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SEISMIC DESIGN OF A CURVED STEEL BOX GIRDER BRIDGE FOR THE SOUTH FRASER PERIMETER ROAD (SFPR)

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ABSTRACT: The South Fraser Perimeter Road (SFPR) is a new 40km four-lane route along the Fraser River from the Highway 17 - Deltaport Way Interchange in Delta, to Highway 15 in Surrey. A new interchange connects the south end of the SFPR to Highway 17 and Deltaport Way. The major bridge structure in the interchange carries the SFPR over the CN Rail Right of Way and Deltaport Way eastbound ramp along a curved alignment at up to a 60° skew. The new 137m long, three span continuous bridge is comprised of three trapezoidal steel box girders with a composite concrete deck. Highway geometry and other site constraints limited the constructible sizes of pier columns and footings to an extent that they were not able to provide the necessary resistance to seismic induced lateral forces. In addition, soil conditions challenged the designers with potential for both settlement and seismic liquefaction potential. This paper presents details of the seismic analyses and design, and highlights the unique solutions. A seismic monitoring system was installed on the bridge that includes displacement transducers and accelerometers on abutments and piers paired with ones on the isolated bridge deck to capture the structural response to seismic events.

1. Introduction

The South Fraser Perimeter Road (SFPR) is a new 40km four-lane route along the Fraser River from the Highway 17 - Deltaport Way Interchange in Delta, to Highway 15 in Surrey. The new SFR @ Deltaport Way Overhead connects the south end of the SFPR to Highway 17 and Deltaport Way. The new overhead carries the Hwy 17 northbound off ramp traffic, and then turns almost 90 degrees over the CN Rail tracks and Deltaport ramp and east toward the SFPR (Figure 1). The bridge has a span arrangement of 35m-67m-35m, and is comprised of three trapezoidal steel box girders with a composite concrete deck. The geometry of the roadways dictated that the north pier (Pier 2) be a portal frame spanning the Deltaport Way ramp. Detailed seismic analyses showed that very large lateral loads made the design of the concrete portal frame and foundations impractical. A potential solution was to use tall elastomeric bearings to support the box girders and isolate the superstructure from seismic loads, thus reducing the lateral loads acting on the substructure to manageable levels.

Liquefaction assessment and ground displacement analysis estimated liquefaction induced displacements at the piers, abutments, and approaches up to 800mm. Vibro-densification ground improvements were designed for under the abutments to reduce the displacements to tolerable levels. Structural components were designed to accommodate liquefaction-induced displacements, and the amount of costly ground improvement required was minimized by taking advantage of the structure's ability to withstand permanent displacements. Detailed imposed-displacement analysis of the structure was used to demonstrate the structure would meet post-earthquake performance requirements. A seismic monitoring system has been installed on the bridge which includes displacement transducers and accelerometers on the abutments and piers paired with ones on the isolated bridge deck to capture the structural response to seismic events.

