Challenges in Assessing the Seismic Vulnerability of two Water Main River Crossings in British Columbia, Canada

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SUMMARY:
This paper presents the results of ground response and pipe-soil interaction analyses undertaken to assess the seismic vulnerability of two key water mains crossing a narrow river and supplying potable water to communities in the Fraser Lower Mainland, British Columbia, Canada. The methodology followed by the project team to establish the site soil conditions, seismic ground displacements, interaction of the lateral spread movements with pipeline, applicable strain criteria to define loss of pressure integrity and continued operation of pipe joints are presented and the challenges faced discussed.

Keywords: Pipeline, Seismic Vulnerability, Water Mains, Pipe-Soil Interaction, Seismic Ground Deformations

1. INTRODUCTION

Metro Vancouver delivers potable water to over 2 million people in the Fraser Lower Mainland, British Columbia, Canada via an extensive system of reservoirs, pump stations and over 500 km of supply mains. Haney Mains Nos. 2 and 3 (HM2 and HM3) that are the subject of this paper are key components of the water delivery system. The mains, 914 mm and 1,372 mm in diameter, cross the relatively narrow Coquitlam River (See Figure 1) where they are buried in shallow trenches. The occurrence of a large earthquake is expected to cause soil liquefaction and lateral spreading of the river banks causing damage and disrupting the supply of water to the communities. In 2009, Metro Vancouver commissioned studies to assess the impact of a large earthquake on the mains and to establish remedial measures required to minimize the risk of loss of functionality of the mains.

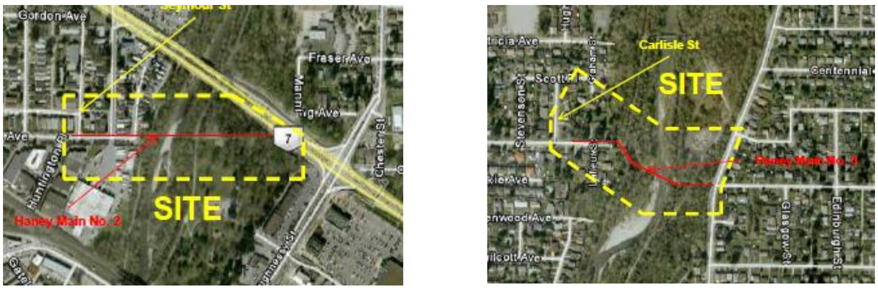

Figure 1. Locations of Pipeline Crossings HM2 and HM3

The seismic vulnerability assessment of large-diameter water mains is a subject of active interest among practicing engineers and academicians. Historical evidence from past earthquakes indicates that seismically-induced permanent ground displacements are the single largest cause of damage to buried pipelines. Data from major earthquakes such as Loma Prieta (1989) and Kobe (1995) indicate that the damage to the water mains is dependent on a number of key factors (see Figure 2, after Kitaura & Miyajima, 1996):

- Peak ground acceleration – damage rate increases with increasing ground accelerations;
- Type of soil deposit which is related to liquefaction susceptibility – damage rate is higher in reclaimed land, alluvial soils, and terrace beds where there is a high risk of soil liquefaction; and
- Type of pipe and condition of pipe joints.

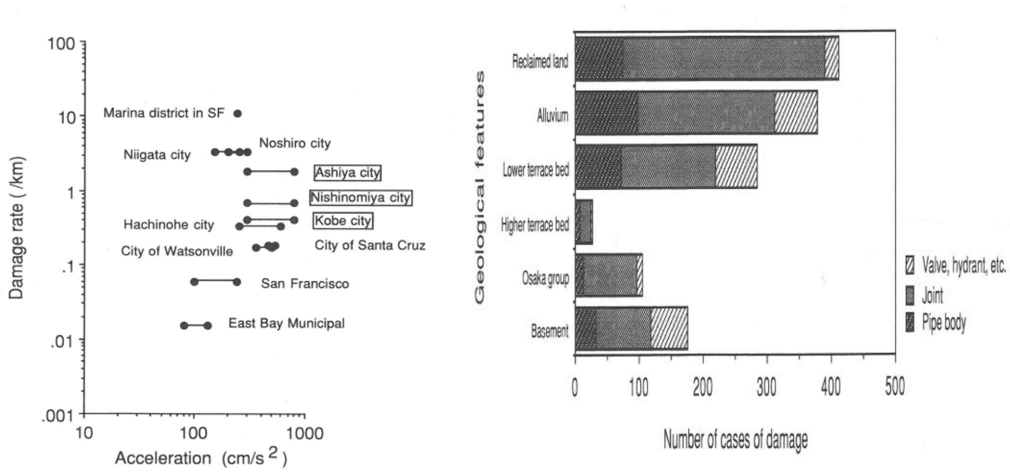

Figure 2. Pipeline Damage Statistics for Water Mains -Past Earthquakes (After Kitaura and Miyajima)

It is important to note that the pipe joints often form the weakest links in a given pipeline system rather than the pipe body itself.

This paper presents some of the methodology followed, outcome of analyses, and challenges faced by the project team in the assessment of seismic vulnerability of HM2 and HM3.

