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SEISMIC AND WIND VULNERABILITY ASSESSMENT FOR THE PATTULLO BRIDGE

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Abstract: The Pattullo Bridge between New Westminster and Surrey in Metro Vancouver was designed and constructed in the mid to late 1930's, a time when the effects of earthquakes on structures received little consideration. Since the bridge opened to traffic, no major seismic rehabilitation and retrofitting has been done to date. The paper presents results of a seismic and wind vulnerability assessment with the purpose to develop an advanced warning system for the Pattullo Bridge. The assessment includes the definition of design criteria, design loads, thresholds and detailed finite element model development. Based on probabilistic seismic hazard analysis, the threshold seismic event was identified. Wind analysis was carried out based on the site-specific wind loads and turbulence criteria developed using historical data. Based on the outcome of this study, it was found that the probability of exceeding a threshold event (seismic or wind) at the bridge site within the expected remaining 10 year services life is unlikely. These findings can be implemented in the warning system development.

1 INTRODUCTION

The existing Pattullo Bridge was constructed in 1936 / 37 and connects the City of New Westminster and the City of Surrey over the Fraser River. The bridge has a total length of approximately 1240 m (4067 ft.) and consists of one steel approach span, concrete girder approach spans, steel deck truss approach spans, and a main arch truss. Although the bridge is now well beyond its intended design life, no major seismic rehabilitation and retrofitting has been done. In order to protect public safety, TransLink wanted to evaluate the feasibility of installing a Wind Monitoring and Seismic Warning System until the bridge is replaced. Before implementing the Wind Monitoring and Seismic Warning System, it is critical to evaluate the threshold seismic and wind event that will initiate the collapse of the bridge. This study is tailored to achieve the following two key purposes as listed below:

- To examine the feasibility of implementing a seismic warning system on the Pattullo Bridge;
- To examine the feasibility of implementing a wind monitoring system on the Pattullo Bridge.

Based on this assessment, an emergency response plan and warning system will be developed that would document the policies and procedures for emergency closures of the Pattullo Bridge during and following a threshold event within the next ten (10) years.

2 EXISTING BRIDGE

The Pattullo Bridge consists of two concrete spans at the north end of the bridge, nine steel truss spans crossing the Fraser River, including the 3-span continuous main span, 20 concrete spans south of the river and three short steel spans at the south end of the bridge. Each of these components are structurally unique, and, as a result the bridge has been broken down into the following main structural parts for vulnerability assessment:

- North Approach Concrete Spans
- Steel Approach Span Trusses
- Steel Main Truss Spans
- South Approach Concrete Spans

A description of each of these components / elements is provided in the following subsections.

2.1 North Approach Concrete Spans

The North Approach Spans crossing Columbia Street consist of two simply supported cast-in-place concrete spans of 21.98 m and 19.37 m (total length about 45.2 m). The 229 mm (9”) thick concrete deck is supported by traditionally reinforced concrete girders. To account for the flare of the bridge deck approaching the abutment, girder lines increase from 7 to 11 in the two end spans. The deck, girders and cantilevering cross beams are connected monolithically by diaphragms. The North Abutment soil embankment is retained by concrete wingwall structures with spread footings and counterforts on the earth-facing side of the back wall. The common support for the two spans consists of a reinforced concrete bent (“Bent A”) with four tapered columns and intermediate shear walls between the columns. The North Approach transitions to the Steel Deck Truss Spans at Pier 0, which is directly founded on hardpan.

2.2 Steel Approach Span Trusses

The simply supported steel deck trusses span from Pier 0 to Pier 2 and from Pier 5 to Pier 9, and are the flanking spans on either side of the continuous steel truss main span. The simply supported spans have lengths of 60.96 m (200 ft) and 76.20 m (250 ft). The span arrangement and bridge elevation is shown in ***Figure 1.1***. The total available deck width between curbs on these spans is 12.04 m. An additional 1.83 m (6 ft) wide sidewalk on the west side is incorporated into the deck. The cast-in-place concrete deck is supported by longitudinal stringers typically spaced at 1.83 m. Deck thickness varies between 178 mm (between stringers) and 229 mm above the stringers. The sidewalk has an average thickness of 127 mm. Stringers are connected to floor beams spaced at 7.48 m to 7.51 m. The floor beams rest on the top chord of the trusses. Both trusses are connected and stabilized by lateral bracing and cross bracing to form a spatial three-dimensional truss structure.