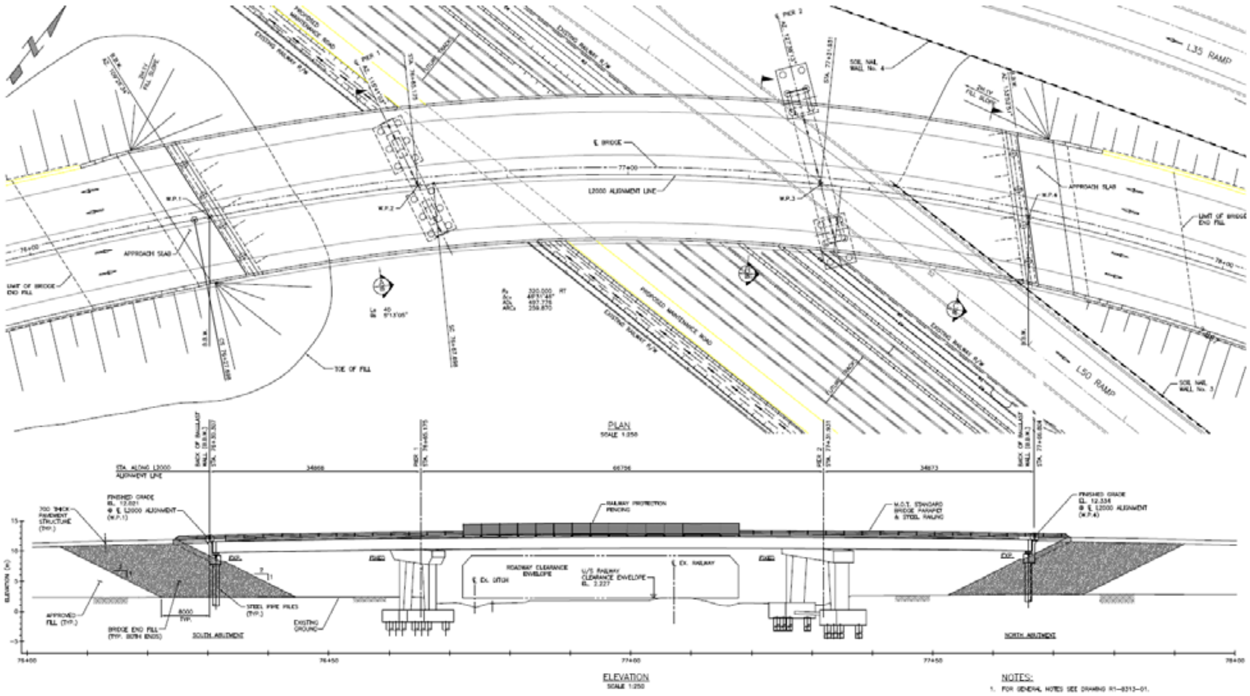

Fig. 1 – General Arrangement of the Overhead Structure

2. Seismic Design Criteria, Geometry Constraints, and Geotechnical Issues

2.1. Seismic Design Criteria

The new overhead structure was defined as an "Economic Sustainability Route Structure" and needed to meet the project specific seismic performance criteria. The approach fill, abutment and overhead structure performance criteria are shown in Table 1.

Table 1 - MoT Project Seismic Performance Criteria

Structure	Design Earthquake			
	Cascadia Subduction	**475 year return period**	**1000 year return period**	**2475 year return period**
Approach Fills	-	Significant Damage	-	-
Abutment	Repairable Damage	Repairable Damage	Significant Damage	No Collapse
Bridge	Repairable Damage	Repairable Damage	Significant Damage	No Collapse

2.2. Site Constraints

Initially the piers were designed to be parallel with the railway and Deltaport Way, but this arrangement resulted in a very high skew of the bridge deck. In order to avoid the design and efficiency problems associated with high skews, the engineers designed a skewed portal frame type pier at Pier 2 where the Deltaport Way ramp pass through underneath. However the available space between the ramp and the CN Rail ROW was only 4.5m wide. In addition, to avoid having to construct a railway protection crash wall between the pier column and the railway, the pier column needed to be 25 feet (7.62m) away from the centre line of the exterior railway track. This limited the size of the pile cap and the number of piles that could be accommodated in this space. All of these constraints meant that the pier column dimensions at Pier 2 were severely limited.

2.3. Geotechnical Issues

The project is located in an area of the Lower Mainland with difficult soil conditions. The site is underlain by compressible soils greater than 100m in depth. The liquefiable soils have a depth of more than 15m. The liquefaction assessment found that the entire sand layer is liquefiable during an earthquake event (Fig. 2). Earthquake induced settlements and lateral spreading are expected to occur due to the dissipation of excess pore water pressure that may develop during earthquake shaking. If untreated, earthquake induced settlement is expected to be about 400mm for the 2475 year return period earthquake. The estimated settlement in the lower silty layer is expected to be in the range of about 150mm to 250mm for the 2475-year return period earthquake. Liquefaction-induced lateral displacements were estimated using the Finite Difference computer program FLAC to be in the range of about 400mm to 800mm if the ground is left untreated (Fig. 3 and Fig. 4).