2. PIPELINE AND INSTALLATION DETAILS

The HM2 crossing was constructed in 1962 and consists of a 914 mm diameter steel pipe with 6.4-mm wall thickness, fabricated from A36 steel plate (yield strength of 248 MPa). The pipeline profile at the river crossing was constructed using pulled bell-and-spigot joints to form a parabolic curve with a chord length of approximately 140 m. Approximately 15 m of HM2 is encased in a 1067-mm concrete pipe where it crosses Lougheed Highway. The newer HM3 crossing was constructed in 1994. At the river crossing, the pipeline has an outside diameter of 1,372 mm and a wall thickness of 12.1 mm, and was fabricated with ASTM A-139, Grade E steel plate (yield strength of 358 MPa). HM3 crosses the river in a series of straight pipe sections with mitre bends used to provide changes in plan and profile. HM3 is lined and coated with coal tar enamel to AWWA C-203. The pipelines are not cathodically protected against corrosion.

Both pipelines were fabricated using welded slip joints and installed by open cut methods. After pipe installation, the trenches in untraveled areas were backfilled with suitable random fill product from the trench excavations.

3. SITE SOIL CONDITIONS AND STRATIGRAPHY

Geotechnical field investigations were undertaken to obtain information on the subsurface conditions. A number of different drilling techniques were utilized due to the presence of coarse-grained soils including cobbles and boulders in the upper layers, underlain by sands and fine-grained soils. These consisted of Becker Penetration, Cone Penetration, and combined Odex/Cone Penetration tests carried down to depths varying from 6 m to 35 m below existing ground surface. The soil stratigraphy and key properties of the substrata in the vicinity of the river crossings are shown in Figures 3 and 4.

