIT ITEMS

Various work items are planned to be constructed in stages to suit funding availability over two or more years. The main items and their staging is outlined in the following sections.

Deck Joints and Concrete Deck Overlay

The bridge superstructure consists of eighteen individual simple spans. Each span has a fixed joint at one end and an expansion joint at the other (figure 6 and 7). This type of articulation allowed the original designs to accommodate temperature expansion/contraction forces and deformations at each pier. However, it also requires a deck joint above each bent and at each abutment. The deck joints have deteriorated and become a major maintenance and repair problem. All deck joints are currently leaking, allowing water ingress onto substructure elements below.

In addition, the north bound and south bound lanes are carried by two closely spaced but separate structures. There is a 16 mm clear opening between the two inside deck edges, with the top of the joint filled with a neoprene seal. A continuous precast barrier sits along the longitudinal joint providing a barrier between the north bound and south bound traffic. Water ingress along the longitudinal joint has caused deterioration to the underside of the deck resulting in concrete spalling and reinforcing steel corroding is some areas. At the pier locations the water leakage through the longitudinal joint compounds the problem created by the leaking transverse deck joints.

The deck will be milled down to the top reinforcing mat and additional deteriorated concrete will be sub-excavated. The deck will then be overlayed with unreinforced concrete over the entire deck surface. Prior to this, the existing transverse deck joints will be removed and deteriorated concrete will be excavated to facilitate installation of new longitudinal reinforcement, forming link slabs and making the deck structurally continuous. This will allow a milling machine to run continuously along the length of the bridge and lower construction costs of the full overlay. The parapet joints will also be sealed to prevent water ingress caused by pooling at the base of the barriers. The continuous deck overlay provides a long-term solution to a problem that has caused repeated cycles of concrete patching and further deterioration.

Connecting all eighteen spans amplifies thermal forces and deformations at the ends of the bridge. These deformations were modelled and analysed to determine if elimination of all deck joints is feasible. For these analyses, a three-dimensional SAP2000 frame model was developed. Soil stiffness was replicated using soil spring supports at fixed intervals along the piles. It was found that with the installation of high-range cellular strip seals at the abutments all intermediate deck joints can be overlayed.

The concrete overlay and link slabs at existing deck joint locations will also improve seismic performance of the bridge. The continuous deck addresses loss of span concerns and eliminates pounding of adjacent spans. The transverse stiffness of the entire structure is increased, increasing redundancy and allowing a significant portion of the seismic loads to resisted at abutments.

Figure 6. Subdeck at Joint Location

Figure 7. Delaminations at Deck Joint

Cap Beams

The cap beam deterioration (figure 8) caused by chloride-laden water leaking through the deck has been a problem on this structure for a number of years. Attempts to patch the concrete have failed as the joints continued to leak and further deteriorate the cap beam concrete. The new continuous concrete deck overlay will halt water ingress to piers, and help in preventing the cap beams from deteriorating further. Cap beam rehabilitation is also needed as noted below.

Figure 8. Cap Beam Deterioration

Figure 9. Cap Beam Delaminations

In some cases the ends of the cap beams have lost significant amounts of concrete due to corrosion of the reinforcing steel and subsequent concrete spalling. Excavation of the concrete must be extended to below the top mat of reinforcing steel, and the steel cleaned and coated prior to the addition of new concrete. The cap beams do not require seismic upgrade in section, however, the seismic assessment exposed the existing shear keys as a seismic vulnerability. The shear keys will be easily reconstructed when the repairs are made to the cap beams.

Bearing Replacement

A number of the plain rubber pads have completely fallen out or shifted significantly and need to be replaced. In addition, the re-articulation of the structure due to the continuous deck overlay will require redesigned bearings to meet the shear and rotational requirements of the new system at some piers. The new bearings will consist of laminated rubber pads (plain rubber with embedded steel plates) and having an increased height of approximately 100 mm compared to the current bearings. This minor increase in bridge height will require minor adjustments to be made in the pavement of the approach roadway.

Installation of the new bearings is expected to follow the installation of the link slabs. Link slabs will not be constructed at bents 4 and 14 until bearing replacement is complete. This prevents existing bearings from facing increased thermal movements. This staging will require simultaneous jacking of a number of girders. Despite this complication, eliminating the existing deck joints with the new deck overlay is considered to be the appropriate first stage in the rehabilitation/retrofit of this structure.

