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FIRST NARROWS AND PORT MANN WATER SUPPLY CROSSINGS SEISMIC VULNERABILITY ASSESSMENT

John Sherstobitoff[1] David Siu[2] Ian Stewart[3] Qishi Chen[4]

SUMMARY

To mitigate a risk of water supply disruption after an earthquake, the Greater Vancouver Water District (GVWD), located in British Columbia, Canada, initiated a seismic assessment of two critical underwater pipelines, the First Narrows and Port Mann crossings. These crossings are key elements in GVWD's water supply system that delivers on average over one billion liters of water daily to two million people in the Greater Vancouver area.

The Port Mann crossing consists of a 1.2 m diameter welded steel pipe, constructed in 1974 primarily using bell-and-spigot joints, installed at shallow depth beneath the bed of the Fraser River. The First Narrows crossing consists of two welded steel pipelines converging to a valve chamber, a large diameter vertical shaft, and finally a tunnel beneath Burrard Inlet. Both pipeline crossings are vulnerable to damage during a major earthquake as they pass through liquefiable soil predicted to impose significant displacements on the pipelines.

The geotechnical analysis utilized the software FLAC, SHAKE, and VERSAT to predict the areas of soil liquefaction and soil movements. The structural analysis used the software ABAQUS, to model the non-linear material properties of the pipelines and the non-linear soil-structure interaction. A key aspect was to develop strain based failure criteria for the variety of pipeline materials and joint types encountered, however special emphasis was placed on the critical bell-and-spigot joints. A full scale replica of the typical bell-and-spigot joint was fabricated, joint samples were tested, and a finite element model of the complete cylinder was developed to correlate the test results to the predicted tensile and compressive failure limits. The paper presents the assessment of the pipelines for 3 levels of earthquake including the Maximum Credible Earthquake conceivable for the region, and the structural and geotechnical remedial options developed to improve the reliability of the crossings.

[1] Manager of Buildings and Infrastructure, Sandwell Engineering Inc., Vancouver, Canada. jsherstobitoff@sandwell.com
[2] Senior Engineer, GVRD, Vancouver, Canada
[3] Vice-President, EBA Engineering Consultants Ltd., Vancouver, Canada
[4] Senior Specialist, CFER Technologies, Edmonton, Canada

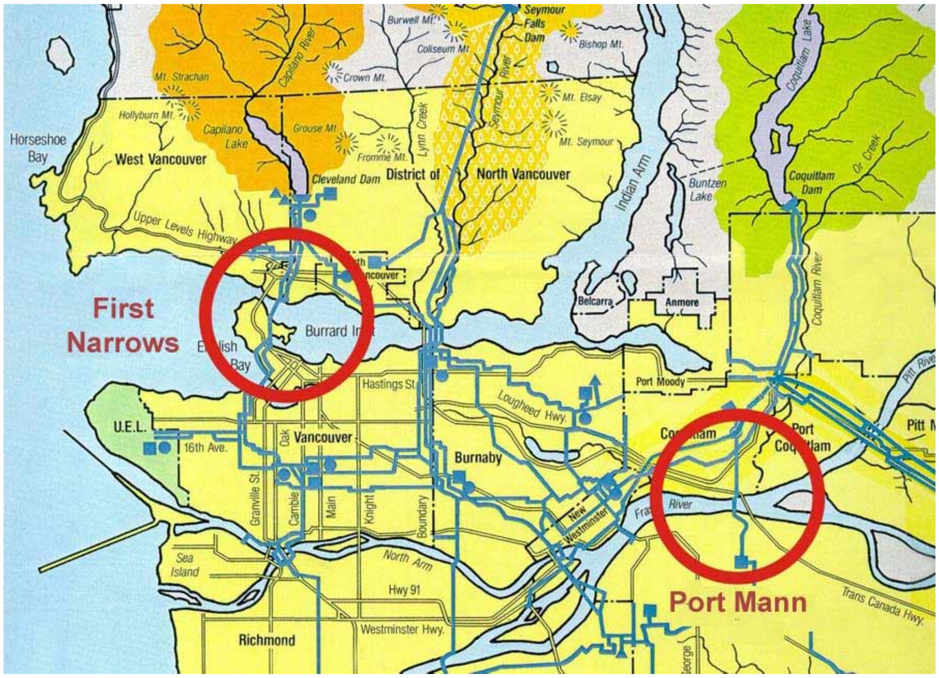

Figure 1 Location of Crossings in Greater Vancouver Water District

FIRST NARROWS WATER SUPPLY CROSSING

First Narrows Crossing - General

The First Narrows Crossing is located approximately 800 m east of the Lions Gate Bridge (see Figure 1) and is used to transmit water under Burrard Inlet from North Vancouver to the City of Vancouver. The crossing collects water from the Capilano Watershed conveyed to the North Valve Chamber via 2m and 1.2m diameter welded steel pipelines called the Capilano Mains No. 4 and No. 5. These mains divide into three pipes within the North Valve Chamber and emerge from the chamber as three 1.2m diameter steel pipelines called Intakes Nos. 1, 3, and 4, before entering the North Shaft. The crossing consists of a vertical shaft approximately 120m long into bedrock (the North Shaft), a concrete grouted tunnel approximately 1km long in the bedrock (Pressure Tunnel), with another vertical shaft approximately 120m long located within Stanley Park (the South Shaft); see Figure 2.