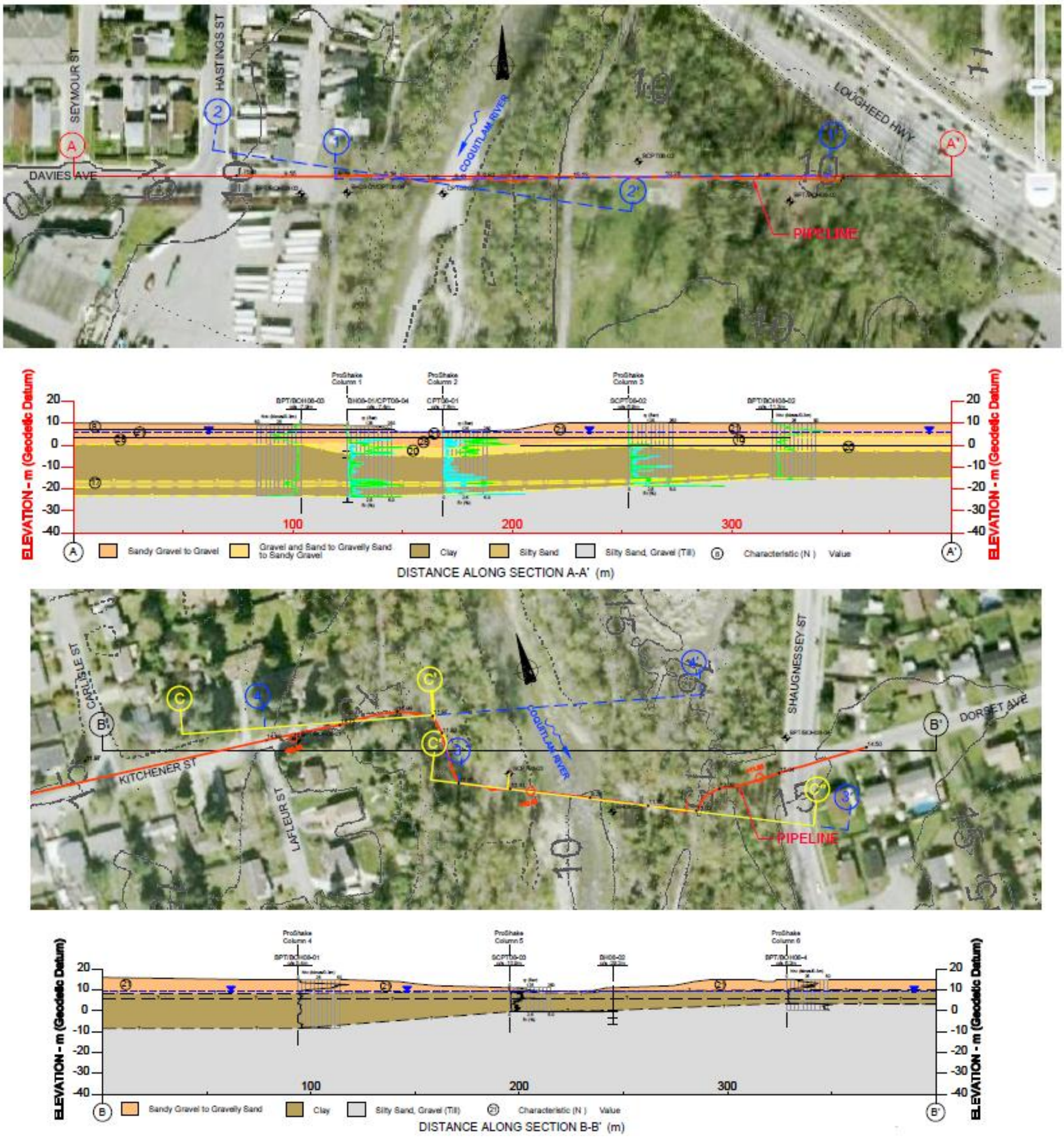

Figure 4. Plan Location and Soil Stratigraphy at HM3

4. SEISMIC PERFORMANCE EXPECTATIONS

According to Metro Vancouver's Seismic Design Criteria, the submarine pipelines at the Coquitlam River Crossings are classified as Level 1 structures. A Level 1 structure is critical to system operation, has little or no redundancy, and may result in substantially reduced service for an extended period of time (months) if failure occurs. Level 1 structures are required to be designed for Immediate Occupancy when subjected to ground motions that correspond to the 475-yr demand and for Life Safety when subjected to ground motions that correspond to the 10,000-yr (MCE) demand.

In addition to assessing the seismic vulnerability for the 475-yr and 10,000-yr demands, at the request of Metro Vancouver, the overall seismic performance of the HM2 and HM3 was also analysed for the 100-yr and 2,475-yr seismic demands.

5. DESIGN SITE-SPECIFIC SEISMIC GROUND MOTIONS

Site-specific ground motion parameters were established for the 100-yr, 475-yr, 2,475-yr, and 10,000-yr demands. The site-specific uniform hazard response spectrum (UHRS) derived from the information downloaded from the Geological Survey of Canada interactive website, developed as part of the fourth generation seismic hazard maps (GSC Open File 4459, 2004), was adopted for the 475-yr demand. These ground motions correspond to a site Class C (360 m/s < Vs < 760 m/s). The response spectrum that is applicable for the 100-year demand was established by uniformly scaling the spectrum for the 475-yr demand with respect to the Peak Ground Acceleration (PGA).

The 2,475-yr and 10,000-yr demands correspond to ground motions at a till outcrop with a shear wave velocity (Vs) of 620 m/s, generated by a magnitude Mw7.25 earthquake at a distance of 55 km from the site. Both the 2,475-yr and 10,000-yr ground motions were originally developed by Abrahamson (2006) as input to the seismic design of the nearby Port Mann Water Supply Tunnel. The moment magnitudes (M_w), firm-ground PGAs, and input firm-ground spectral accelerations (Sa) at selected periods that correspond to the design earthquake demands are summarized in Table 1.

Table 1. Firm-Ground (Class C) Acceleration Response Spectra for Design (5% Damping)

Earthquake Scenario	Moment Magnitude (M_w)	PGA_{Firm_Ground}	Sa at 0.1 Seconds	Sa at 0.3 Seconds	Sa at 1.0 Seconds
100-yr Demand	6.5	0.12g	0.19g	0.21g	0.07g
475-yr Demand	7.0	0.26g	0.42g	0.45g	0.16g
2,475-yr Demand	7.25	0.46g	0.86g	0.95g	0.44g
10,000-yr Demand	7.25	0.70g	1.33g	1.48g	0.70g

Applicable acceleration time-histories were established by modifying recorded acceleration time-histories from past earthquakes using spectral matching to the target uniform hazard response spectra.

6. PIPELINE ACCEPTANCE CRITERIA

Acceptance criteria for submarine pipeline analyses are generally defined in terms of longitudinal tension and compression strains. It is well-understood in the pipeline industry that the welded slip joints commonly used in the water industry to construct pipelines create an off-set in the pipe wall that makes the pipeline highly susceptible to wrinkling and severe strength degradation under longitudinal compression. Also, variation in joint geometry such as actual stab-in depth, joint alignment, and weld quality can substantially influence the strain capacity of welded slip joints. Given that the actual joint configurations for the existing pipeline are not known, it was not possible to estimate the strain capacity with a high degree of confidence.

Acceptable strain criteria were conservatively established for two scenarios; *pressure-integrity and continued-operation.* An assessment of the response of modest mitre bends concluded that the modest mitre bends will not likely behave substantially worse than the typical straight pipeline slip joint. The strain criteria established for HM2 and HM3 are summarized in Table 2:

Table 2. Strain Criteria for Slip Welded and Welded Pipe Joints

Main No.	Strain Criterion[1]	Slip Welded Pipes		High Quality Full Penetration Girth-Welded Pipes	
		Pressure-Integrity	Continued-Operation	Pressure-Integrity	Continued-Operation
HM2	Compression Strain Limit	0.5%	0.07%	1.2%	0.12%
	Tension Strain Limit	0.5%	0.3%	2%	1%
HM3	Compression Strain Limit	0.5%	0.12%	1.6%	0.29%
	Tension Strain Limit	0.5%	0.3%	2%	1%

[1] tension and compression strains averaged over a length of pipe equal to one pipe diameter.