For the 60.96 m (200 ft) spans, the spacing between truss centre lines is a constant 8.53 m (28 ft.) whereas, the two 76.20 m (250 ft) truss spans function as transition spans to the wider continuous truss main spans. The transition span truss spacing increases linearly from 8.53 m to 13.56 m (44.5 ft.) along its length. In general, both trusses are different in cross sectional composition. Following the original nomenclature, the west truss (at the side of the sidewalk) is called the “Heavy Truss” and the east truss is called the “Light Truss”.

2.3 Steel Main Span Trusses

The continuous steel truss between Pier 2 and Pier 5 includes a main span of 137.16 m (450 ft) and side spans of 106.68 m (350 ft). The bridge superstructure transitions from a deck-truss system (side spans) to a through-truss system (in the main span). The general composition of the deck, stringers and floor beams is identical to those of the steel truss approach spans. The spacing of floor beams varies between 8.83 m and 9.65 m. The distance between the truss centre lines is kept constant throughout the entire main span at 13.56 m (44.5 ft.). The continuous trusses have a fixed support on Pier 4 leaving all other piers unrestrained to allow for thermal expansion.

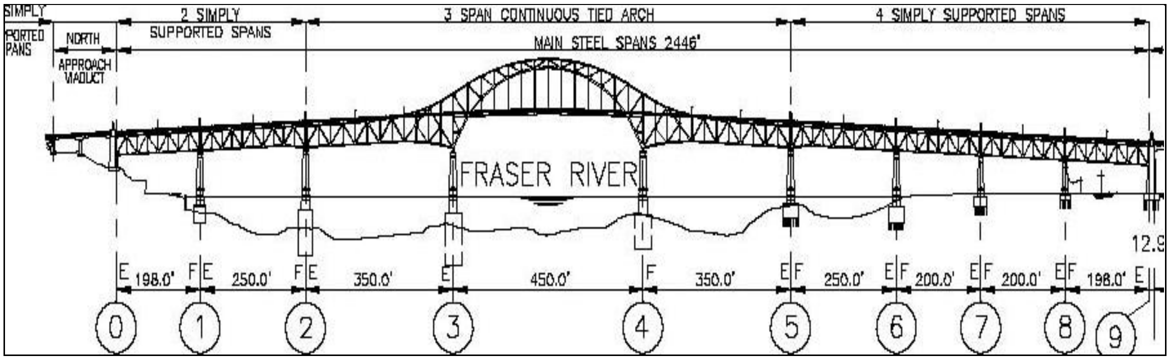

Figure 1: Pattullo Bridge – North Approach Spans and Steel Truss Spans

2.3 South Approach Concrete Spans

This segment consists of a fully cast-in-place concrete deck on three shallow simply supported haunched concrete girders. The girders are reinforced using rebar and steel trusses. Structural integrity of the superstructure is greatly improved by transverse concrete beams and end diaphragms. Spans vary between 17.9 m (58.7 ft) and 31.7 m (104 ft) in length and are designed as simply supported spans. Steel tie rods were installed between 1982 and 1988 to connect the individual spans together as part of a minor seismic retrofit. All bents (Pier 10 to Pier 29) have the same configuration: two hollow tapered columns, connected by a crosshead at the top. The columns are supported by individual pile caps on timber piles. Concrete tie beams connect the pile caps together.

The old timber spans between Pier 29 and the South Abutment were replaced in 2009 by a single span steel composite structure.