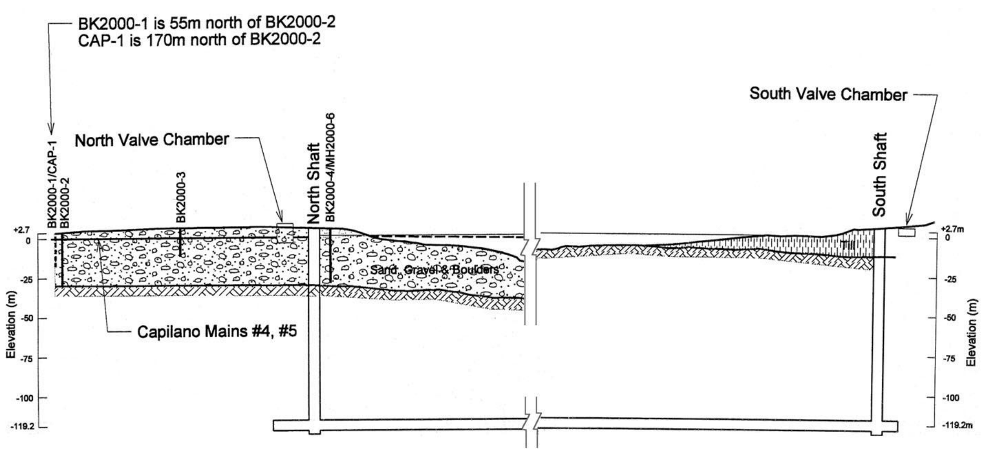

Figure 2 Cross Section of First Narrows Crossing

The North Shaft of the crossing fractured in both 1982 and 1986. The 1982 break was attributed to corrosion that caused leakage and deterioration of the joint between the welded steel pipe (caisson lining) and the Bonna Pipe at a depth of approximately 37 m below the ground surface. The resulting leakage caused water boils at the ground surface adjacent to the shaft. The break was clamped and the area outside the break was injection grouted to fill voids and stop leakage. The 1986 break occurred at a joint in the Bonna Pipe at a depth of approximately 43 m below the ground surface. The break resulted in a complete failure of the No. 1 shaft intake pipe and minor horizontal movements and minor tilt of the North Shaft towards Burrard Inlet. The 1982 and 1986 breaks caused disturbance to the surrounding rock and the soils adjacent to the North Shaft. As a result, grouting was undertaken between December 1987 and January 1988 to fill large voids, and to add structural strength to the disturbed rock and soil. Ground densification improvement work (vibroflotation) was carried out in February 1988 in an approximately 6 to 8 m radial pattern around the shaft. Also in 1988, approximately 9 m of all three intake pipes was replaced with new steel pipe complete with 'Dresser' type couplings and tie rods at each end capable of accommodating minor differential movement between the North Valve Chamber and the North Shaft.

First Narrows Crossing – Seismicity and Geotechnical

The seismic vulnerability assessment was carried out for three levels of earthquake with 100, 475 and 10,000 year return periods. The earthquake with a return period of 10,000 years is considered as the Maximum Credible Earthquake (MCE). Measured acceleration time histories from past earthquakes were selected and modified to fit target response spectra for the three levels of earthquakes. The peak firm ground accelerations (PGA) of these modified acceleration time histories were 0.44g, 0.21g and 0.08g for the MCE event, the 1 in 475-year earthquake, and the 1 in 100-year earthquake, respectively.

A geotechnical field investigation program was carried out on the north shore and consisted of 5 Becker Penetration Tests (closed-end casing), 2 Becker sampling holes (open-end casing), and 1 mud-rotary drill hole near the North Shaft. The results of the field investigation were used in support of the detailed geotechnical assessment of the First Narrows Crossing.

Free-field liquefaction assessment was carried out for a soil column at the north shore of the Burrard Inlet. The cyclic shear stress ratios (CSR), calculated from the site response analysis using SHAKE, were used in the liquefaction analysis. The procedures used to assess liquefaction potential of the granular soil followed the guidelines of NCEER [1] using the liquefaction resistance chart as a function of the normalized field Standard Penetration Test (SPT) blow counts, (N1)60. For Becker Penetration tests, the (N1)60 values were derived from the BPT blow counts using the interpretation methods proposed by Harder and Seed [2] and Sy and Campanella [3].

Results of liquefaction analyses indicated that liquefaction was discontinuous with depth and was likely to occur at depths from 9 m to 12 m and from 19 m to 22 m at the north shore of the Burrard Inlet under the MCE event. Analyses also indicated that shallow liquefaction at depths less than 8 m was likely to occur in a localized area about 170 m to 230 m north of the North Shaft, affecting the incoming Mains No. 4 and No. 5. The analysis indicated that the offshore soil under the Burrard Inlet would also experience soil liquefaction under the MCE event. Liquefaction hazard associated with the 1 in 475-year earthquake was found to be significant but less severe than the MCE event. Only limited soil liquefaction was predicted on the north shore of the Burrard Inlet under the 1 in 100-year earthquake.