An evaluation of the as-is condition of pipe joints of an operational pipeline that was constructed many years ago and cannot be taken out of service for inspection and testing is impractical. Therefore, the seismic vulnerability assessment assumed that the pipes themselves were in good condition with the intent to qualitatively gauge the effects of actual conditions based upon past operating experience with HM2, HM3, and mains of similar construction and age operated by Metro Vancouver.

7. EARTHQUAKE-INDUCED GROUND DISPLACEMENTS

Defining the extent to which seismically induced ground displacements at river banks propagate back from the crest of the bank or shoreline is difficult. Although it is feasible that these displacements can propagate to a considerable distance from the shoreline, field observations (Bartlett & Youd, 1992) suggest that liquefaction-induced displacements tend to reduce to relatively small values within about 200 to 300 m landward from the shoreline.

For a given site, mechanics-based numerical methods provide the most powerful and practical way of estimating or predicting seismically-induced displacements and patterns of displacement. Geological cross sections and geotechnical models are often developed using the soil conditions encountered at the banks of the river, within the river and some distance outside of the river banks. Assumptions are made on the geological conditions that exist in the zones where test hole information does not exist. The resulting estimates of displacements and patterns of displacements reflect these assumptions and often require careful interpretation.

The analyses were undertaken in two phases. 1D analyses were undertaken initially to assess sensitivity of soil response to ground motions, soil stratigraphy, stability of river banks, and for selection of critical input motions for the Phase 2 detailed analyses. Due to space limitations, Phase 1 results are not presented in this paper.

2-D coupled stress-flow deformation analyses were performed using the computer code FLAC to quantify the permanent ground deformations anticipated for the different seismic demands. FLAC models the domain of interest by a collection of elements or zones. The behaviour of each element in response to the applied forces and boundary conditions in the FLAC model is dictated by a prescribed stress-strain law. In addition program subroutines can be used to implement specific constitutive relations to model phenomena such as liquefaction and the associated stiffness and strength reductions.

The characteristic behaviour of the potentially liquefiable coarse-grained materials under cyclic loading conditions was modelled with the constitutive model referred to as UBCSAND. This model was selected because it is capable of capturing liquefaction phenomena and liquefied soil behaviour. UBCSAND is an effective stress, nonlinear, elasto-plastic constitutive model developed by Dr. Peter M. Byrne and his colleagues at the University of British Columbia. The model was specifically developed for non-linear, effective stress analysis of liquefiable soils and has been implemented in the computer code FLAC and calibrated against laboratory element tests and centrifuge tests on embankments (e.g., Puebla et al., 1997, Byrne et al., 2004). The seismic response of the fine-grained soils underlying the site was modelled using the constitutive model UBCHYST, also developed by Dr. Byrne and his colleagues. The post-earthquake vertical settlements included both shear and volume change induced deformations. The latter settlements were estimated from empirical correlations.

The geological cross-sections used for the 2-D ground response analyses are based on the stratigraphic units shown on Figure 4, Section A-A’ for the HM2 crossing, and Figure 4, Section B-B’ for the HM3 crossing. However, the ground surface profile of the 2-D model for the HM3 crossing was developed by combining Sections C-C’ and C’-C” shown in plan on Figure 4. The purpose of combining these two, non coplanar, ground surface profiles in a single 2-D model was to take into account the direction of the highest slope gradients within the area where the pipeline crosses Coquitlam River. The capability of the 2-D model for capturing this 3-D topographic feature is considered key to the analysis because stability problems and maximum ground displacements tend to occur in the direction of the maximum slope gradient. Otherwise, a truly 3-D model would be required to capture the topographic features at the site of the HM3 crossing and the geometric characteristics of the pipeline alignment. In contrast, the HM2 pipeline alignment is straight and the highest slope gradients diverge from the pipeline alignment only at the west bank, where the inclination of the slope in the direction of the maximum slope gradient is only about 1º greater than in the direction of the pipeline alignment. Hence, the ground surface profile of the 2-D model for the HM2 crossing is based on that of Section A-A' (Figure 3). The ground surface profiles of both 2-D models are based on topographic survey data, complemented with available elevation contour data. The 2-D models extended about 150 m west and east from the crest of the river banks.

8. PIPE-SOIL INTERACTION ANALYSIS

Loads are induced in a buried pipeline whenever it is subjected to movement relative to the surrounding soil. This may occur when the soil restricts the free movement of a pipeline or when the pipeline attempts to resist the movement of the surrounding soil. Figure 5 shows the deformed shape of a buried pipeline subject to horizontal ground displacement similar to what can occur at the boundary of a soil slide. The ground movement causes the pipeline to displace laterally with respect to the soil, which results in the soil applying pressure on the pipeline and the pipeline pushing away the soil. This loading imparts longitudinal strains and curvature to the pipeline on both sides of the boundary of ground displacement.

The longitudinal strain condition in a buried pipeline crossing a zone of ground movement varies directly with soil restraint conditions, i.e., the greater the resistance of the soil to the relative displacement of the pipeline within the soil mass, the more concentrated the loads become at the location of ground movement, and the larger the pipe strains must become to conform to any abrupt discontinuities in the ground movement pattern.

