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SEISMIC ASSESSMENT AND RETROFIT OF THE KNIGHT STREET BRIDGE, VANCOUVER, BC.

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ABSTRACT

The existing 1.5 km-long Knight Street Bridge (KSB), constructed in 1974, spans the Fraser River and connects the City of Vancouver on the north to the City of Richmond on the south of the river. The site is located in Seismic Zone 4, with a moderate to high level of seismicity. Associated Engineering (BC) Ltd. carried out a seismic assessment and retrofit design for this major river crossing. A two-level assessment and design criteria was specified, with functional and collapse prevention criteria corresponding to the 475 and 1000-year return period seismic demands. Seismic deficiencies included cap beam shear, shear keys at concrete girder approach spans, column-cap beam joint shear, pile cap shear and flexural deficiencies at timber-pile supported piers, and footing shear at piers on spread footings. Particular challenges were presented by unique as-built bridge details such as large column-cap beam joints, the curtailment of the column bars within the mid-height of the joint region, very light volumetric ratios of transverse ties and longitudinal reinforcing, and the existence of lap-spliced column ties in the columns at potential plastic hinges. Seismic retrofit measures and several critical rehabilitation deficiencies were simultaneously addressed in our design, such that immediate value was achieved by the owner (TransLink) over and above pure seismic risk mitigation.

Introduction

The Knight Street Bridge is one of the nine major bridges in the Metro Vancouver area. This crossing over the Fraser River was completed in 1974, and connects the City of Vancouver on the north, to the City of Richmond on the south of the river. It also provides access to Mitchell Island located between the two river channels. The bridge typically comprises four lanes, flared to six locally, two sidewalks, and is approximately 1500 m long. It includes three distinct bridges: the South Channel Crossing, the North Channel Crossing, and the Marine Drive Overpass. It is one of the most important crossings within the Lower Mainland, carrying over 100,000 vehicles each day. The Knight Street Bridge was considered as a “Lifeline Bridge” within the context of the Canadian Highway Bridge Design Code (CAN-CSA/S6-00) due to its importance in the Metro Vancouver transportation network and the traffic volume it carries. It is

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located in a seismic Zone 4, which is a moderate to high level of seismic risk. The bridge was devolved by the province to TransLink, which also provided the money for retrofitting this important structure.

A two-level assessment and retrofit design for this facility was specified as follows:

- Improve the bridge so that, after a design earthquake (475-year return period), it will perform at a functional level.

- Improve the bridge's ultimate seismic performance to at least a minimal level to increase the likelihood the bridge will survive a large earthquake (1000-year return period) without collapse.

Our performance assessments make specific reference to these performance levels. In their essence, the two performance levels are summarized for the purpose of our retrofit strategy development as:

- Following the 475-year event the bridge can be used by some traffic, including the public.

- Following the 1000-year event, the crossing need not be passable, but damage is not to increase the risk of collapse of the bridge.

A deformation based seismic assessment was carried out for this structure. Global, segmental and local models were produced to carry out the various assessments. Local models were used to develop equivalent linear properties and carry out non-linear pushover analyses. These were employed to assist in our performance evaluation of component demands at various displacement levels. Moment-curvature demands were determined for each potential plastic hinge, and shear demands were evaluated based on pushover analyses. Component capacities were determined in relation to the inelastic demands at each applicable location. Similarly, beam-column joint shear stress demands and capacities were assessed carefully. The effects of tension-shift in cap beams and columns were also considered due to rebar terminations. In addition, footings and column-footing joints were assessed.

Seismicity levels and performance requirements were based on project specific criteria in conjunction with the Canadian highway bridge design code (CSA-S6-06) for this important structure.

Figure 1: River Span Elevation

Seismic Demands

Response spectrum analyses were performed on linear local models for piers N4, S3, S8, S16, S24 and S30. Uniform hazard response spectra were used corresponding to the 1 in 475 year and 1 in 1000 year events.

Linear segmental models were developed for piers S22 through S26 and the south river span. In each response spectrum analysis of the local pier models, the numbers of modes considered were such that over 90% of the modal mass participated in the response. Peak modal forces and displacements were combined using the Complete Quadratic Combination (CQC) method. Demands from longitudinal and transverse directions were combined using the Square-Root-sum-Square (SRSS) method combination approach when biaxial demands were deemed to be significant. All response spectra analyses used a system damping of 5%.

Pushover Analyses:

Pushover analyses were carried out for the five piers mentioned above. These were performed using two different computer analysis packages, namely SAP 2000 and ADINA. The SAP 2000 package was used to carry out the pushovers using both the linear and non-linear versions. Non-linear analyses using the SAP 2000 linear version utilized a set of linear models for each pier with the non-linearity represented by hinges, rotational springs, and imposed moments. The SAP non-linear version and ADINA utilized hinge elements and a displacement controlled solution scheme to represent the plastic hinge regions automatically. An excellent correlation was fond between the solutions obtained from SAP 2000 linear and non-linear versions.