At the south shore of the Burrard Inlet, the soil was found to consist of dense glacial till deposits and therefore was not considered to be liquefiable. Historical observations made during construction of the near horizontal Pressure Tunnel indicated that the tunnel was excavated through a sequence of sandstone, shale and slickensided shale. Based on the geological data at locations along the pressure tunnel and at the South Shaft, it was considered that shaking from any of the 3 levels of earthquake considered would unlikely affect the performance of these structures.

Limit equilibrium analyses were conducted to assess the flow slide potential of the soil slope at the north shore of the Burrard Inlet. Liquefiable soil zones identified from the liquefaction assessment were modeled using the residual strengths of liquefied soils. The residual strengths were in general estimated based on the normalized SPT (N1)60 values using the relationship proposed by Idriss [4]. Limit equilibrium analyses indicated that a flow slide was not likely to occur at the north shore of the Burrard Inlet as a result of soil liquefaction.

Earthquake induced ground deformations were computed using the multi-degree of freedom dynamic analysis computer programs VERSAT [5] and FLAC [6]. An effective stress finite element analysis was carried out in the VERSAT approach that considered pore water effects on soil stiffness and strength. The synthesized total stress approach developed by Beaty and Byrne [7] was incorporated into the finite difference computer program FLAC. The earthquake induced ground deformations were also estimated from the Bartlett and Youd empirical equations [8] [9]. The ground displacements along the pipeline alignment determined from the geotechnical analyses were recommended for use as input to the structural performance analyses of the pipeline.

The MCE ground displacements used in the structural analysis are noted in Table 1.

Table 1 – First Narrows MCE Ground Displacements

Location	Longitudinal	Vertical	Transverse
200m north of shaft	140mm	150mm	50mm
North Valve Chamber	280mm	140mm	30mm
North Shaft			
-top 10m	350mm	40mm	50mm
-next 10m	80mm	<5mm	30mm
-below 20m	<30mm	nil	<20mm

The force-displacement relationship between the embedded pipelines and the surrounding soil mass was simplified through the use of bilinear soil springs attached to the pipeline. Both mean and upper-bound spring values were determined for use in the soil-structural analyses for each of the three levels of earthquake. The general procedures as suggested by O'Rourke and Liu [10] were followed in the evaluation of spring constants in the current study. Yield displacements recommended in the current study were compared to a range of values published in the ASCE 1984 procedures [11] and found to be in agreement. Nonlinear spring constants were also determined for the North Shaft structure and the North Valve Chamber piled foundation.

First Narrows Crossing – Pipelines and North Shaft

No material testing was carried out, since no pipe material representative of the original construction was available. The strain limit criteria for all pipe sections and connections was developed based on existing knowledge and accepted industry practice with consideration of the joint type, steel properties, pipeline diameter and wall thickness. The use of strain limits to set the "failure criteria", that is a deformation based, rather than force based approach is a methodology being adopted more and more internationally, including adoption by Professor Miyajima, one of Japan's leaders in this field retained to provide peer review for the project.

The pipeline strain limits were set as the lower bound strain limits of the joints, since locations of the joints was unknown along the lengths of pipe. These limits were 0.29 – 0.32% tension and 0.23 – 0.34% compression for the butt strap joints, depending on diameter and wall thickness. The riveted plate connections for the original steel innerlining in the North Shaft was set at 0.11% tension and compression.

The pipeline crossing was modeled in a single 3D model using the general purpose non-linear finite element analysis software ABAQUS; see Figure 3. The top 37m of the North Shaft, located in soft soil, consists of a 7.3m outside diameter (OD) by 1.5m thick reinforce concrete outer shaft, an 0.8m layer of unreinforced infill, a 2.64m diameter steel innerlining plate, with a 100mm thick by 2.44m inside diameter (ID) reinforced concrete innerlining. The 1987 repair added a new steel liner of 2.13m diameter. The lower portion of the North Shaft, located in rock, consists of a 2.95m ID concrete caisson cast against a 3.35m OD rock shaft, a layer of unreinforced concrete, and a 2.44m ID Bonna pipe (steel plate with shop applied reinforced concrete lining). All layers were modeled separately, yet to behave compositely, each was assigned appropriate non-linear properties different in both tension and compression. Furthermore, the behavior of the "Dresser couplings" (essentially a clamped connection incorporating a rubber gasket), tension-only tie rods, and composite steel and concrete beams supporting the pipeline were modeled in significant detail in an effort to capture the overall system behaviour. The modeling parameters for the pipeline components and the soil-structure interaction were similar to those described later for the Port Mann Crossing work. For the 1/100 event, the analyses indicated that no areas of the pipeline had strains that exceeded the strain limit criteria. For the 1/475 event, the analysis indicated that Intake No. 3 has the highest probability of failure, either by rupture of the pipe itself or at one coupling joint, with a similar probability of failure of Intake No. 4 at one coupling.