Ground displacement imposes a deformation-limited load on buried pipelines. That is, the soil loads exist only when there is relative displacement between the pipeline and the surrounding soil. The analysis of soil-pipeline interaction effects for permanent ground displacement requires the utilisation of analytical procedures that can account for in-elastic pipeline behaviour, the non-linear behaviour of the surrounding soil mass, and large displacement effects. Non linear finite element procedures are generally appropriate for this type of analysis. The advantage of using the finite element procedure is that it allows a more complete assessment of the behaviour of the pipeline. Stresses, strains, and displacements can be determined for virtually any number of locations of interest, and the sensitivity of the results to parameter variation can be investigated with relative ease. However, as with most non linear finite element applications, care must be exercised to control problem complexity, as convergence of the non-linear solution can be difficult to achieve at high strain and large displacements, particularly considering the potential for pipeline buckling under compressive loads.

8.1 Soil Restraint/Spring Model

The three-dimensional soil restraint on pipe response can be represented schematically by a series of discrete springs (see Figure 6). The ground displacement pattern is input to the model as displacements of the base of the soil springs. These springs represent the non-linear, stress-dependent behaviour of soils in the axial, transverse horizontal and transverse vertical directions, respectively. The formulation of non-linear load-deformation relations provides the most accurate characterization of soil-pipe interaction, but bi-linear representations are generally sufficient for large displacement problems. In some cases, such as pure horizontal ground displacement and a straight pipeline alignment, the model can be simplified by a two-dimensional representation, but three-dimensional models are preferred for crossings involving three significant components of ground displacement or more complicated pipeline alignments. A three-dimensional model was used for the assessment of seismic hazards for the subject river crossings. Inelastic pipeline behaviour was simulated by specifying a nonlinear stress-strain curve for the pipeline steel.

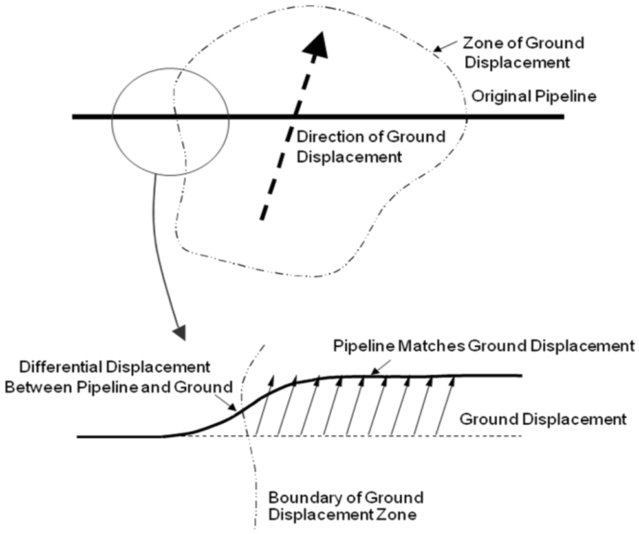

Figure 5. Pipeline Response to Ground Movements

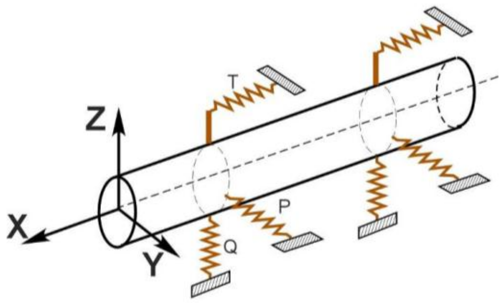

Figure 6. Soil Restraint to Pipe Movement Modelled Using 3D Non-Linear Springs

8.2 Pipeline Model

The pipeline was analyzed by the finite element procedure by dividing into a number of straight or curved pipe elements, which varied in length along the line. The segment of the pipeline used in the model must be long enough to appropriately characterize the behaviour at the location of ground movement, *i.e.*, it should extend beyond points of virtual anchorage on each side of the zone of ground displacement (the points where axial soil friction is sufficient to oppose axial forces generated by ground displacement). Elements were made shorter in regions of critical interest near the boundary of the zone of ground movement, and longer in segments of the pipeline that were more remote from the area of ground movement. For the subject crossings, the pipeline element lengths were approximately equivalent to one pipe diameter for the entire model.

The finite element analysis of the pipeline was performed using the ANSYS computer program. ANSYS is a widely-accepted general purpose finite element stress analysis program that has the capability to account for geometric, material, and boundary condition non-linearities. Specifically, non-linear spring elements were used to simulate soil restraints and plastic pipe elements were used to represent the pipeline. The large displacement analysis option of ANSYS was invoked to account for the geometric changes in stiffness due to large transverse movements of the pipe. Details on the use and capabilities of ANSYS can be found in the program user's manual (ANSYS, Inc., 1999).