Abutments

The elimination of the transverse deck joints increases thermal movements at the abutments. A high range multi-cellular strip seal will be installed between the end of the deck and abutment backwall to accommodate increased movements. Presently, the space between the back face of the girders and back wall varies between 60-125 mm. The space required for amplified movements is a minimum of 70 mm. New bearings at abutment locations to accommodate these movements will require an increase in height of approximately 75 - 100 mm. To achieve these increases, the existing backwall location and height will be modified.

Shear Key Capacity
The seismic assessment of the shear keys indicated that the odd-numbered bents, and Bent 2 and 16 require additional shear key capacity. Shear keys will be incorporated into the detailed design of the rehabilitation that must be performed on each cap beam. This item was identified during the seismic assessment and retrofit during cap beam rehabilitation will be cost effective.

Abutment Sliding
At the north end of the bridge the abutment was found to have insufficient frictional resistance, by a large margin, to withstand the transverse seismic loads. Additional resistance may be obtained by driving piles adjacent to the existing abutment and placing a reinforced concrete cap to tie into the abutment. A sliding abutment option was also considered, but predicted to cause excessive displacement demands in adjacent retrofitted piers.

Tie Beams
The seismic assessment of each of the three selected bents demonstrated shear failure mechanisms in the tie beams connecting the base of each column. These elements are considered to be secondary members to the overall system, however, they provide significant stiffness to the seismic response of the structure, and affect the distribution of seismic demands within each bent. Although the shear failure of these elements would not lead directly to structural collapse the consequences of allowing them to fail was examined. The assessment shows that without the stiffness provided by these elements the displacement demands would increase and cause shear failures of primary members such as columns, joints, and cap beams to occur during the design earthquake. Maintaining the structural integrity of the tie beams eliminates the need for seismic retrofit work on the cap beams and joints and significantly reduces the displacement demands calculated by the global model. In effect, these tie beams should be considered "primary" elements in the seismic resistance of this bridge.

Glass fibre reinforced polymer (GFRP) jackets to increase the shear strength of the beams is considered an appropriate, economic, and constructible solution. Each tie beam is accessible at grade and will require minimal excavation for full exposure. The beams can be, jacketed around their full perimeter and the jacket can be painted to satisfy aesthetic requirements and provide protection from UV deterioration.

CONCLUSIONS

The Colebrook Road Overpass in Surrey, British Columbia, Canada provides a good platform for the presentation of integrating seismic retrofit measures with serviceability and general maintenance items. The proposed retrofit scheme provides the City with good value as repairs are done with seismic benefits at a fraction of the cost of completing them separately. Some seismic retrofit details can actually lower future maintenance costs. For example, the installation of the link slabs at the deck joint locations. The bridge performs much better seismically, the future maintenance is eliminated on the joints, on the cap beams and columns, and driver comfort is increased.

Seismic assessments of existing bridges in poor condition is often a good idea prior to developing any serviceability retrofits, as many details can improve both cases. With the seismic assessment complete and seismic vulnerabilities exposed. The repairs can be made with both retrofits in mind, whether it be strengthening of existing cap beams for seismic loads during the concrete repairs of the caps, or the elimination of deck joints. Many details are beneficial to both.

Using the techniques outlined in Priestley et al. [1]. A deformation based retrofit can be obtained, rather than the traditional force based retrofit. Force based retrofits result in increasing the strength and cross section of member, which means increased section stiffness and increased loading during seismic activity. Using the flexibility and plastic hinge theories provides an accurate assessment and of deformation demands and deformation capacity can be obtained. Carefully modelling the structure and the flexibility of the foundation becomes important and deformation demands must be accurately accounted for.

ACKNOWLEDGMENTS

The authors wish to acknowledge the bridge owner, the City of Surrey, Project Supervisor, Pat Zoerb for his contributions to the project described in the paper.

REFERENCES

1. Priestley N, Seible F, Calvi GM. "Seismic Design and Retrofit of Bridges" John Wiley and Sons, 1996
2. Wilson JC, Tan BS. "Bridge Abutments: Formulation of a Simple Model for Earthquake Response Analysis" ASCE Engineering Mech. Journal, (August) 1990
3. Maroney BH, Chai YH. "Bridge Abutment Stiffness and Strength Under Earthquake Loadings"