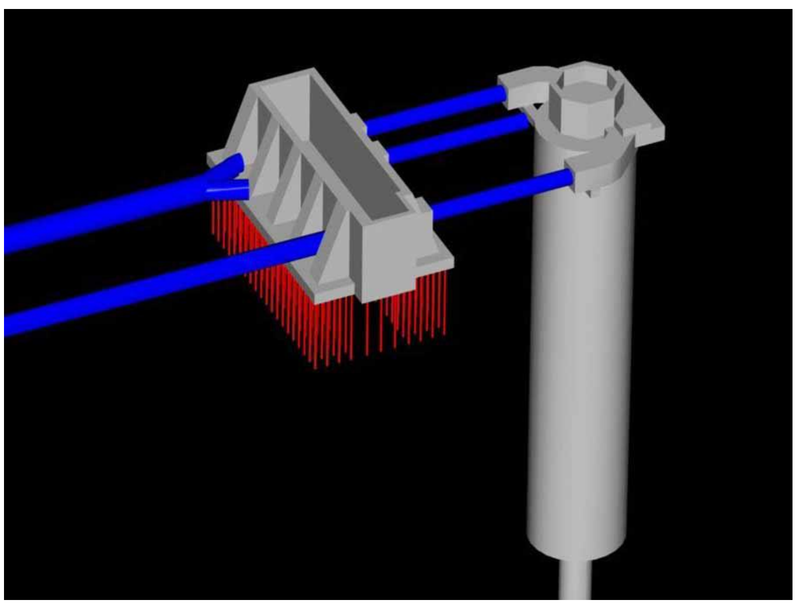

Figure 3 First Narrows Crossing – ABAQUS Model of Valve Chamber, Shaft, Pipelines

For the MCE event probable failure areas for which remediation was recommended are:

- Intake No. 3 pipe body at face of North Shaft, tie rods at two couplings, and center ring at one coupling;
- Intake No. 4 tie rods at both couplings;
- Intake No. 1 pipe body at face of North Shaft, and center ring at one coupling;
- Cantilever beams on North Shaft supporting Intake Nos. 1 and 4

The high strains in the above noted locations are caused primarily by vertical settlement of the North Valve Chamber relative to the North Shaft, downward movement of the soil around the Intake pipes, and by the outward movement of the Shaft relative to the Chamber.

Remediation was also recommended at the following two areas for the 1/475 and MCE events:

- Cross connection piping located between Mains No. 4 and 5 some 100 m north of the North Valve Chamber; to prevent rupture of the 305 mm cross connection piping should differential movement occur between the two mains.
- Connection of the concrete above-ground access chamber at the North Shaft, to prevent the potential of the chamber "sliding" off the shaft and potentially damaging mechanical components within.

First Narrows Crossing - Remediation

Based on the results of the geotechnical and structural assessment a number of seismic remediation options have been considered to upgrade the probable failure locations listed above. Both geotechnical and structural methods were considered, and order-of-magnitude costs (Year 2000, Canadian dollars) developed as follows:

Remediation for 1/475 Event

- Compaction grouting beneath the piles supporting the North Valve Chamber ($170,000), or
- Replace tie rods on Intake Nos. 3 and 4 ($40,000) and externally reinforce Intake No. 3 near the North Shaft ($50,000) totaling $90,000, or
- Replace tie rods on Intake No. 4 ($20,000) and replace Intake No. 3 with a section of pipe incorporating flexible joints ($190,000) totaling $210,000

The first option will virtually eliminate the relative vertical displacement between the North Valve Chamber and the North Shaft, and would likely reduce the pipeline strains to acceptable levels. This option would also be adequate for the MCE event. However, the potential for ground heaving affecting adjacent piping exists.

The second option relies on local reinforcing or replacement of weak components or sections of two Intake pipes. Furthermore, the upgrade would only be adequate for this level of earthquake; a slightly higher level earthquake could result in rupture to other sections of these pipes or Intake No. 1.

The last option provides Intake No. 3 the ability to survive both 1/475 and MCE level events, however, it leaves the other two Intakes upgraded only for the 1/475 level event and vulnerable for the MCE event.

Remediation for MCE Event

- Compaction grouting beneath the piles supporting the North Valve Chamber ($170,000), or
- Replace Intake Nos. 1, 3, and 4 with sections of pipe incorporating flexible joints ($550,000); see Figure 4.

The first option will virtually eliminate the relative vertical displacement between the Chamber and the Shaft, and would likely reduce the pipeline strains to acceptable levels. However, the potential for ground heaving affecting adjacent piping exists.

The second option will accommodate MCE and larger differential movements between the North Valve Chamber and the North Shaft. The opportunity exists to install the flexible joints in phases for each Intake line, and gain installation experience for each subsequent phase.