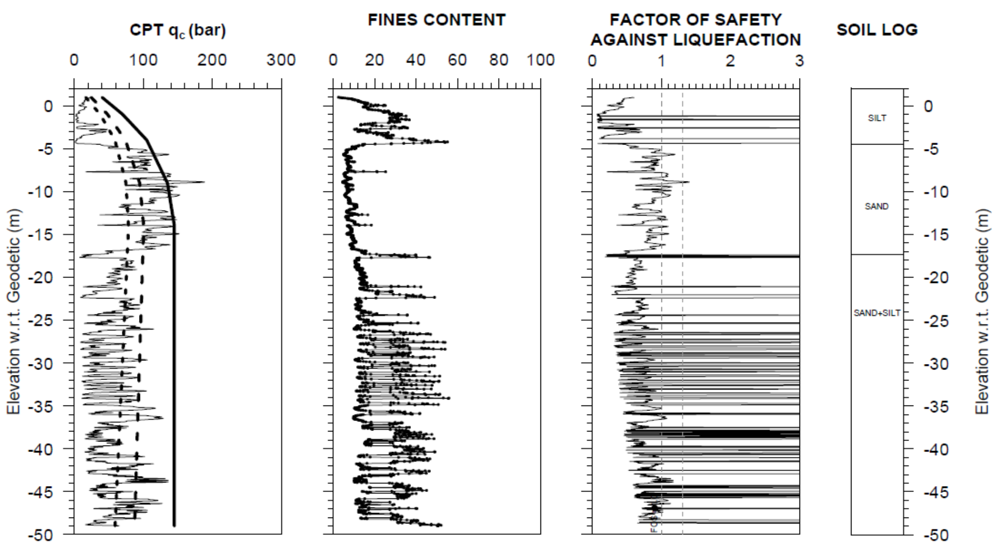

Fig. 2 – Liquefaction Assessment of the Overhead Structure Site

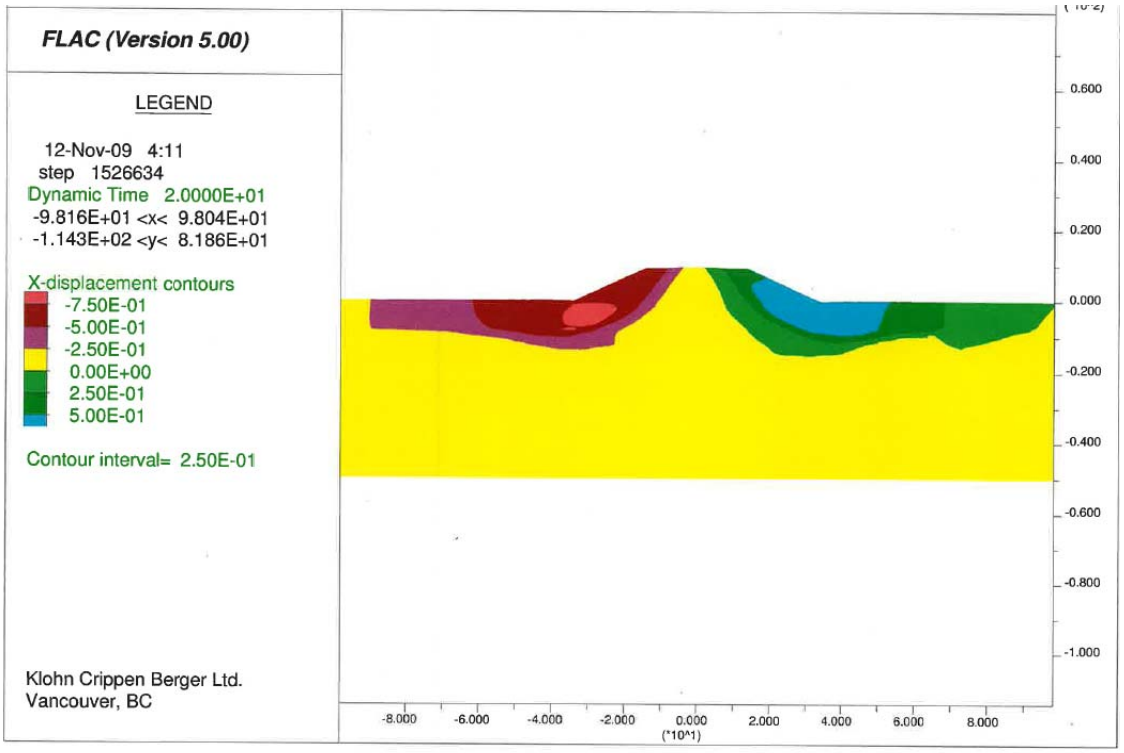

Fig. 3 – Liquefaction-induced Displacement Analysis with FLAC

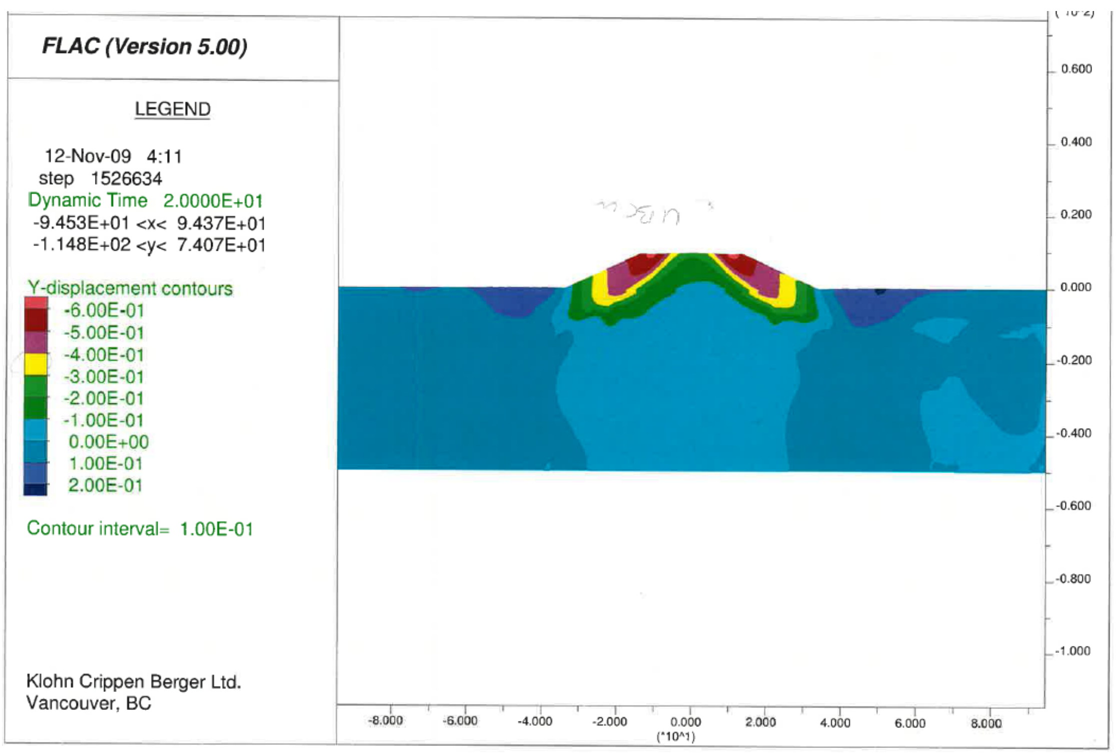

Fig. 4 – Liquefaction-induced Displacement Analysis with FLAC

3. Preload and Ground Improvement Strategies

The results of the liquefaction analysis and limit equilibrium stability analyses indicated that ground improvement would be required to meet the seismic performance criteria of the bridge structures as set by MoT. Liquefaction assessment and ground displacement analysis were used to estimate seismic induced post-liquefaction displacements at the piers, abutments, and approaches. Vibro-replacement stone column ground improvements to a depth of about 18 m were designed around the abutments of the bridge to reduce movements to tolerable levels. The extents of improvements needed to meet the post-seismic design criteria were determined using FLAC analyses. Structural components were designed to accommodate the seismic induced displacements, such that the amount of costly ground improvement required could be minimized by taking advantage of the structure's ability to withstand relatively large permanent displacements.

Settlement analysis of the soils indicated an extensive preload and surcharge program would be required to meet performance requirements for the roadway embankments and bridge approaches, which exceeded 10m in height. The preload was placed early in the project using an existing day labour contract the Ministry of Transportation had with a local contractor to fast track the schedule. The settlement of the fills and any impacted utilities were monitored and correlated to settlement analysis estimates to confirm the required duration of preloads. A soil-nail and shotcrete wall with a cast-in-place concrete face was designed to allow reuse of the preload sand as permanent embankment fills, by reducing the horizontal extent of the fill adjacent to new traffic lanes.

The required extents of ground improvement, to satisfy the post seismic performance criteria, are shown in Fig. 4 and Fig. 5 below.