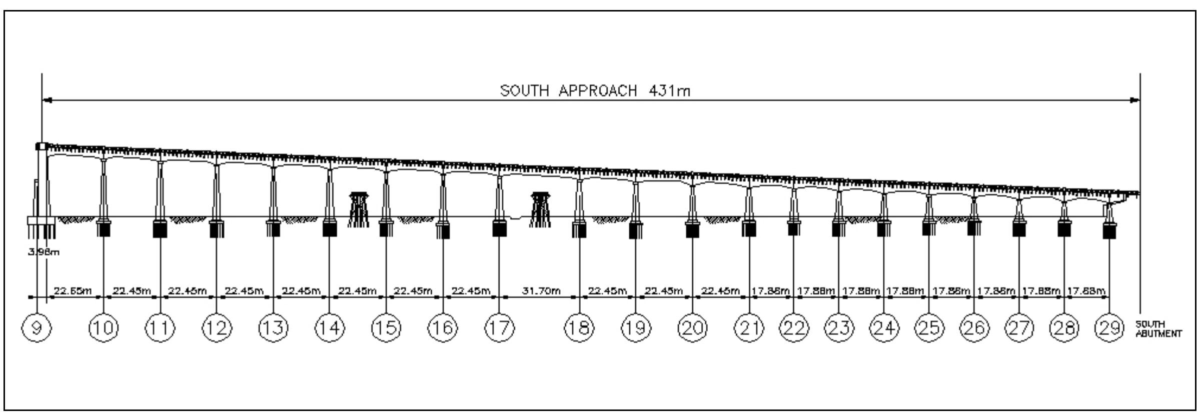

Figure 2: Pattullo Bridge – South Approach Spans

3 METHODOLOGY AND FINITE ELEMENT MODELING

The main objective of this seismic and wind assessment is to identify the specific seismic and wind event and failure mechanisms that would compromise the structural integrity of the bridge resulting in a partial or complete collapse of the bridge. The seismic assessment is carried out through a combination of Response Spectrum Analysis and Non-linear Static Pushover Analysis. A 3D finite element computer model of the Pattullo Bridge steel spans from Pier 0 to Pier 9 has been used to analyse the seismic vulnerability of the bridge. The model was developed using the finite element program SAP2000 considering soil structure interaction. The soil-structure interaction was modeled considering the foundation compliance springs provided by the geotechnical engineer (Golder, 2013).

All steel elements including truss members, deck and sidewalk stringers and floor beams are modeled as frame elements with 6 degrees of freedom at each node. Member properties are computed per built-up section details provided on the original shop drawings. The concrete columns, beams, and foundation supports including caissons and middle sections are also modeled as frame elements. The equivalent properties of all concrete elements are computed using the information provided on the available drawings. The model mass included the self-weight of all steel members, the 200 mm thick concrete deck slab which is about 14.0 m wide, the 50mm average asphalt overlay, and added mass at Piers 0 and 9 to account for the tributary area of the associated concrete approach spans. The pier concrete strength is considered as 38 MPa based on test results and concrete reinforcement steel strength fy is 230 MPa in accordance with CL.14.7.4.4 of the Canadian Highways Bridge Design Code (CHBDC - S6-14).

4 BRIDGE SEISMIC RESPONSE ASSESSMENT

The seismic response analysis was carried out to find the critical seismic event that would initiate the collapse of the bridge. Seismic analysis of the current bridge condition has been undertaken to confirm seismic events that would lead to the safety-critical conditions for the bridge. A number of seismic and structural rehabilitation assignments have been undertaken in the last ten years. Most recent, are the reports done by Associated Engineering (2012), Parsons (previously Delcan) (2013) and COWI (previously Buckland & Taylor) (2015) in which they collectively identify the various seismic vulnerabilities and thresholds. Based on the previous assessment, it has been identified that Pier 3 and Pier 4 in the main steel span are the weakest link in the bridge.

The seismic evaluation of this structure is based on the CAN/CSA-S6-14 (CHBDC-S6-14) and the BC MoTi Seismic Retrofit Design Criteria (2005). Seismic demands on the structure are determined through a combination of Response Spectrum Analysis and Non-linear Static Pushover Analysis. Response spectrum analysis was carried out using site specific response spectrum provided by the geotechnical engineer (Golder) for the Patullo Bridge site. Figure 3a shows the median uniform hazard spectrum for Pattullo Bridge for different return periods. Since the soil condition at different pier locations are quite different, the geotechnical engineer provided different response spectrum for different pier locations. The response spectrum for different pier locations corresponding to the 475 year return period is shown in Figure 3b.

Through response spectrum analysis, it was confirmed that Pier-3 is the critical link and failure would compromise the structural integrity of the bridge resulting in a partial or complete collapse of the tributary structure. Transverse and longitudinal pushover analyses was performed using SAP2000 in order to establish the collapse mechanism and the deformation capacity of Pier-3. Based on the pushover analysis, it was found that under the 475-year earthquake the collapse threshold PGA for Pier-3 is approximately 0.05g. At this level of PGA corresponding to the 475-year earthquake, Pier-3 will reach its collapse capacity thus leading to a safety-critical condition for the bridge. According to geotechnical assessment (Golder, 2013), the PGA for a 475-year earthquake at the Pattullo Bridge site is 0.25g. This indicates a D/C ratio of 5.0.