The analyses utilized two elements available in ANSYS, a plastic straight pipe element (PIPE20) and a nonlinear spring element (COMBIN39). PIPE20 is a uniaxial element with tension-compression, bending, and torsion capabilities. The element has six degrees of freedom at each node: translations in the nodal *x*, *y*, and *z* directions, and rotations about the nodal *x*, *y*, and *z* axes. The element input data include two nodes, the pipe outer diameter and wall thickness, optional stress factors, and the isotropic material properties. The element has plastic, creep and swelling capabilities. The PIPE20 element is assumed to have "closed ends" so that the axial pressure effect is included. Options are available for outputting element forces plus bending stress, direct stress, elastic strain, and plastic strain for any of eight, equally-spaced, locations around the circumference of the pipe. COMBIN39 is a unidirectional element with non-linear generalized force-deflection capability that can be used in any analysis. The element has longitudinal or torsional capability in one, two, or three-dimensional applications. The longitudinal option, which is applicable to pipe-soil interaction, is a uniaxial tension-compression element with up to three degrees of freedom at each node: translations in the nodal *x*, *y*, and *z* directions. The element has large displacement capability for which there can be two or three degrees of freedom at each node. The element is defined by two node points and a generalized piecewise-linear force-deflection curve. The COMBIN39 element is non-linear and requires an iterative solution.

9. RESULTS

Typical results showing the extent of soil liquefaction and ground displacements computed for HM2 are shown in Figures 7 and 8, respectively. The liquefaction potential of the only coarse-grained layer at the HM3 crossing was assessed to be low for all seismic demands because the layer is above the water table and because of its inherent, relatively high, hydraulic conductivity. For all practical purposes, soil zones developing an excess pore pressure ratios greater than 70% were considered to have liquefied. It can be seen that the soils below the river channel and a thin layer at a depth (of 27 to 29 m) have developed excess pore pressure ratios for HM2.

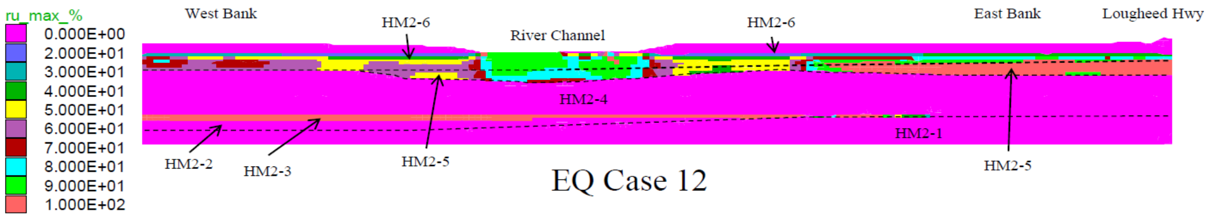

Figure 7. Typical Zones of Computed Maximum Excess Pore Pressure Ratio for HM2 (10,000-yr Demand)

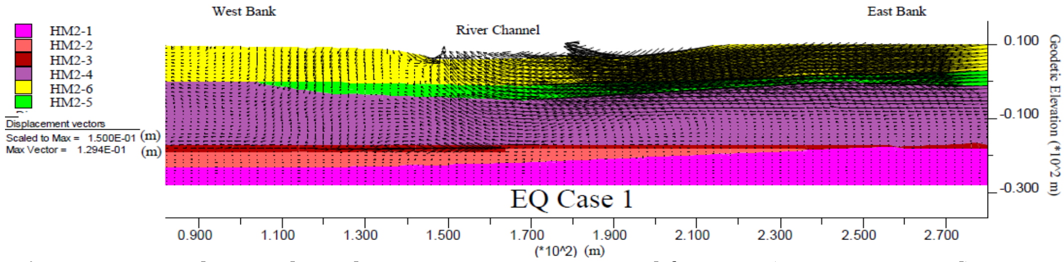

Figure 8. Typical Ground Displacement Patterns Computed for HM2 (2,475-yr Demand)

The coupled stress-flow deformation analyses conducted using FLAC resulted in computed maximum permanent ground surface displacements at the HM2 crossing ranging from about 0.1 to 0.2 m for the 2,475-year demand, and from about 0.4 to 0.5 m for the 10,000-yr demand. The analyses resulted in slightly smaller maximum permanent ground surface displacements at the HM3 crossing ranging from about 0.2 to 0.3 m for the 2,475-year demand, and from about 0.4 to 0.5 m for the 10,000-yr demand.

9.1 Profiles of Ground Displacement at Pipe Centreline

In order to analyse the soil-pipe interaction effects, permanent displacement profiles along the pipe centerline are required. From a 2D deformation analyses model, these displacements can be established relatively easily if the vertical pipeline profile is known and if the pipeline is straight in plan. Due to the numerous changes in direction of the HM3 alignment and due to compatibility with the input required for the pipeline structural analysis, quantifying the ground displacements along HM3 was complicated. Deriving the applicable displacements for HM2 was relatively straightforward, since the pipeline is straight in plan view.