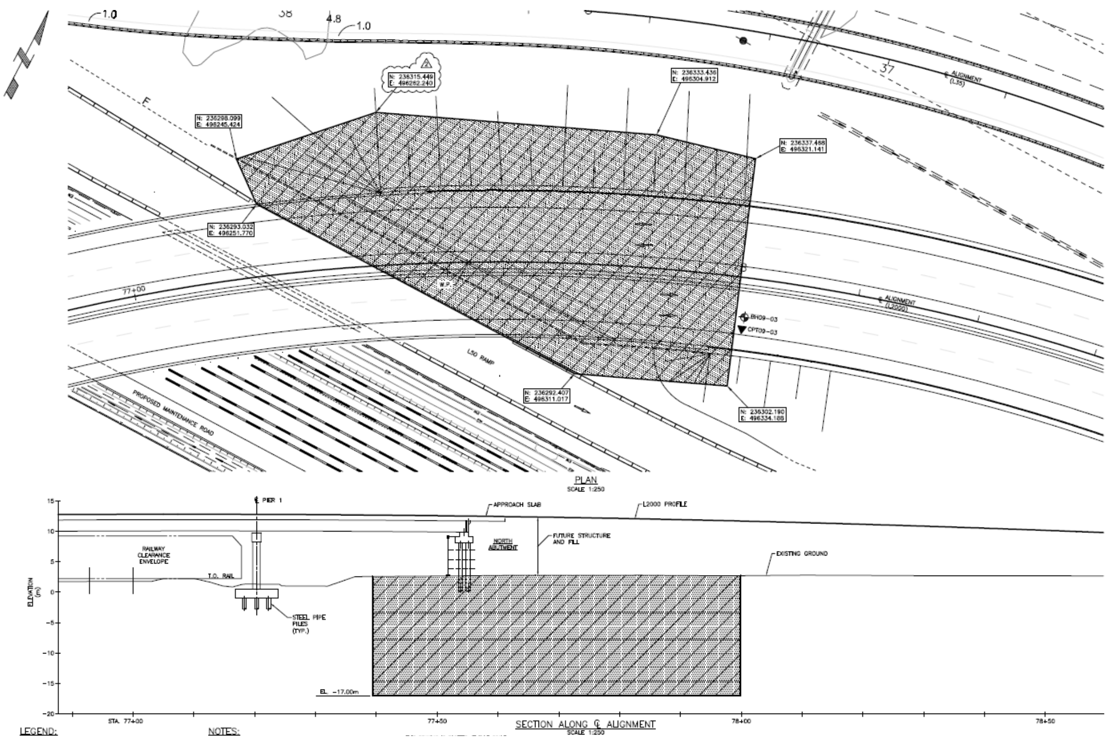

Fig. 5 – Ground Improvement at North Abutment

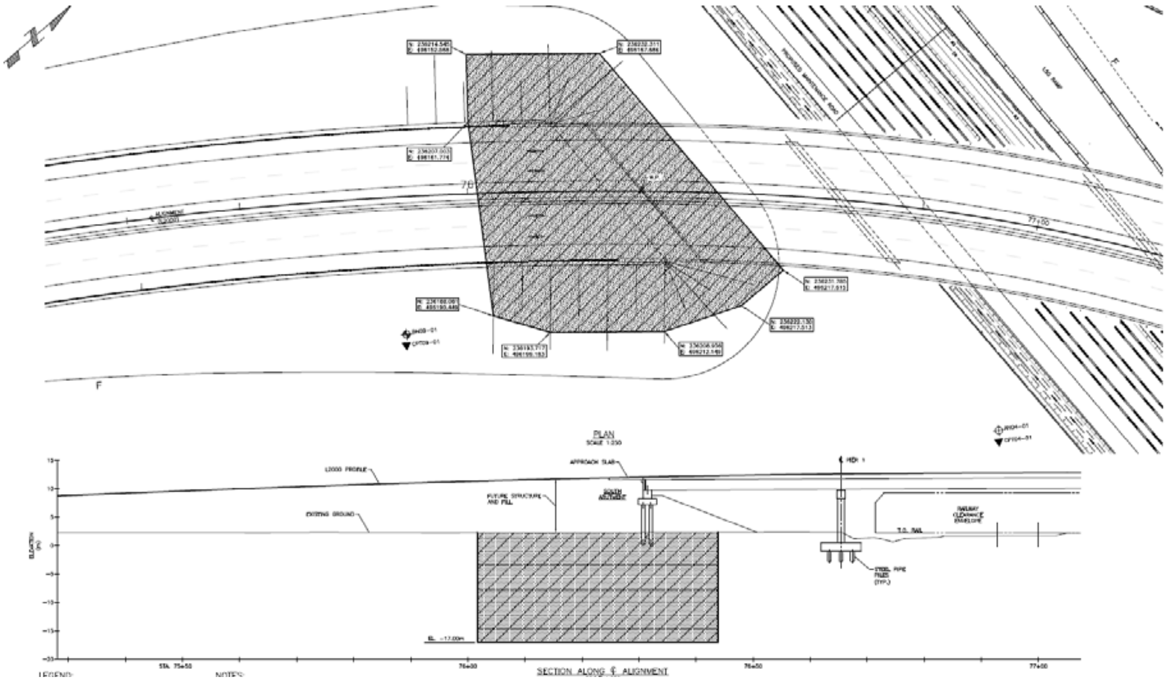

Fig. 6 – Ground Improvement at South Abutment

The structures are founded on steel pipe piles with diameter of 1067mm. Abutment piles are closed-ended and terminate at about 15 m depth within the ground improvement zone. Pier piles are open-ended and terminate at up to 52 m depth.