Further analyses with earthquakes of different return periods were carried out to identify the safety-critical seismic event for the bridge. It was found that, the threshold for the bridge is a 1 in 30 year seismic event. Figure 4 shows the capacity-demand spectrum for Pier-3 under the 30 year earthquake event. For this analysis, yielding of the pier is considered as the collapse critical mechanism (not ductility) due to the uncertainty associated with the material properties and reinforcement detailing. It is assumed that beyond yield point the structure is susceptible to collapse.

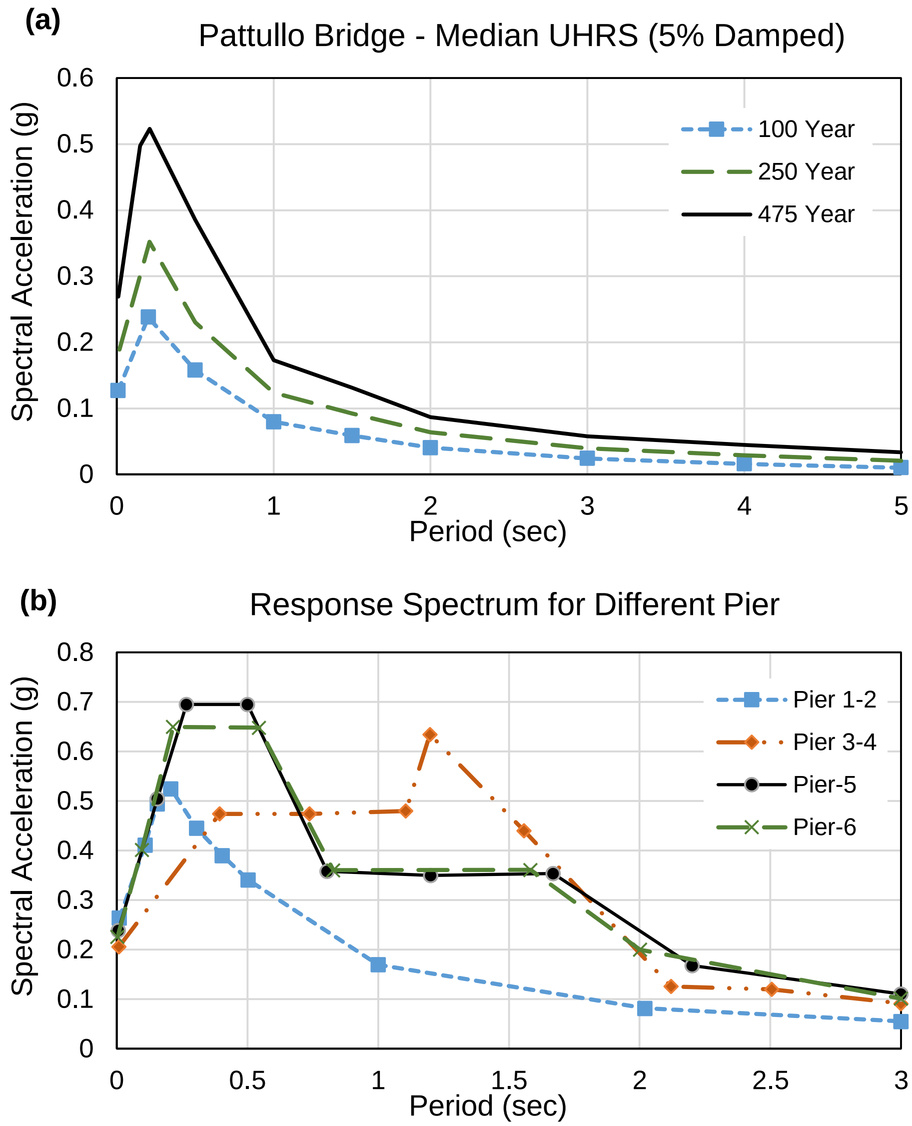

Figure 3: Pattullo Bridge Site Specific Response Spectrum

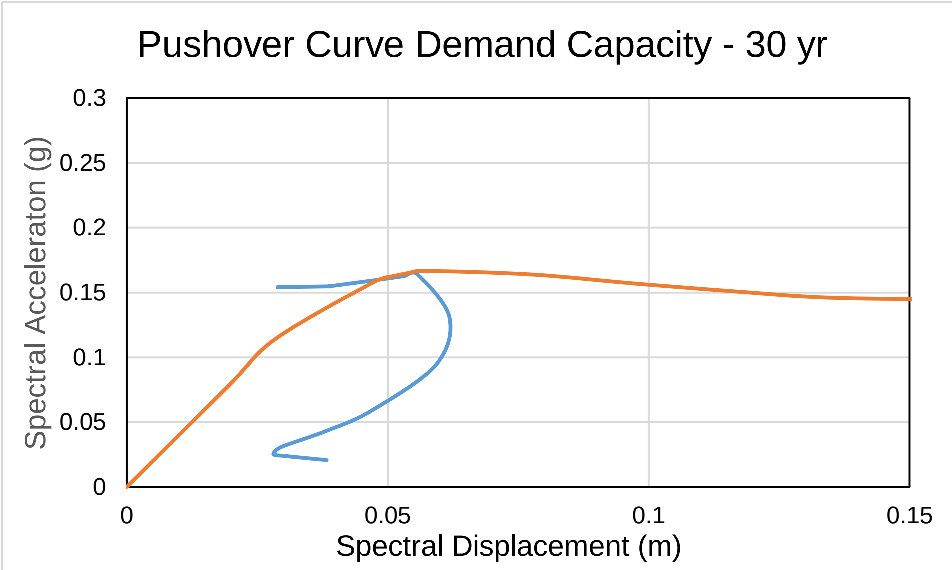

Figure 4: Capacity-demand spectrum for Pier-3

For the 30 year return period event, the collapse PGA is identified as 0.055g. For the prescribed collapse acceleration PGA of 0.055g, the hazard curve in Figure 5, provide the mean annual frequency exceedance of 0.035 corresponding to return period of 30 years (Figure 6). The probability of at least one event exceeding the prescribed collapse acceleration of the Pattullo Bridge, assuming a Poission distribution event is given by:

$$P = 1\text{-}\exp(-\lambda.t)$$

where, λ is the annual mean of occurrence and t is the time period of interest. For t equal to 50 years and λ equal to 0.035, there is an 83% probability of exceeding the collapse acceleration. For t equal to 10 years (expected time before the bridge is replaced) and λ equal to 0.035, there is a 30% probability of exceeding the collapse acceleration.

For seismic hazard analysis, it is necessary to specify how the ground motions attenuate with distance from the source of the earthquake. In this study, the attenuation relations used in the 2013 Delcan Seismic Vulnerabilty Assessment Report are used (Delcan, 2013). These attenuation relations have been developed by the Geological Survey of Canada (GSC) and are available in tabular form. These tables are based on modified forms of the Boore et al. (1997) attenuation relations for crustal earthquakes and Youngs et al. (1997) for subcrustal earthquakes. Table 1 summarizes the magnitude and distance for median collapse PGA of 0.055g at Pattullo Bridge, Site Class C and attenuation for crustal and subcrustal ground motions. The distance range over which earthquakes of different magnitudes can cause accelerations equal to or greater than the median collapse acceleration 0.055g is shown in Figures 7 and 8. Figure 7 is for crustal events and Figure 8 is for subcrustal events.