The design ground displacement profiles used for pipeline structural analysis were developed by gradually decreasing the displacements to zero either by following the slope of the selected profile, or if the selected profile is approximately horizontal, by simply assuming a linear variation over a distance of 100 m. The liquefaction-induced settlements (where applicable) were developed separately using empirical methods and considered as input in addition to the horizontal movements. The resulting ground displacement patterns for one of the 10,000-yr earthquake time-histories are shown in Figure 9 for HM2. The decision to assign zero ground displacement at a distance of 100 m, although somewhat arbitrary, is necessary for the analysis of a linear system such as a pipeline, and is considered conservative. Typical results of pipelines to the applied permanent ground displacement patterns established for a typical input ground motion time-history are shown in Figure 10 for HM2. Similar analyses were carried out for HM3. In Figure 10, the results are presented in the form of maximum pipeline strain magnitude (absolute value of tension or compression strain) developed along the pipeline. The pipeline analyses were performed for selected ground displacement profiles considering the upper and lower bounds.

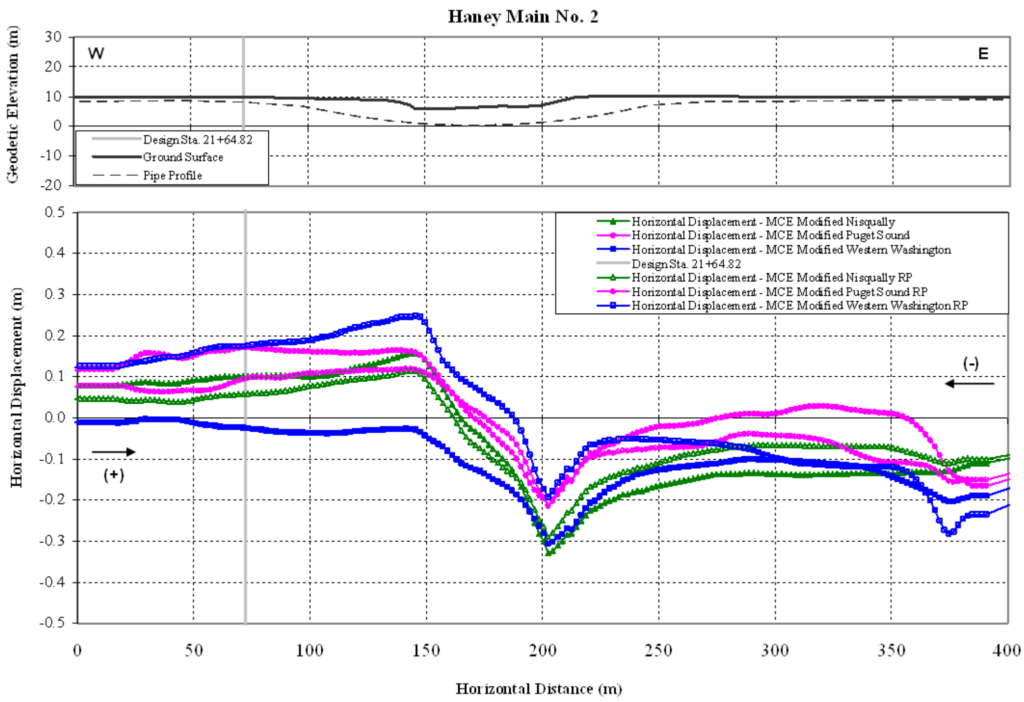

Figure 9. Computed Variations in Horizontal Displacement Profiles for HM2 (10,000-yr Demand)