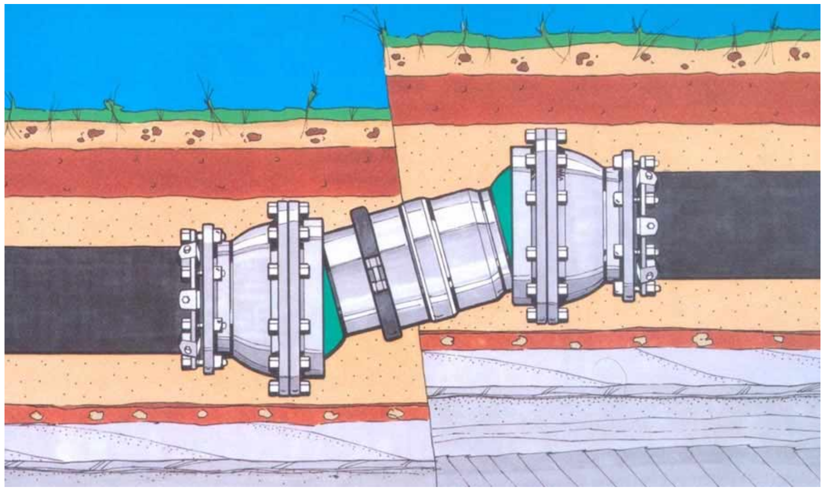

Figure 4 Typical Flexible Joints Recommended to Accommodate Differential Movement; such as Flex-Tend by EbbaIron

Other remediation includes:

- Installation of buried flex joint at cross connection piping between Mains ($50,000), or
- Soil remediation in the vicinity of the cross connection piping ($40,000); and
- Connecting walls of access chamber above North Shaft to the shaft using dowels or a concrete confinement ring at the base of walls ($10,000).

In total, the estimated cost to provide remediation suitable to protect the integrity of the crossing for the MCE event is about $600,000.

PORT MANN WATER SUPPLY CROSSING

Port Mann Crossing - General

The Port Mann Crossing was installed in 1974 and is located at a bend in the Fraser River, where the river turns from roughly a south-west direction to a west alignment; see Figure 1. The pipeline section considered in this study extends approximately 1.2 km from a valve chamber beneath the Port Mann Bridge in Coquitlam (north side of the river) to a valve chamber in Surrey (south side of river). As originally constructed, the crossing consisted of a 1220 mm (48 inch) steel plate pipe and was originally buried to a depth of 4.5 to 9.0 m below the river bed.

Due to the ongoing scour, by 1991, the riverbed coincided with the top of the pipe near the south shore. During a heavy spring freshet in 1997, scour around the pipe accelerated significantly. The pipeline failed as a result of the loss of soil which caused large pipe displacements in May 1997. From August 1997 until early 1998, a temporary 650 mm (26 inch) diameter pipe was connected to maintain water supply. In early 1998, a new section approximately 150 m long of 1220 mm diameter steel pipe was used to replace the damaged section. The new pipe section was replaced along approximately the same alignment and grade as the original pipe. The new pipe section is protected by a riprap apron, which extends 16 to 22 m upstream, 29 to 37 m downstream and has a minimum thickness of 2.5 m at the crown of the pipe.

Port Mann Crossing – Seismicity and Geotechnical
The seismic vulnerability assessment was carried out for the same three levels of earthquake as for the First Narrows Crossing.

A geotechnical field investigation program was carried out on land at the Port Mann site and consisted of 4 mud-rotary drill holes, 5 Seismic Cone Penetration Tests (SCPT) and 1 auger hole. The results of the field investigation were used in support of the detailed geotechnical assessment of the Port Mann Crossing.

Free-field liquefaction assessment, similar to that previously described for the First Narrows Crossing, was carried out at various locations along the pipeline alignment. Results of analyses indicated that liquefaction is discontinuous with depth and is likely to occur at depths from 11 m to 33 m along the pipeline at the north shore of the Fraser River under the MCE event. The Fraser River sediments within the river would experience widespread soil liquefaction under the MCE event. At the south shore of the Fraser River, the liquefaction hazard was considered less severe compared to the north shore because the sandy soil at the south shore carries a fairly high fines content. Liquefaction hazard associated with the 1 in 475-year earthquake is less severe than the MCE event. No soil liquefaction was predicted on either shore under the 1 in 100-year earthquake.

As for the First Narrows Crossing, limit equilibrium analyses were conducted to assess the flow slide potential of the north and south river banks, and the soil slope within the Fraser River. Limit equilibrium analyses indicated that a flow slide is not likely to occur at either the north or south river bank as a result of soil liquefaction.

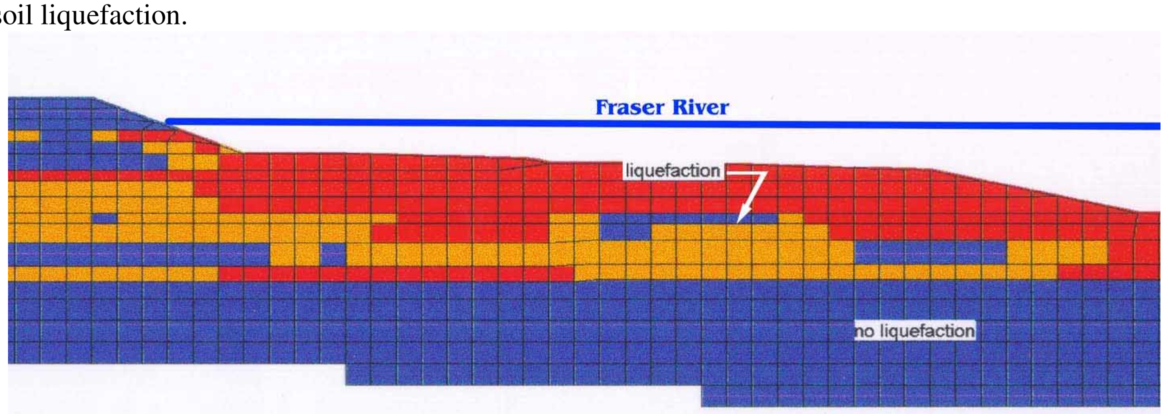

Figure 5 Port Mann Crossing – Geotechnical Model to Assess Liquefaction, North Half