Assessment Approach

For each of the bridge elements analyzed, a detailed performance evaluation was completed. The evaluations considered member strength, displacement and ductility demands.

Displacement:

Our basic assessment approach was displacement based. For each bridge component the anticipated deformation demands were calculated and the effects of the deformations on each member were considered. The imposition of the deformations on the bridge structure is what generates the strength and component ductility demands.

Bending and Curvature:

The displacement of the bridge structure generates bending moments in the structural components comprising the lateral load resisting system. Moment-curvature (M-C) plots were generated using the EXTRACT software package and checked using first principles and a strain-compatibility approach. By plotting the displacement demands on the on the generated pushover curves the flexural demand for each member can be assessed. The flexural demands comprise elastic demands and plastic demand if plastic hinging is predicted to occur.

For each plastic hinge location, the curvature demands were assessed. The curvatures in these locations are directly related to the strains in concrete and rebar. Due to the deficient detailing of the column ties, we designated the occurrence of failure corresponding to "general", meaning significant spalling in cover concrete. The acceptable concrete cover compressive strain was limited to a value of 0.005 for the 1000 year demands. Our assessment was based on curvatures and strains rather than rotational ductilities for clarity and ease of interpreting various behaviors.

Shear:

Shear demands were calculated based on the capacity design principles. Shear capacities are dependent on curvature demands at each location and were determined for each demand level.

Joint Shear:

Joint shears are generated when bending is transferred through a continuous joint in reinforced concrete. Joint shears generate tensile stresses in concrete resulting in cracking of the joint and are difficult to assess in some as-built cases. Equivalent force transfer mechanisms have been developed which correlate with test data and past performance of concrete bridge structures. However, the theoretical models typically assume a reinforcement to be continuous through the height and width of the joint. In the Knight Street Bridge piers, the column reinforcement is terminated near mid-height of the joints resulting in an ill-conditioned mechanism. The cap beam-column joints were assessed for joint shear as per Priestly, Seible and Calvi (1996). Three models were employed for this assessment:

- The first model assumed the column bars extend through the joint (simplest, least conservative)
- The second model assumed a reduced joint height to suite the length of the column bars (more complex, may be conservative)
- The last model investigated whether the column bars have sufficient embedment to resist a cone-pullout failure mode.

Footings:

The footings for each assessed pier were investigated for stability, bending capacity and shear capacity. Effective pile cap widths were determined as per Priestly, Seible and Calvi (1996). In the footing assessment, we assumed that no tension could be generated in the timber piles. Footing-column joints were also evaluated as part of the assessment.

Evaluation Results and Engineering Judgments for Seismic Behavior of Critical Components

This section discusses the results of our engineering assessments and judgments on the expected behavior of key bridge components, the implications relative to the seismic performance criteria, and the considerations that factored into our recommendations and design of seismic retrofits.

Beam-column Joints:

Beam-column joints are essential to the ability of piers to sustain lateral loads. Typical pier cap beams are 600 mm wider than the columns, such that the beam-column joints are relatively wide. However, the reinforcing bar details within the joint is poor for sustaining seismic demands. Column bars curtail within the joint between 900 mm to 1200 mm above the beam soffit. No column ties are present within the joints, and beam ties in joint areas only comprise two legs of stirrups on the outer faces. The curtailment of column rebar within the mid-height of the joints coupled with lack of distributed rebar in the joint areas is likely to lead to very poor crack distribution. There is thus a likelihood of formation of a single major crack within the joint area under high seismic demands. It is also possible however, that the column bars will lose their development capability prior to the formation of a major joint crack. Either of these failure modes implies the loss of a pier to reliably carry sustained cyclic lateral loads. Hysteresis loops of as-built bridge piers having pre-1980's reinforcing details have shown extremely poor performance with sudden shear failures. In comparing the assessed performance of the joint region to the performance requirements, the requirement that '…damage is not to increase the risk of collapse' was considered not to be met by the joint performance as described above

Cap Beams and Columns:

Pier cap beams and columns were assessed based on the Seismic Retrofit Design Criteria developed specifically for this project. Considerable effort went into the formulation of the assessment criteria to apply to these elements. The key aspect of the criteria was in demonstrating acceptable extreme fiber compressive strains and curvatures for the columns. Of concern were the very light volumetric ratios of transverse ties and the existence of lap spliced column ties. Consequently, the lap splices would open after cover spalling resulting in loss of concrete and steel tie contribution to shear resistance, accelerated core damage and column failure. However, in general, our assessment showed surprisingly little column retrofit is required.