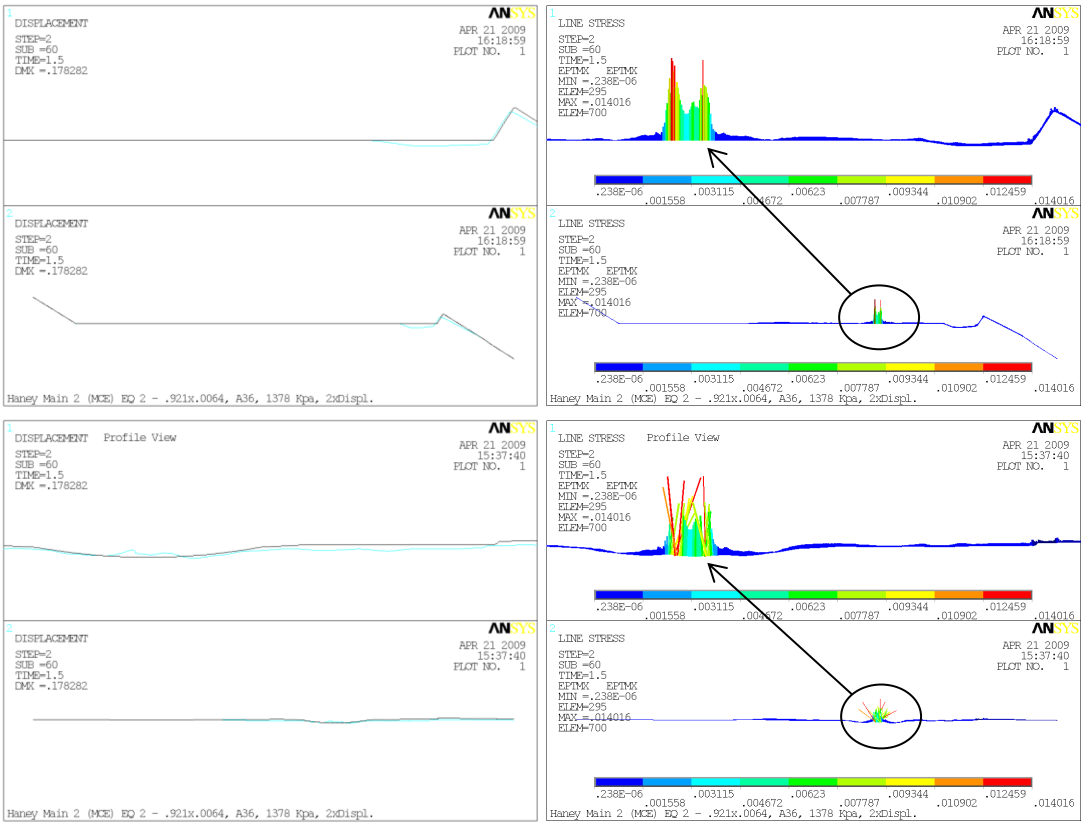

Figure 10. Typical Results from ANSYS Soil-Pipe Interaction Analyses for HM2 (10,000-yr Demand)

10. CONCLUDING REMARKS

The seismic performance of the HM2 and HM3 is assessed to be unsatisfactory under the 10,000-yr ground motions. Results from the pipeline analyses indicate that both pipelines are severely strained at a fraction of the 10,000-yr ground displacements. The most likely location of pipeline failure for the HM2 crossing is beneath the river. The most likely locations of pipeline failure for the HM3 crossing are the mitre bends at the sag-bends and over-bends of the pipeline.

The seismic performance of HM2 and HM3 under current conditions is also assessed to be unsatisfactory under the 2,475-year ground motions. The performance of HM3 is considered to be marginally unacceptable for ground displacement patterns associated with the 2,475-year ground motions given that the estimated maximum compressive strains are close to the 0.5% limit adopted for this assessment. Based on these findings and cost considerations, it was recommended to replace the pipelines at this river crossing to meet Metro Vancouver's seismic performance criteria. Temporary by-pass measures that would quickly establish the post-earthquake minimum flow required at these crossings were considered, but they were assessed to be impractical given that they rely on availability of emergency crews, access to site, and river flow conditions etc.

The seismic performance of HM2 and HM3, however, was assessed as meeting Metro Vancouver's seismic performance criteria when subjected to the 100-year and 475-year seismic demands under "as-is" conditions.

The results of the analyses confirm that pipelines crossing rivers underlain by potentially liquefiable soils that could cause lateral spreading of the ground are vulnerable to damage when subjected to strong ground shaking. This finding is in agreement with the historical damage data for pipelines, and is especially true for slip welded pipes that have low tensile and compressive strain capacity to induce damage. Establishing both the magnitude and pattern of ground movements caused by ground shaking and soil liquefaction at the pipe centerline are important in the assessment of seismic vulnerability. Establishing realistic ground movements along the pipe centerline was a challenge for HM3 due to the changes in pipeline location in plan and elevation. The assessment of ground movements requires 2D ground response analyses that consider both pre- and post-earthquake response of geological materials. Equally important are the soil-pipe interaction springs that dictate the loads imposed on the pipeline as a result of relative movements.

Since it is impractical to inspect an existing pipeline that is in operation, establishing conservative strain limits for pipe joint failure is important. Discretization of the pipeline into elements/zones and modeling the non-linear and inelastic response of the pipe and applying the kinematic loads incrementally was important in achieving convergence in the analysis, especially at large ground displacements where pipe buckling is predicted to occur.

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REFERENCES

Abrahamson, N. (2006). Report prepared for Metro Vancouver entitled Seismic Hazard Analysis for the Port Mann Tunnel, May 1, 2006.

Barlett, S.F., and Youd, T.L. (1992). Empirical analysis of horizontal ground displacement generated by liquefaction-induced lateral spread, *Technical Report NCEER-92-0021, National Center for Earthquake Engineering Research, Buffalo, New York*.

Byrne, P.M., Park, S-S., Beaty, M., Sharp, M., Gonzalez, L., and Abdoun, T. (2004). Numerical modeling of liquefaction and comparison with centrifuge tests, *Canadian Geotechnical Journal*, **41: 193** 211.

Dorey, A.B., Cheng, J.J.R., and Murray, D.W., 2001. Critical Buckling Strains for Energy Pipelines, Structural Engineering Report No. 237, University of Alberta, Department of Civil and Environmental Engineering, April.

PRCI (Pipeline Research Council International), 2004. Guidelines for the Design and Assessment of Natural Gas and Liquid Hydrocarbon Pipelines, report prepared by D.G. Honegger and D.J. Nyman, Catalog No. L51927.

Puebla, H., Byrne, P.M., & Phillips, R. (1997). Analysis of CANLEX liquefaction embankments: prototype and centrifuge models. *Canadian Geotechnical Journal*, **34(5)**, 641-657.