As for the First Narrows Crossing, the earthquake induced ground deformations were computed using the multi-degree of freedom dynamic analysis computer programs VERSAT and FLAC and mean and upper-bound spring values were determined for use in the soil-structural analysis. The MCE horizontal ground displacements used in the structural analysis were in the order of 1.4m at the north shore and 0.6m at the south shore, both inwards towards the river.

Port Mann Crossing – Bell and Spigot Testing Results and Strain Limit Criteria
Based on literature review and discussions with local welders involved in the original fabrication, a replica of the bell and spigot joint was fabricated using original API 5L X52 pipe body which was removed during the 1998 repair project and subsequently stored by GVWD; see Figure 6.

Figure 6 Port Mann Crossing
Salvaged Piping used for Bell and Spigot Joint Fabrication

Typical fabrication misalignment was purposely built into the welded joint to simulate the original construction tolerances, with gaps up to 13mm as noted in Figure 7. Furthermore, 26 coupons were tested with the objective to assess the strength and ductility of the joints, as part of the procedure to develop the strain limit criteria; see Figure 8. Six strap coupon tests were carried out to specifically assess the behaviour of the bell and spigot joints.

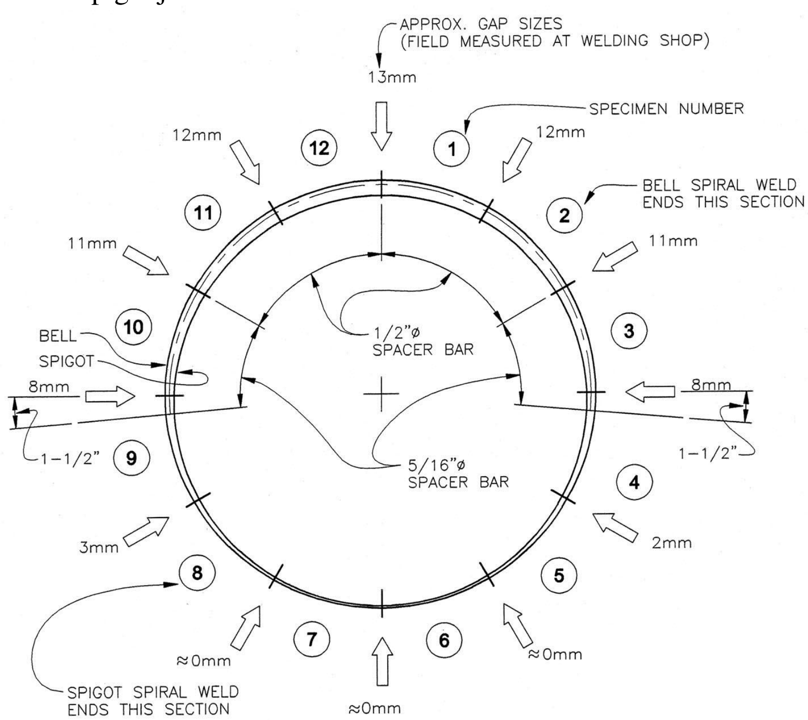

Figure 7 Bell and Spigot Joint Fabrication Misalignment

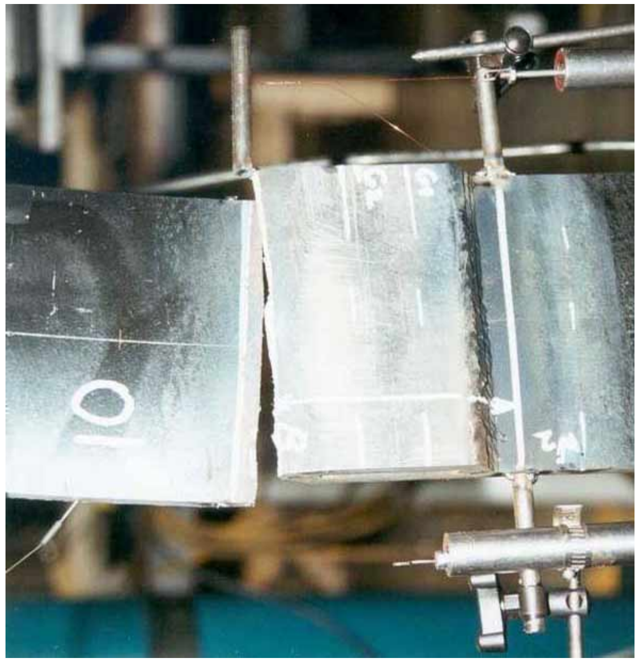

Figure 8 Typical Fractures of Bell and Spigot Test Coupons

Non-linear finite element ABAQUS models of each strap coupon was created to determine the peak local strains at the failure stress levels, and correlated with the actual strap coupon testing results. Three non-linear ABAQUS models of the complete pipe cylinder were then created to determine a limiting average tensile strain limit, calibrated to the strap coupon modeling results; see Figure 9.