Similarly, pier cap beams were assessed in accordance with criteria adopted specifically for this structure and pertinent details. Beam ties were found to be considerably more generous than column ties, and contain 135^0 hooks at the corners. As such the consequences of flexural hinging and increasing compressive strains were less for cap beams than for columns, and few beams appeared to require retrofit. The effects of 'tension shift' effects in cap beams were assessed based on hand calculations using the moment demand gradients and from the pushover analysis, and section strength changes due to rebar terminations. Tension shift effects did not show migration of plastic hinges away from the joints for the piers assessed as part of this evaluation.

Footings and Pile Caps:

Pile cap shear and flexural failures were predicted at timber pile-supported piers (S2 through S8, S13 through S30). Fracture of a pile cap might not directly compromise gravity load-carrying capability. However, it would result in uninspectable damage to the pile cap, would tend to increase liquefaction-induced settlements, and compromise the bridge's seismic resistance for large earthquakes and during the period of potential aftershocks.

Footing shear failures at piers on spread footings on till (N6, N5, N4) were also predicted. Fracture of these footings would cause a reduction in pier lateral stiffness, but would not necessarily imply either the 475 year or 1000-year objectives would not be satisfied.

Bearing and Lateral Load Demands:

The existing shear keys on the top of cap beams were determined to be weak and lacking in sufficient capacity to transfer seismic shear to force a mechanism in the piers. Shear key failures were predicted at all concrete girder approach spans. In the as-built condition, shear transfer between the piers would take place through friction between the rubber bearings and the concrete elements. Our assessment showed that the required coefficient of friction to cause a mechanism in the piers was substantial (in excess of 0.8 in some cases).Although a significant shear transfer deficiency would not indicate collapse but could prevent the use of the bridge following the 475-year event. A more reliable behavior would be expected from a yielding pier than in one with intermittent bearing slip.

Geotechnical Considerations:

The existing soil conditions were assessed including liquefaction potential and lateral spreading. The geotechnical aspects of the project were completed by others and are not discussed in detail in the paper.

Seismic Retrofits

In order to address the vulnerabilities identified several retrofit strategies were considered. Significant considerations in selecting the preferred retrofit strategy included effectiveness in achieving performance criteria, cost, and opportunities to include rehabilitation works within the required retrofit works. Significant effort was directed balancing the benefits of structural retrofits with ground improvements. The selected retrofit strategy included addresses the vulnerabilities identified above and includes the following structural retrofits:

- Pier cap strengthening, column weakening.
- Concrete shear keys.
- Concrete pier joint retrofit.
- Pile cap footing overlays at approach piers.
- Deck joint link slabs.
- Longitudinal restrainers at expansion joints.

The following discussion outlines the key retrofit details included in the selected retrofit strategies.

Pier Cap Strengthening, Column Weakening

In order to provide ductile behaviour and ensure column hinging Carbon FRP shear and flexural strengthening was detailed at selected piers to strengthen the pier cap beams to resist over strength column flexural demands.

At selected locations where the pier cap beams were significantly deficient against over strength column flexural demands a novel approach was adopted. At these locations, the columns were weakened in flexure by cutting four vertical reinforcing bars. The selected bars were cut at the top and bottom of the theoretical hinge zone. By locally reducing the column capacity the need for costly strengthening of the pier cap beams was eliminated.

Concrete pier joint retrofit

As discussed earlier, the existing reinforcing detailing including short curtailment of vertical column bars complicated the assessment of the pier beam column joints. Additionally the massive size of the joints made post tensioning retrofit measures inappropriate. In order to provide a reliable joint shear mechanism vertical reinforcing was installed in the cap beam to extend the column reinforcing to near the top of the joint. The additional reinforcing was installed in holes cored from the bottom and pressure grouted to provide full bond. In an attempt to lower the joint shear stresses concrete fillets were added to the soffit of the cap beam to increase the effective joint depth and width. Lastly, carbon FRP reinforcing was installed to resist joint shear tensions. The FRP was oriented on the bias to act efficiently in the principle tension direction.