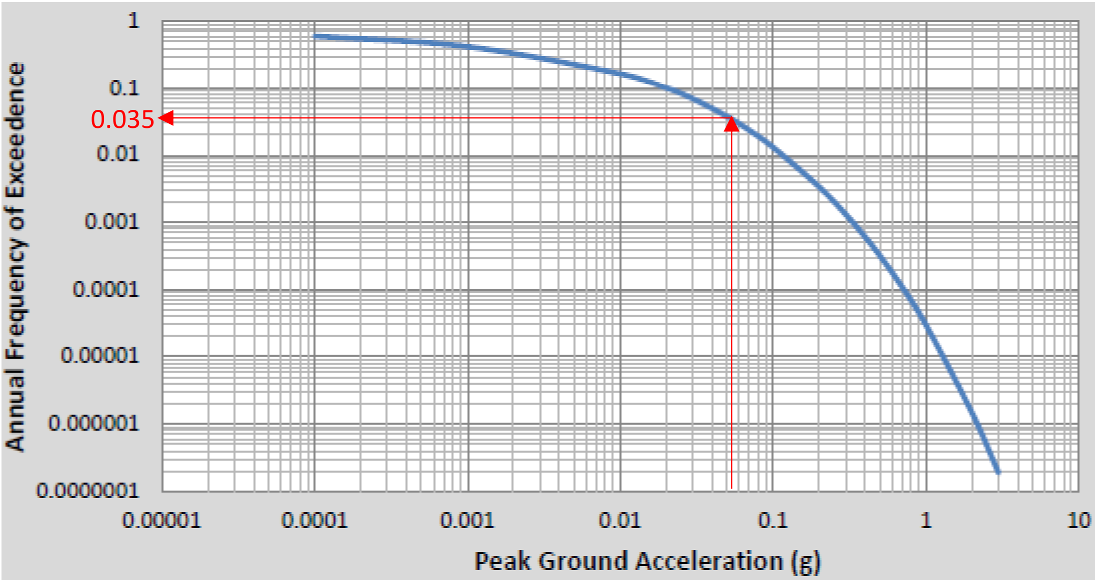

Figure 5: Seismic Hazard Curve for Site Class C at Pattullo Bridge site

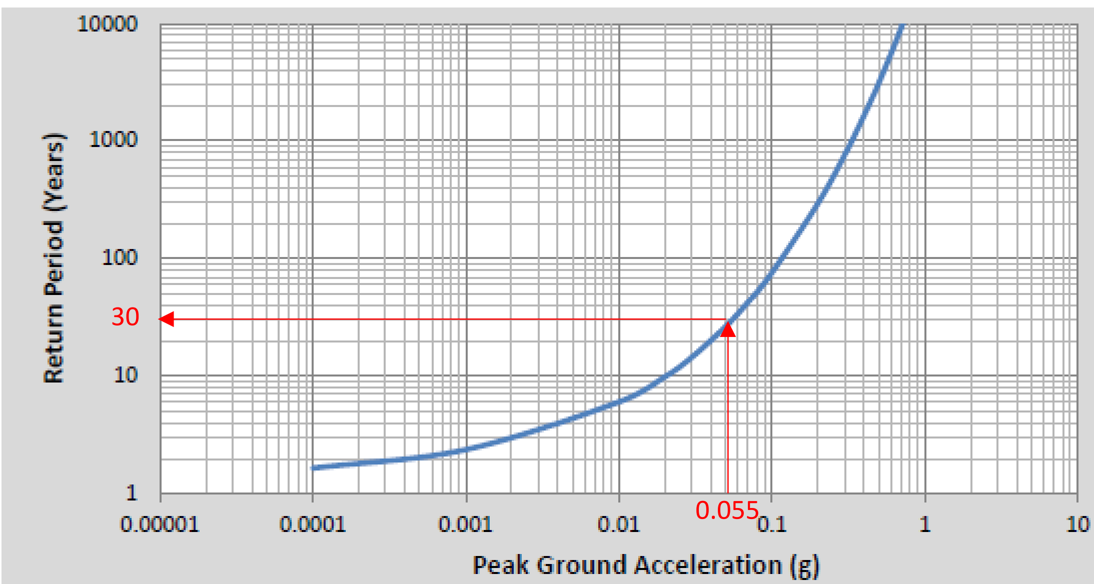

Figure 6: Return Period for Site Class C at Pattullo Bridge site

Table 1: Magnitude-site distance relationship for Pattullo Bridge for crustal and subcrustal ground motions

	Distance, R (km)	
Magnitude, M	Crustal EQ	Subcrustal EQ
5	30	10
6	58	76
7	107	140
8	142	244