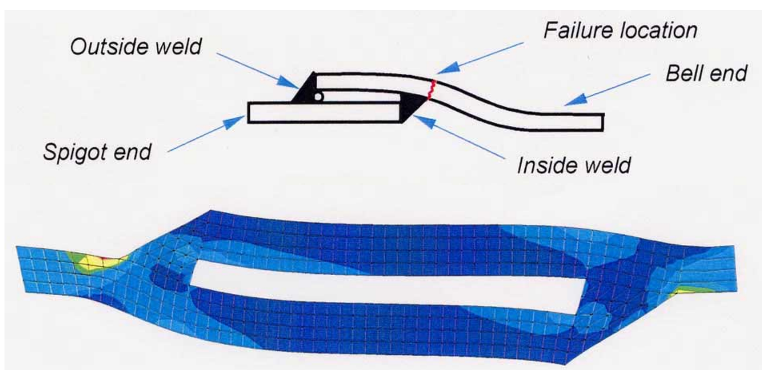

Figure 9 Fabricated Test Joint (top) compared to 3D Model Results showing High Localized Stress at observed Failure Location

The material testing program used the latest measuring equipment (extensometers, LVDT's, etc,) to measure load, displacement, and strain of all samples completely through to failure. Furthermore, the fractured test samples were inspected using a scanning electron microscope to characterize the nature of the fracture, brittle or ductile, and help determine why and where the fracture initiated. CFER specified the strain limit criteria for the original pipe and 2 newer sections of pipe in the crossing based on this testing and analysis, and their previous experiments and analyses. The compressive strain limits are governed by local buckling with the tensile strain limits governed by the girth weld ductility and local stress concentrations at the bell and spigot joints. For the current study for the 3 different pipe materials, the general compression strain limits are in the range of 0.23% to 0.28% and the general tensile strain limits are in the range of 0.22% for the bell and spigot joints to 0.50% to 0.75% for the pipe body and girth welds. However, the strain limit set for 3 joints of apparently poor condition in the 1998 replacement section, fabricated with single fillet weld butt strap joints, was set at 0.045%.

Port Mann Crossing – Pipeline Evaluation

The pipeline crossing was modeled by Sandwell in a single 3D model using the general purpose non-linear finite element analysis software ABAQUS. The steel pipelines were modeled using "PIPE32" elements, with element lengths approximately two pipe diameters. Non-linear stress-strain curves were input for each different pipe material, using the Ramberg-Osgood equation to characterize the curves. The pipe elements are supported at their ends by non-linear soil spring elements to simulate the support and movement of the surrounding soil; see Figure 10. Both upper bound and mean soil spring values were analyzed for each of the three earthquake scenarios. A program was written to calculate the geometry for each node of the structural model and to extrapolate the geotechnical data regarding soil displacement to all 600 nodes and 1800 soil springs including transformation of data from the global coordinate system used in the geotechnical model, to the local coordinate system (skewed in space) required for the structural model. Similarly, a program was specifically written to deal with over 30,000 data points of output regarding pipeline strain in order to evaluate the pipe behavior relative to the failure strain criteria in an effective and timely manner.

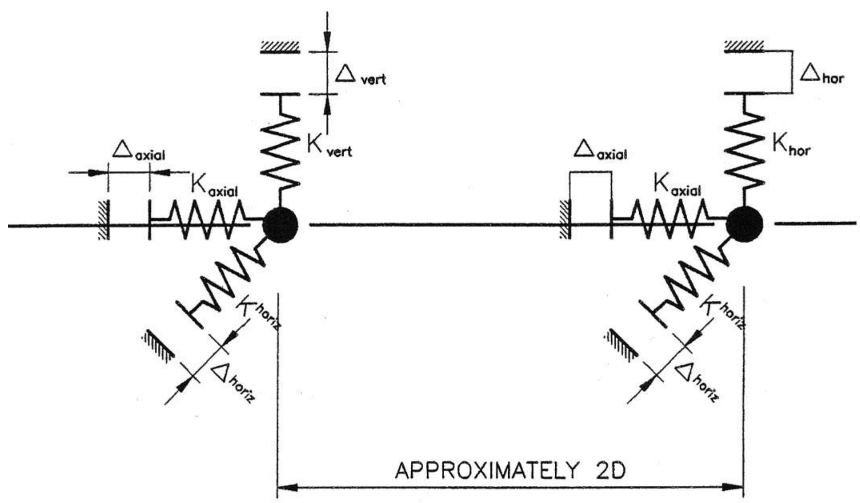

Figure 10 Typical Pipeline and Soil-Spring Modeling in ABAQUS

For the 1/100 event the analysis indicated no probable failure areas in the pipeline. For the 1/475 event two probable failure areas are at joints and a bend located under the river only near the south shore. No problem areas exist on the north shore or north half of the river. Prior to accepting probable "failure" at these locations it is recommended that more detailed review of historical records regarding their fabrication be undertaken. If these areas prove stronger than assumed then the strain limits may not be exceeded.