The required CPT cone tip resistance for areas of ground improvement are summarized in Table 2.

Table 2 – Required CPT Tip Resistances

REQUIRED CONE TIP RESISTANCE		
qc (bars)		
Geodetic Elevation	Fines Content	
(m)	< 10%	10% to 20%
-1	70	50
-4	105	75
-9	135	95
-14	145	100
-17	145	100

For many of the seismic and post-seismic failure surfaces, the critical zones are quite shallow and pass through the upper silt. Ground improvement using stone columns can be used to increase the strength of this material, but in some areas flattening of the approach side slopes, toe berms, or preloading, could achieve the required factor of safety. Combinations of these solutions were used on the approaches.

4. Structural Design Strategies and Optimization

4.1. Superstructure Design

One of the many design challenges for this overhead structure was minimizing the girder depth to lower ramp profiles and thus create shorter, more cost-effective bridge approaches. This not only decreased project cost but also enabled a more flexible alignment design. Various superstructure options were examined and a 3-span bridge consisting of three steel trapezoidal box girders with a composite concrete deck was considered the optimal solution. Carefully designed cross-frames and lateral bracing were used to help control deflections due to deck torsion, which helped minimize the constant depth girders and weights, which in turn reduced the seismic lateral loads on the substructures.

4.2. Isolated Superstructure to Reduce Lateral Seismic Loads

Detailed seismic analysis showed that very large seismic induced lateral loads were expected in the portal frames and piled foundations. This resulted in relatively large portal frame beam and column sizes, a large number of piles, and an excavation for pile caps between the CN Rail track and Deltaport Way ramp, which was impractical to construct. In order to solve these problems, the lateral force demands on substructures needed to be substantially reduced. The use of seismic isolation bearings (Figure 7) is an effective method to significantly reduce the loads on substructures, and made it possible to reduce substructure component size and footing size to practical dimensions. Although they are more expensive than pot bearings, the use of isolation bearings reduced the construction costs overall.

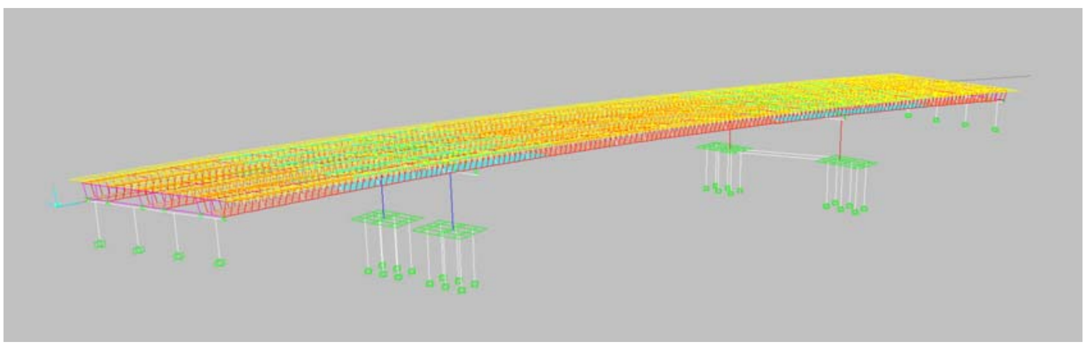

Fig. 7 – FE Model for Seismic Analysis

Fig. 8 – Isolation Bearings

Elastomeric isolation bearings were designed as a vertical-load carrying device that also provides lateral flexibility so that the period of vibration of the total system is lengthened sufficiently to reduce the force response (Fig. 8). The typical force response with increasing period is shown schematically in the typical acceleration response curve in Fig. 8. A reduction in base shear forces occurs as the period of vibration of the structure is lengthened. The extent to which these forces are reduced primarily depends on the nature of the earthquake ground motion and the period of the fixed base structure. However, the additional flexibility needed to lengthen the period of the structure will give rise to relative lateral displacements across the flexible mount (Fig. 8). In addition, it is undesirable to have a bridge that will vibrate perceptibly under frequently occurring loads, such as wind or braking. In this particular bearing design, modified elastomers which have a high initial elastic stiffness were utilized to provide rigidity at these service loads. The properties of the elastomeric bearings are shown in Table 3. The bearings were tested in accordance with The 1999 AASHTO Guide Specifications for Seismic Isolation Design. The bearings were tested in compression with a 5-minute sustained proof load of 1400KN and 5380KN for bearings at abutments and piers respectively. The bearings were also tested in combined compression and shear, the compressive loads were 500KN and 2810KN for bearings at abutments and piers respectively when the bearings are subject to five fully reverse cycles of shear loading to a displacement of +/- 125mm.