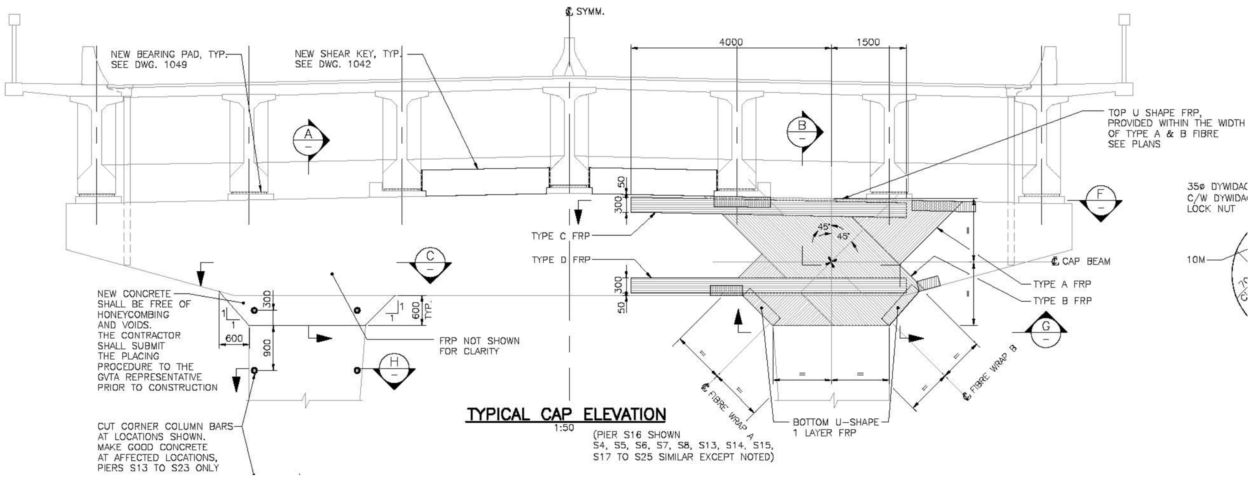

Figure 2 - Typical Joint Retrofit

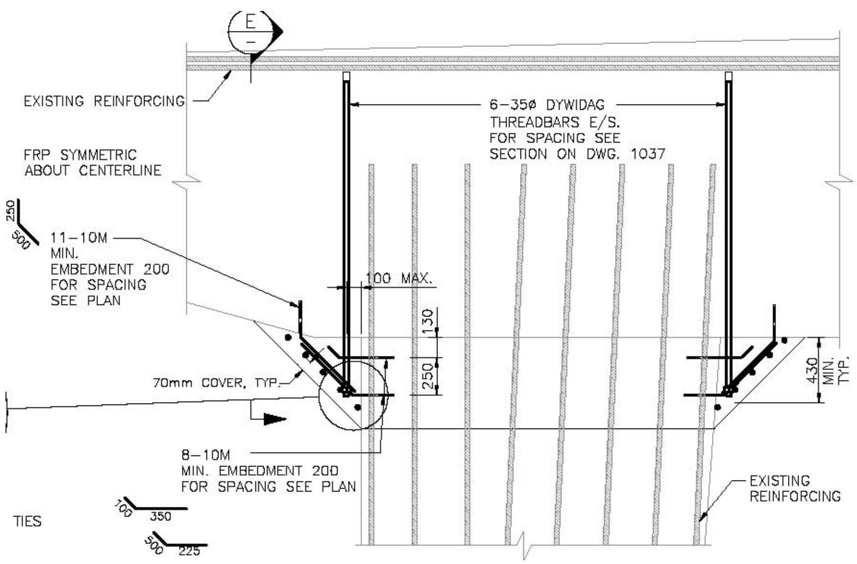

Figure 3 - Joint Retrofit Reinforcing

Deck Joint Link Slabs

Where ever possible retrofit solutions which provide multiple benefits preferred. A good example of this was the replacement of many deck joints with link slabs. The link slabs included in the retrofit served three purposes. Primarily they create a contiguous deck diaphragm across adjacent bridge spans. Secondly they act as longitudinal restrains preventing unseating the approach span girders. Lastly, they provide durability and maintenance benefits. Deck joints require frequent maintenance and leaking deck joints are a primary contributor to substructure deterioration. By replacing the joints with link slabs future joint maintenance is eliminated and substructure deterioration due to water ingress is minimized.

Consideration was given to a variety of link slab configurations, from replacing every approach span expansion joint to replacing every second joint. Replacing all of the joints would have meant a significant re-articulation of the bridge and replacement of all of the bridge bearings, a costly exercise. Ultimately the solution was to typically replace two of every three approach span expansion joints with link slabs. This configuration did not require significant bearing replacement.

Conclusion

The Knight Street Bridge, constructed in 1974, a 1.5 km long is a critical link in the Vancouver regional transportation network located with in a high seismic zone.. A two-level assessment and design criteria was specified, with functional and collapse prevention criteria corresponding to the 475 and 1000-year return period seismic demands. The detailed seismic assessment identified a variety of deficiencies including foundation, pier cap, beam column joints, and loss of span vulnerabilities. The retrofit strategy developed included conventional retrofit details in conjunction with unique retrofit solutions to address the identified vulnerabilities.

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