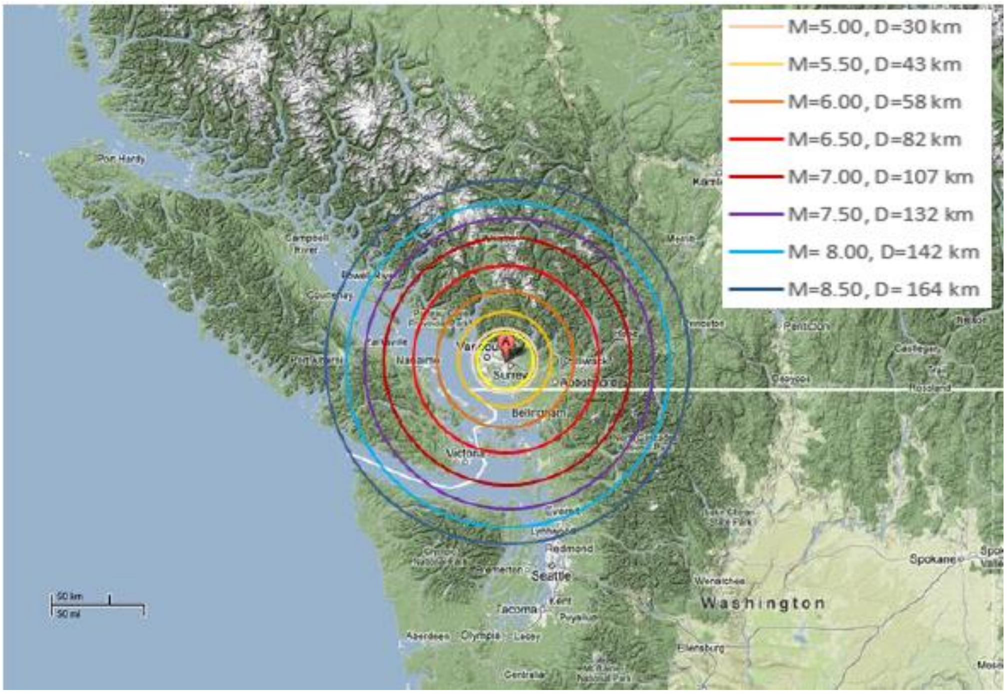

Figure 7: Magnitude and Distance for median collapse PGA = 0.055g at Pattullo Bridge, Site Class C and GSC modified Boore et al. [6] attenuation for crustal ground motions

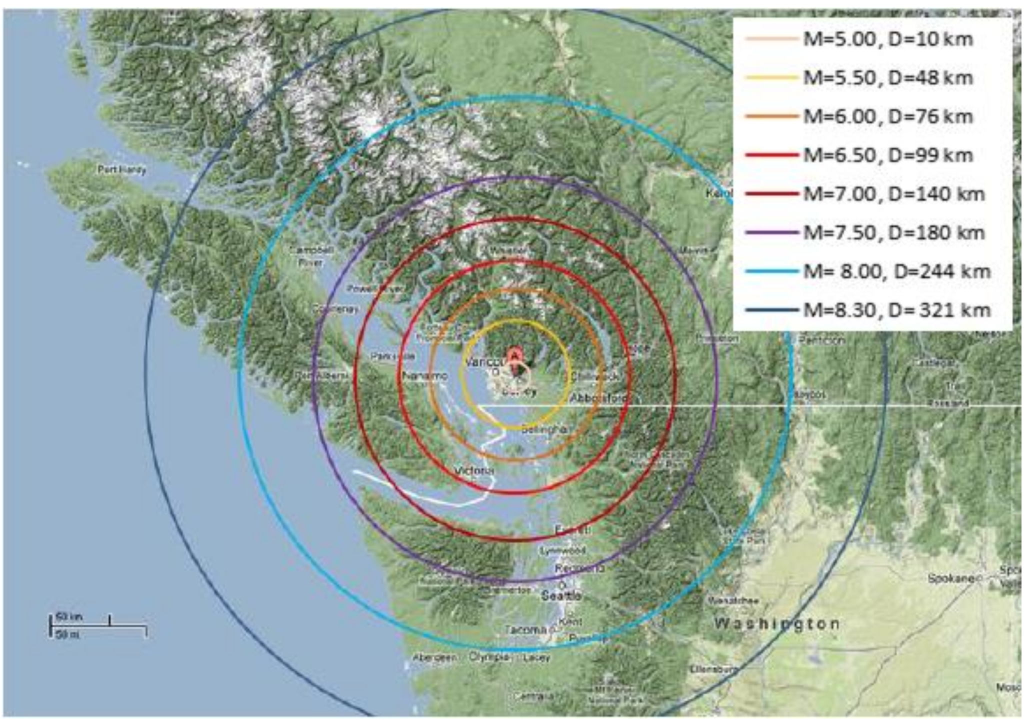

Figure 8: Magnitude and Distance for median collapse PGA = 0.055g at Pattullo Bridge, Site Class C and GSC modified Youngs et al. (1997) attenuation for subcrustal ground motions