Table 3 – Properties of Isolation Bearings

	South Abutment	Pier 1	Pier 2	North Abutment
Diameter of Bearing (mm)	622	800	800	622
Thickness of Bearing (mm)	180	240	240	180
Effective Stiffness of Bearing for Model (N/mm)	2093	2649	2649	2093

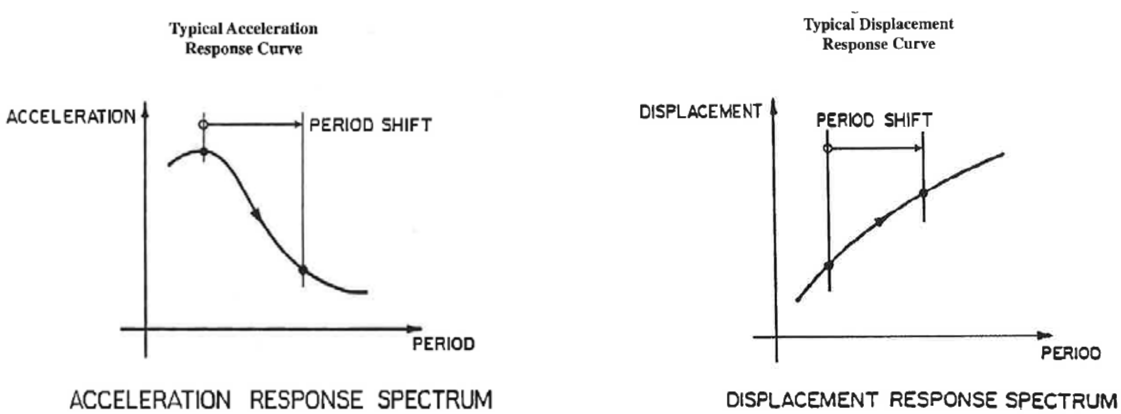

Fig. 9 – Typical Response Curves for Isolated Structures

A comparison of forces at the column base at Pier 2 is shown in Table 4. There are significant force reductions which allowed a more efficient design and improved constructability.

Table 4 – Forces Comparison at Pier 2 Base due to Earthquake (Isolated vs. non-isolated)

	Forces at Column Base at Pier 2		
	Lateral Bending (KN*m)	Lateral Shear (KN)	Axial Force (KN)
Isolated Structure	6585	4711	2573
Non-isolated Structure	12171	8720	4493

4.3. Design the Superstructure to Accommodate the Post Earthquake Settlement and Displacement

Structural components were designed to accommodate liquefaction induced settlements and displacements, and the amount of costly ground improvement required was minimized by taking advantage of the structure's ability to withstand relatively large permanent displacements. Detailed imposed displacement analysis of the structure was undertaken to demonstrate the structure would meet post-earthquake performance requirements.

5. Seismic Monitoring Program

A seismic monitoring system is installed on the bridge, which includes displacement transducers and accelerometers installed on abutments and piers paired with ones on the isolated bridge deck to capture the structural response to seismic events (Fig. 9 and Fig. 10).