For the MCE event with mean soil springs the two main failure areas are a long section in compression under the river off the north shore and the 1998 replacement pipe section under the river near the south shore. All other areas exceed the limits marginally (< 10%) or could be termed 'conditional' failures where the strain limits may not be exceeded if:

- 1998 replacement joint detail is better than assumed
- bend beneath river on south shore is stronger than assumed

- no bell and spigot joints used near bends beneath drainage ditches (could be confirmed by local excavation).

Thus, the best scenario would be that remediation for two sections beneath the river, one on each side, is required. However, unless further information regarding joint details and fabrication are determined, remediation for six areas is recommended.

For the MCE event with upper-bound springs the best scenario would require remediation for four areas at the drainage ditch area, the main north bend, the section beneath the river on the north side and the upward bend on the south shore, even if the other uncertainties were proven better than assumed.

Port Mann Crossing - Remediation
Based on the results of the geotechnical and structural assessment a number of seismic remediation options were considered to upgrade the probable failure locations of the pipeline. Both geotechnical and structural schemes were considered, and order-of-magnitude costs developed:

Search of current technology suitable for structure remediation of the pipeline was carried out including:

- carbon fibre and epoxy bonded to the interior of the pipe to strengthen the pipe body or joints; this has been used successfully in the USA;
- flexible joints that can accommodate some 0.5 metres of elongation and 12 degrees of rotation to allow the pipe to deform along with the soil; to date such joints for 1.2 metre diameter pipes that can be fully buried have been installed in California and Singapore, however smaller diameter installations have been used extensively in the USA and successfully protected pipelines in recent earthquakes; and
- in-situ pipe welded repairs completed internal to the fully-flooded pipe, requiring swimming with full scuba gear some 500 m inside the pipe to reach the repair locations; such work would incorporate significant planning, coordination and safety controls to meet both local regulatory and GVWD requirements.

Remediation for 1/475 Event
- Further investigation of GVWD records to confirm 1998 joint details and details at south shore upward bend: if inconclusive then consider remediation by either
- geotechnical stabilization of south slope ($250,000 - $800,000), or
- replacement of weak sections of pipeline ($300,000) plus external repair of joints ($500,000), or
- installation of flex-joints ($900,000).

Remediation for MCE Event
- Further investigation of GVWD records to confirm 1998 joint details and details at south shore upward bend, plus local excavation to confirm details under drainage ditch area; if inconclusive then consider remediation.
- South shore
  - geotechnical stabilization of south shore ($250,000 - $800,000), or
  - replacement of weak sections of pipeline ($700,000) plus external repair of joints ($500,000), or
  - installation of flex-joints ($900,000)
- North shore
  - geotechnical stabilization of north shore (~$1.2 M), or
  - installation of flex-joints ($800,000)

In total, the estimated cost to provide remediation suitable to protect the integrity of the crossing for the MCE event is about $2 M. Due to the high cost, other options such as a new tunnel through non-liquefiable soils has since been considered.

CONCLUDING REMARKS

The consultant team combined expertise of three principal companies: EBA for all geotechnical and project aspects, Sandwell for the overall structural analysis and evaluation, and C-FER for the material testing and analysis to specify the failure limits. Workshops that brought together all key client and team staff , peer reviewers, and technical resources of GVWD resulted in a thorough understanding of all aspects of the project.

Some highlights of the assessment include:

- locating a limited amount of pipe material salvaged from a recent repair and developing a testing program to advance the understanding of the pipeline behavior under severe loading conditions;
- advancing the state of knowledge regarding the behavior of bell-and-spigot joints by constructing a replica of the Port Mann joint incorporating anecdotal descriptions of historical construction methods obtained from original welders, following a thorough literature review of previous testing of bell-and-spigot joints
- carrying out both coupon testing of a replica bell-and-spigot joint and verification by computer simulation of the testing to extrapolate the behavior to the actual full size pipe joints;
- developing specific deformation based "failure criteria" for many different pipe materials and joint types, while accounting for the vast uncertainty inherent to the process of fabrication and field installation of the original pipelines. This was completed despite no relevant documentation records of the construction;
- prediction of the complex behavior of the soil around the pipelines using two independent computer simulations, all based on limited geotechnical soil data;
- evaluation of current seismic mitigation procedures suitable to eliminate the potential for failure, including the use of carbon-fibre reinforcing of the joints, the use of flexible pipe joints that can accommodate significant elongation and rotation; in-pipe underwater welded repairs, and soil stabilization schemes.

During the course of the project, there was no disruption to the operating system, nor to the parks and rail yard during the on-land geotechnical drilling and testing program. Potentially disruptive offshore drilling was avoided by making use of existing data following an extensive search for such material. The remediation schemes considered would essentially become unnoticeable after the work is complete, and are designed to minimize both long term and temporary construction impacts to the parks and industrial operations on each side of the crossings. The remediation schemes will ensure no catastrophic release of chlorinated water into either the Fraser River or Burrard Inlet following an earthquake, and sustain the critical water supply to the majority of the Greater Vancouver region.

Further details regarding the assessments can be found in the consultant's reports to GVWD [12] [13].

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