4 BRIDGE WIND RESPONSE ASSESSMENT

The objective of this segment of the study was to find the critical wind event that would lead to the safety-critical conditions for the Pattullo Bridge. The wind analysis was carried out using the results of RWDI's wind engineering study (RWDI 2015). RWDI provided a static wind pressure distribution for a 100 year return period event that resulted in a design wind speed of 32.3m/s at the deck level that is 48.2m above the water level. RWDI investigated the directional probabilities of exceeding mean wind speeds corresponding to various return periods. The bridge main span is aligned approximately along an axis of 120 degrees to 300 degrees from North (i.e. approximately east-southeast to west-northwest). Therefore, winds normal to the spans would blow from approximately 30 degrees (north-northeast) and 210 degrees (south-southwest). RWDI provided 66 different load cases corresponding to these two wind directions (30^0 and 210^0). For each load case, the wind loads on the bridge were given as distributed vertical, lateral, along-the-deck, and torsional moment loads. For this assessment, the loads were rationalized and applied to the structure as a single load case comprising of distributed vertical, lateral, along-the-deck, and torsional moment loads which were applied simultaneously to the deck system members and the truss members.

Due to the limited scope of this assessment, winds normal to the heavy truss spans blowing from approximately 210 degrees (south-southwest) were applied to the FE model that represents the most critical strong wind condition. Figure 9 shows the schematic representation of the wind directions.

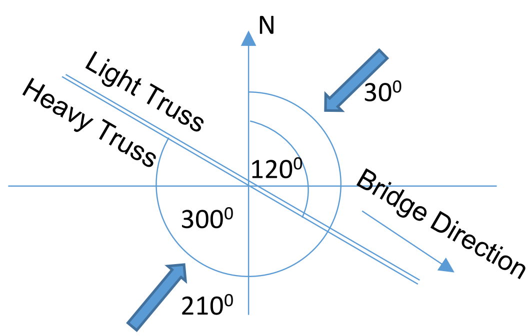

Figure 9: Wind Load Directions at Pattullo Bridge Site

The global force demands for the piers were calculated based on the combined factored dead and wind loads. The load factor for the wind load in S6-14 (CHBDC-S6-14) is 1.4 whereas in S6-06 (CHBDC-S6-06) it is 1.5. In the previous wind assessment report from COWI (2015), the wind load factor was considered to be 1.5. In order to be consistent, a similar load factor is considered in this assessment; however, could be reduced if seen to be too conservative.

For columns and crossheads, the shear and flexural capacities were calculated per S6-14 (i.e. 0.75 and 0.9 for concrete and steel reinforcement, respectively). Similar to the seismic analysis, it has been concluded that Pier-3 is the critical link under the wind load both laterally and longitudinally. Wind analyses results showed that, Pier-3 has a D/C ratio of 1.20 considering the RWDI recommended wind speed of 32.3m/s at the deck level (48.2m above the water level) for the 100 year return period wind. The corresponding mean hourly wind speed at 10m above the water level level is 26.3m/s which is lower than the code specified (S6-14) wind speed of 29.1m/s at the bridge location. According to the analysis result, a wind speed of 26m/s at deck level (48.2m above the water level) will be the resulting critical wind speed for the existing bridge. Any wind event with higher wind speed will result in life endangering damage on the bridge as well as the bridge approaches.

5 CONCLUSIONS

Based on the wind and seismic assessment study of the Pattullo Bridge, considering the limited scope of the study, the following conclusions are drawn:

- The threshold seismic event for the existing Pattullo Bridge is 1 in 30 year earthquake.
- For M=6.0 crustal earthquake needs to be located 58 km from the bridge site and 76 km for the subcrustal earthquake to initiate a critical condition for the bridge.
- The threshold wind speed for the existing Pattullo Bridge is 26 m/s at the deck level (48.2m above the water level).
- Based on this assessment, the probability of exceeding a threshold event within the expected remaining 10 year services life is unlikely.

ACKNOWLEDGEMENT

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DISCLAIMER

The results and discussions presented in this study are based on the analysis results and do not necessarily reflect the official position of Translink.

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