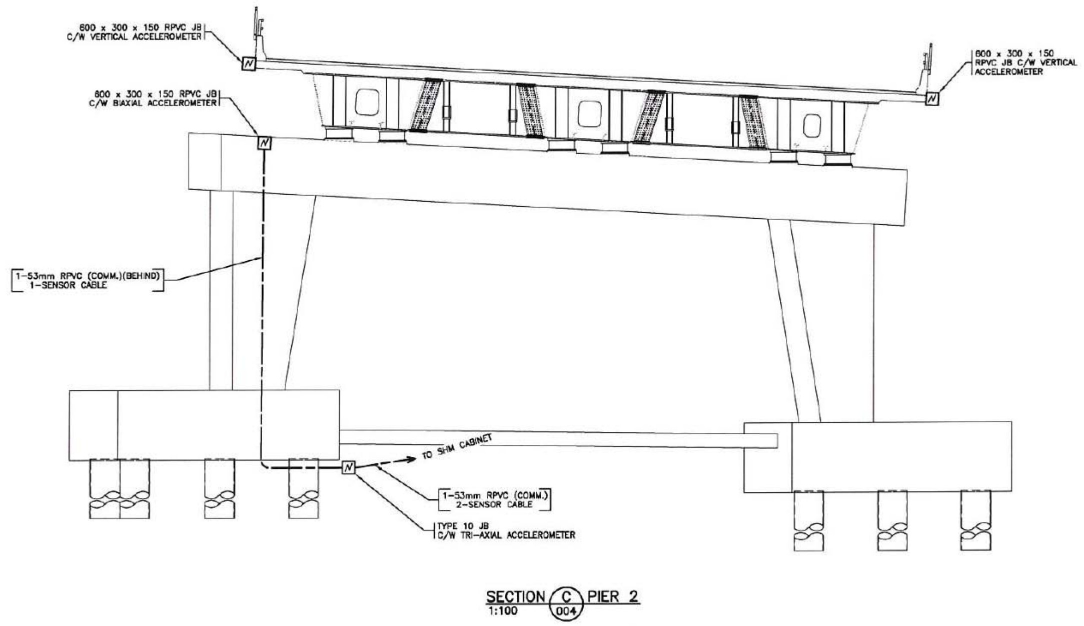

Fig. 10 – Seismic Monitoring System at Pier 2

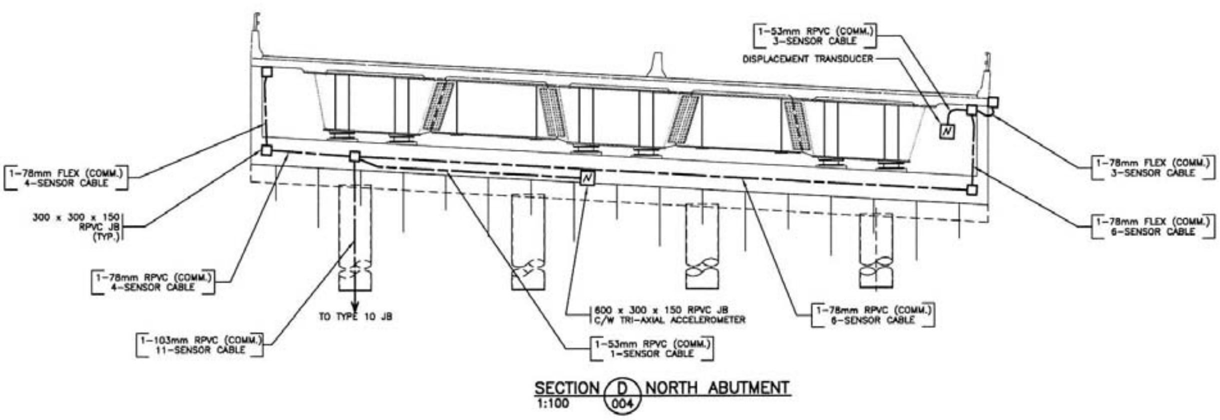

Fig. 11 – Seismic Monitoring System at North Abutment

6. Conclusions

In summary, the key design features and analysis and design techniques used to deal with the many challenging issues and constraints on this project are:

- Analysis showed that the post-earthquake factor of safety could more efficiently be increased by building local toe berms, flattening the slope or preloading to overconsolidate the surficial silt around the abutments. The amount of costly vibro-replacement stone column ground improvement to a depth of about 18 m was reduced, and structural analysis of the liquefaction induced foundation movements was used to demonstrate the bridge met the project performance criteria.
- Sophisticated FLAC analyses were used to predict the expected displacement and earthquake induced settlements and to determine the extent of the ground improvement required.
- The relatively long span lengths, the need to keep the profile low, the superstructure relatively light, avoid temporary support in the railway right-of-way, and fabricate complex geometry gave steel a distinct advantage over concrete.
- Tall elastomeric seismic bearings were used to isolate the superstructure, substantially reducing loads on the substructure. This allowed the design of piers and foundations which could fit the geometry constraints alignment of the CN Rail tracks and Deltaport Way at Pier 2.
- The bridge was designed to accommodate liquefaction-induced movements, and the amount of costly ground improvement required was minimized by taking advantage of the structure's ability to withstand permanent displacements.

7. Acknowledgements

The project team consists of Klohn Crippen Berger Ltd. as the prime consultant managing a multi-discipline design team and performing project management, structural design, and geotechnical design. R.F.Binnie and Associates Ltd. as the sub-consultant for the project is responsible for the alignment design, drainage and utility design. The Contractor was Griffels Westpro and the B.C Ministry of Transportation and Infrastructure are the Owner, and managed the delivery of the project. All are gratefully acknowledged.